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**Federal Highway
Administration**

Bridge Inspector's Reference Manual

BIRM
Volume 1

NATIONAL HIGHWAY INSTITUTE
Training Solutions for Transportation Excellence

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Volume 2

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Volume 2

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Chapter 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 21^{st} Century
TRB	-	Transportation Research Board
TWG	-	Technical Working Group

Chapter 1
Bridge Inspection Programs

Topic 1.1 History of the National Bridge Inspection 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, the *Bridge Inspector's Reference Manual (BIRM)*, updates *Manual 90* and reflects over 20 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 (NBIP)**, it is helpful to review the development of the program.

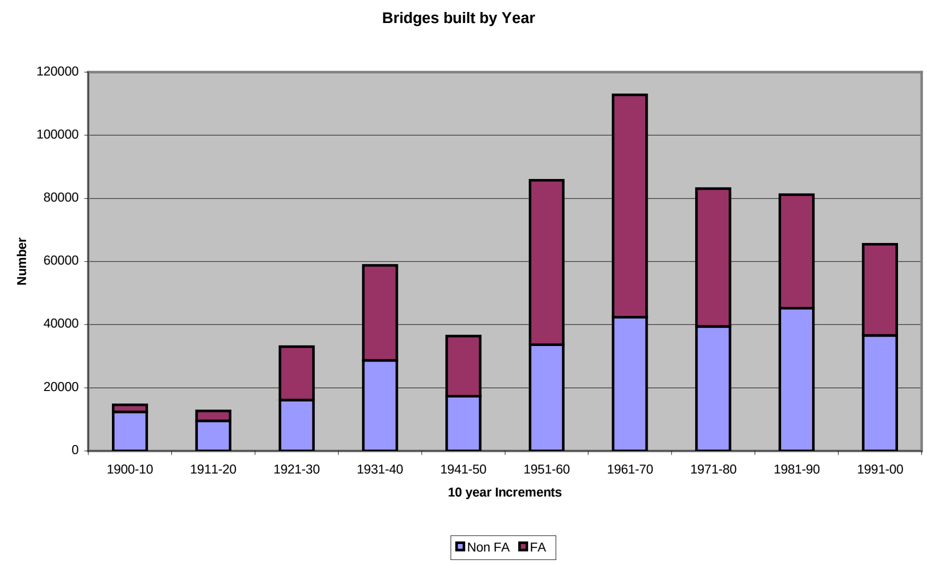

Figure 1.1.1 Number of Bridges Built since 1900

1.1.2 History of the National Bridge Inspection 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 2,235-foot Silver Bridge, at Point Pleasant, West Virginia, collapsed into the Ohio River on December 15, 1967, killing 46 people (see Figure 1.1.2).

Figure 1.1.2 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

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 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 over 593,000 bridges in the national inventory, approximately 86% were 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 forty-eight months 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 built on and improved 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. The *BIRM* was also updated in 2011.

On December 14, 2004, the revised NBIS regulation was published in the *Federal Register*. The updated NBIS took affect January 13, 2005. Implementation plans were to be developed by April 13, 2005 to be fully implemented by January 13, 2006.

The Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) was signed into law in August 2005. SAFETEA-LU represents the largest surface transportation investment in the Nation's history. SAFETEA-LU builds on and improves the initiatives established in ISTEA and TEA-21. Since being signed into law in August 2005, SAFETEA-LU has undergone several extensions past its original 2009 expiration, resulting in guaranteed funding until the end of Fiscal Year (FY) 2011. Multi-year legislation is expected to provide funding following FY 2011.

Over the years, varying amounts of federal funds have been spent on bridge projects, depending on the demands of the transportation infrastructure. Figure 1.1.3 illustrates the fluctuations in federal spending and shows current trends.

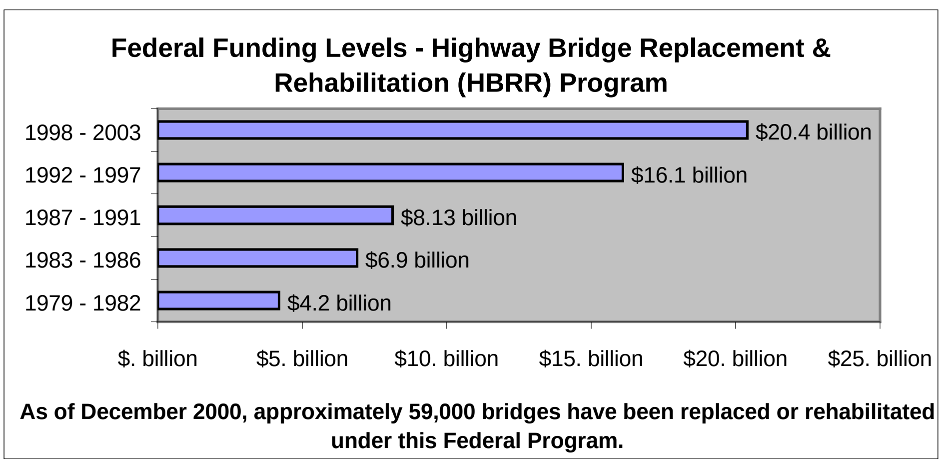

Figure 1.1.3 Federal Funding Levels (1979 – 2003)

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 2005 National Bridge Inspection Standards appear in Appendix A. The standards are divided into the following sections:

- Purpose
- Applicability
- Definitions
- Bridge inspection organization
- Qualifications of personnel
- Inspection frequency
- Inspection procedures
- Inventory
- Reference manuals

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 structure inspection through the National Highway Institute (NHI):

- "Engineering Concepts for Bridge Inspectors" (NHI Course Number FHWA-NHI-130054)

  This one-week course is a pre-requisite for FHWA-NHI-130055 and 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.

- "Introduction to Safety Inspection of In-Service Bridges" (NHI Course Number FHWA-NHI-130101)

  This web-based course is another possible pre-requisite for FHWA-NHI-130055 and presents engineering concepts, as well as inspection procedures and information about bridge types, bridge components, and bridge materials. The course is for new inspectors with little or no practical bridge inspection experience.

- "Safety Inspection of In-Service Bridges" (NHI Course Number FHWA-NHI-130055)

This two-week course is for 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 evaluation. Lessons include critical findings identification and inspection of fracture critical members (FCM's), underwater inspection, culverts, field trips, case studies, and non-destructive evaluation. Several special bridge types may also be discussed at the host agency's request.

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

  This three-day course provides a review of the National Bridge Inspection Standards (NBIS) and includes discussions on structure inventory items, structure types, and the appropriate codes for the Federal Structure, Inventory and Appraisal reporting. A three-and-a-half day option is also available, which includes additional case studies.

- "Underwater Bridge Inspection" (NHI Course Number FHWA-NHI-130091)

  This four or five-day course provides an overview of diving operations that will be useful to agency personnel responsible for managing underwater bridge inspections. This course also fulfills the requirement due to the latest changes of the National Bridge Inspection Standards, which require bridge inspection training for all divers conducting underwater inspections.

- "Underwater Bridge Repair Rehabilitation and Countermeasures" (NHI Course Number FHWA-NHI-130091A)

  This two-day course provides training in techniques for selecting and executing repairs to below water bridge elements. The primary goal is to enable design engineers to select, design, and specify appropriate and durable repairs to below water bridge elements. The secondary goal of the course is to train staff in effective construction inspection of below water repairs.

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

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

- "Bridge Inspection Non-Destructive Evaluation Showcase (BINS)" (NHI Course Number FHWA-NHI-130099)

This one-day course allows bridge inspectors to identify the components of handheld NDE inspection tools and techniques, inspection strategies and NDE techniques. Inspection tools will include eddy current, ultrasonic, infrared thermography, impact echo, and ground penetrating radar.

- "Stream Stability and Scour at Highway Bridges" (NHI Course Number FHWA-NHI-135046)

  This three-day course provides training in the prevention of hydraulic-related failures of highway bridges by identifying stream stability and scour problems at bridges and defining problems caused by stream instability and scour. The magnitude of scour at bridge piers and abutments and in the bridge reach will be estimated.

- "Stream Stability and Scour at Highway Bridges for Bridge Inspectors" (NHI Course Number FHWA-NHI-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).

- "Pontis Bridge Management" (NHI Course Number FHWA-NHI-134056)

  This two and one-half day course covers the entering and editing of inspection data, developing a bridge preservation policy, performing bridge network level analyses, developing bridge projects, running Pontis and Infomaker reports, and refining Pontis results.

- "Pontis Bridge Management and InfoMaker Module" (NHI Course Number FHWA-NHI-134056A)

  This three and one-half day course covers the entering and editing of inspection data, developing a bridge preservation policy, performing bridge network level analyses, developing bridge projects, running Pontis and Infomaker reports, and refining Pontis results. It also includes an overview of InfoMaker 9.0 as it relates to Pontis. It covers those aspects most used by the Pontis users as well as the ability to query data, create new report libraries, modify existing Pontis structure list layout, and modify an existing Pontis report.

- "Pontis Bridge Management InfoMaker Module" (NHI Course Number FHWA-NHI-134056B)

  This one-day course provides an overview of InfoMaker 9.0 as it relates to Pontis. It covers those aspects most used by the Pontis users as well as the ability to query data, create new report libraries, modify existing Pontis structure list layout, and modify an existing Pontis report.

- "Inspection and Maintenance of Ancillary Highway Structures" (NHI Course Number FHWA-NHI-130087)

This two-day course provides training in the inspection and maintenance of ancillary structures, such as structural supports for highway signs, luminaries, and traffic signals. Its goal is to provide agencies with information to aid in establishing and conducting an inspection program in accordance with the FHWA "Guidelines for the Installation, Inspection, Maintenance, and Repair of Structural Supports for Highway Signs, Luminaries, and Traffic Signals".

- "Inspection of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes" (NHI Course Number FHWA-NHI-132080)

  This three-day course is part of a series to develop a training and qualification/certification program for field inspectors. A partial list of lessons addressed in the course are MSE wall and RSS types and durability; construction methods and sequences; alignment control; methods of fill and compaction control; plans, specifications, and the geotechnical report; shop drawings; and safety.

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, 5th Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials, with 2010 Interims.
- 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, Errata Sheet 03/ 2004.
  - http://www.fhwa.dot.gov/bridge/mtguide.pdf
- FHWA. *Bridge Inspector's Reference Manual.* Washington, D.C.: United States Department of Transportation, 2002, Revised 2006, 2011.
- AASHTO. *Manual for Bridge Evaluation, Second Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials, 2011.
- AASHTO. *Guide Manual for Bridge Element Inspection.* Washington, D.C.: American Association of State Highway and Transportation Officials, 2011.

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Chapter 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

Topic 1.2 Responsibilities of the Bridge Inspector

1.2.1 Introduction

Bridge inspection has played, and continues to play, an increasingly important role in providing a safe infrastructure for the United States. As the 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 topic presents the responsibilities of the bridge inspector/engineer and qualifications for bridge inspection personnel.

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
- Maintain accurate bridge records
- Fulfill legal responsibilities

Maintain Public Safety and Confidence

The primary responsibility of the bridge inspector is to maintain public safety and confidence. The general public travels the highways and bridges without hesitation. However, when a bridge fails, the public's confidence in the bridge system is violated (see Figure 1.2.1).

The engineer's role is:

- To incorporate design safety factors.
- To provide cost-effective designs.
- To review and evaluate reports.
- Rate each bridge as to its safe load capacity

Engineers include a margin of safety in their designs to compensate for variations in the quality of materials and unknowns in vehicular traffic loadings through the life of the structure. In older bridges, especially those designed prior to the use of computer software programs and modern design codes, margin of safety also compensated for a lack of precise calculations and construction loading conditions.

The inspector's role is:

- To provide thorough inspections identifying bridge conditions and defects.
- To prepare condition reports documenting 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 Mianus Bridge Failure

Protect Public Investment

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

The current funding available to rehabilitate and replace deficient bridges is not adequate to meet 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 engineer's role is:

- To continually upgrade design standards to promote longevity of bridge performance such as the implementation of high performance materials and better performing bridge joints.

The inspector's role is:

- To continually be on guard for minor problems that can become costly repairs.
- To recognize bridge components that need repair in order to maintain bridge safety and avoid the need for costly replacement.
- To make recommendations to close a bridge if necessary.

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:

- Purpose
- Applicability
- Definitions
- Bridge inspection organization

- Qualifications of personnel
- Inspection frequency
- Inspection procedures
- Inventory
- Reference manuals

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 any user.

The Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) was signed into law in August 2005. SAFETEA-LU represents the largest surface transportation investment in the Nation's history. SAFETEA-LU builds on and improves the initiatives established in ISTEA and TEA-21. Since being signed into law in August 2005, SAFETEA-LU has undergone several extensions past its original 2009 expiration, resulting in guaranteed funding until the end of Fiscal Year (FY) 2011. Multi-year legislation is expected to provide funding following FY 2011. Information on SAFETEA-LU can be found on the FHWA website:

http://www.fhwa.dot.gov/safetealu/factsheets/bridge.htm

Maintain Accurate Bridge Records

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

a. A structure history file facilitates the identification and/or monitoring of deficiencies.

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

Year	Abutment A	Abutment B
2011	4-3/16"	3-1/2"
2009	4-3/16"	2-1/4"
2007	4-1/8"	1-1/8"
2005	4"	1"

   Looking at year 2011 measurements only indicate that Abutment A has a more severe problem. However, examining the changes each year, it is noted that the movement of Abutment A is slowing and may have stopped, while Abutment B is changing at a faster pace each inspection cycle. At the rate it is moving, Abutment B probably surpasses Abutment A by the next inspection.

b. To identify and assess bridge deficiencies and to identify and assess bridge repair requirements. Be able to readily determine, from the records, what repairs are needed as well as a good estimate of quantities. Maintain reports on the results of the bridge inspection together with notations of any action taken to address the findings of such inspections.

c. To identify and assess minor bridge deficiencies, to identify and assess bridge maintenance needs, and preservation needs in a similar manner to the repair requirements. Maintain relevant maintenance and inspection data to allow assessment of current bridge condition.

d. To be able to quickly obtain pertinent structure information to respond to emergency events such as fire on or below the structure, severe flooding, and navigational or vehicular collision.

e. To maintain load carrying capacity to facilitate the routing of overweight/over-height vehicles.

To ensure accurate bridge records, proper record keeping needs to be maintained. Develop a system to review bridge data and evaluate quality of bridge inspections. Bridge files are to be prepared as described in the *AASHTO Manual for Bridge Evaluation*. Record the findings and results of bridge inspections on standard State or Federal agency forms.

Fulfill Legal Responsibilities

A bridge inspection report is a legal document. Make descriptions specific, detailed, quantitative (where possible), and complete. Do not use vague adjectives such as good, fair, poor, and general deterioration, without concise descriptions to back them up. To say “the bridge is OK” is just not good enough.

Example of inspection descriptions:

Bad description: “Fair beams”

Good description: “Reinforced concrete tee-beams are 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 top surface area of the deck as indicated indicated on field sketch, see Figure 42.”

Bad description: “The bridge is dangerous”

Good description: “Section loss exists on Girder G5 at 10 feet north of centerline of bearing at Pier 1. Original flange thickness 1.5 inches. Measured thickness 0.991 inches.”

Include phrases such as “no other apparent defects” or “no other defects observed” in any visual assessment.

Do not alter original inspection notes 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. Use the proper equipment, methods, and qualified. If the inspection is a special or interim inspection, explained explicitly in the report.

Current NBIS Requirements

The National Bridge Inspection Standards (NBIS) are regulations that were first established in 1971 to set national requirements regarding bridge inspection frequency, inspector qualifications, report formats, and inspection and rating procedures.

The NBIS can be found in the Code of Federal Regulations, Part 65, Title 23, Subpart C which is on the Bridge Technology site located on the FHWA website:

http://www.fhwa.dot.gov/bridge/nbis.htm

The NBIS set minimum, nationwide requirements. States and other owner agencies can establish additional or more stringent requirements.

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, Part 650, Subpart C, Section 650.309, (23 *CFR* 650.309), lists the qualifications of personnel for the National Bridge Inspection Standards (Appendix B of this Manual). These are minimum standards; therefore, state or local highway agencies can implement higher requirements.

Program Manager

The program manager is in charge of the organizational unit that has responsibility for bridge inspection, reporting, and inventory. The minimum qualifications are as follows:

1) Be a registered Professional Engineer, or have ten years bridge inspection experience; and
2) Successfully complete a Federal Highway Administration (FHWA) approved comprehensive bridge inspection training course.

Team Leader

The team leader is responsible for planning, preparing, and performing the inspections of individual bridges as well as the day-to-day aspects of the inspection. NBIS calls for a team leader to be present at all times during each initial, routine, in-depth, fracture critical and underwater inspection. There are five alternative ways to qualify as a team leader:

1) Have the qualifications specified for the Program Manager; or
2) Have five years bridge inspection experience and have successfully completed an FHWA-approved comprehensive bridge inspection training course; or
3) Be certified as a Level III or IV Bridge Safety Inspector under the National Society of Professional Engineer's program for National Certification in Engineering Technologies (NICET) and have successfully completed an FHWA-approved comprehensive bridge inspection training course, or
4) Have the following:
    i) A bachelor's degree in engineering from a college or university accredited by or determined as substantially equivalent by the Accreditation Board for Engineering and Technology;

   ii) Successfully passed the National Council of Examiners for Engineering and Surveying Fundamentals of Engineering examination;
   iii) Two years of bridge inspection experience; and
   iv) Successfully completed an FHWA-approved comprehensive bridge inspection training course, or

5) Have the following:
   i) An associate's degree in engineering or engineering technology from a college or university accredited by or determined as substantially equivalent by the Accreditation Board for Engineering and Technology;
   ii) Four years of bridge inspection experience; and
   iii) Successfully completed an FHWA-approved comprehensive bridge inspection training course.

Inspector Qualifications

There are no specific federal guidelines for bridge inspectors. The main responsibility of a bridge inspector is to assist the team leader in day-to-day aspects of the inspection. Training is not required but it is recommended for non-team leaders. Any technical background is obtained through education and hands-on experience enables the inspector to successfully complete the tasks at hand. The goal is for the inspector to learn the correct inspection methods and to evaluate bridge components and elements consistently.

1.2.4 Liabilities

The dictionary defines tort as "a wrongful act for which a civil action 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. 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. 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 scrutinized to determine if conditions are documented thoroughly and for the "proper" reasons. Therefore, be as objective and complete as possible. State if something could not be inspected and the reason it was not 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 bridge railing, and fell 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 12 feet deep and 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 three to four 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 six 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. A total of 13 inspections were conducted on the structure. An engineering report accuses the county engineer of poor engineering practice.

1.2.5 Quality Control and Quality Assurance

The NBIS requires Quality Control (QC) and Quality Assurance (QA) procedures to maintain a high degree of accuracy and consistency in the highway bridge inspection program. Accuracy and consistency are important since the bridge inspection process is the foundation to the bridge management systems. FHWA has developed a recommended framework for a bridge inspection QC/QA program (see Topic 1.3).

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Chapter 1
Bridge Inspection Programs

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Topic 1.3 Quality Control and Quality Assurance

1.3.1 Introduction

Title 23, *Code of Federal Regulations (CFR)*, Part 650, Subpart C, Section 313, paragraph (g), Quality Control and Quality Assurance, requires each state to assure that systematic Quality Control (QC) and Quality Assurance (QA) procedures are being used to maintain a high degree of accuracy and consistency in their inspection program. The FHWA has developed a recommended framework for a bridge inspection QC/QA program to assist bridge owners in developing their QC / QA programs.

Accuracy and consistency of the data is important since the bridge inspection process is the foundation of the entire bridge management operation and bridge management systems. Information obtained during the inspection is used for determining needed maintenance and repairs, for prioritizing rehabilitations and replacements, for allocating resources, and for evaluating and improving design for new bridges. The accuracy and consistency of the inspection and documentation is vital because it not only impacts programming and funding appropriations, it also affects public safety.

1.3.2 Quality Control

Quality Control (QC) is the establishment and enforcement of procedures that are intended to maintain the quality of the inspection at or above a specific level. If an inspection program is decentralized, the state program manager is still responsible for QC.

1.3.3 Quality Assurance

Quality Assurance (QA) is the use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program. This is accomplished by the re-inspection of a sample of bridges by an independent inspection team. For decentralized state inspections or delegated inspection programs, the QA program can be performed by the central staff or their agent (e.g., consultants). If the inspections are centralized within the state, consultants or a division are to perform the QA program separate and independent of the inspection state 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 QC/QA program is a means by which periodic and independent 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 enhance the inspection program through improved inspection processes and procedures, training, and quality of the inspection report.

1.3.4 Quality Control and Quality Assurance Framework

The FHWA has developed the following recommended framework for a bridge inspection QC/QA program.

A. **Documentation of QC/QA Program**:

1. Develop, document, and maintain a bridge inspection manual that contains Quality Control/Quality Assurance (QC/QA) procedures in accordance with this recommended framework.
2. Elaborate on the purpose and benefits of the QC/QA program.
3. Provide appropriate definitions.

B. **Quality Control (QC) Procedures**

1. Define and document QC roles and responsibilities.
2. Document qualifications required for Program Manager, Team Leader, Team Member, Load Rater and Underwater Bridge Inspection Diver.
3. Document process for tracking how qualifications are met, including:
    a. Years and type of experience.
    b. Training completed.
    c. Certifications/registrations.
4. Document required refresher training, including:
    a. NHI training courses, other specialized training courses, and/or periodic meetings.
    b. Define refresher training content, frequency, and method of delivery.
5. Document special skills, training, and equipment needs for specific types of inspections.
6. Document procedures for review and validation of inspection reports and data.
7. Document procedures for identification and resolution of data errors, omissions and/or changes.

C. **Quality Assurance (QA) Procedures**

1. Define and document QA roles and responsibilities.
2. Document procedures for conducting office and field QA reviews, including:

a. Procedures for maintaining, documenting, and sharing review results; including an annual report.

b. Establish review frequency parameters. Parameters should include:

    i. Recommended review frequency for districts/units to be reviewed (e.g. review each district once every four years). Or establish number of districts/units to be reviewed annually.

    ii. Recommended number of bridges to review.

c. Procedures and sampling parameters for selecting bridges to review. Procedure should consider:

    i. Whether the bridge is or is not posted.

    ii. Bridge's deficiency status.

    iii. Whether the bridge is programmed for rehabilitation or replacement.

    iv. Whether the bridge has had critical findings and the status of any follow-up action.

    v. Bridges with unusual changes in condition ratings (e.g. more than 1 appraisal rating change from previous inspection).

    vi. Bridges that require special inspections (underwater, fracture critical, other special).

    vii. Location of bridge.

d. Procedures for reviewing current inspection report, bridge file, and load rating.

e. Procedures to validate qualifications of inspector and load rater.

f. Define "out-of-tolerance" for condition rating and load rating. (e.g. rating of +/- 1 or load ratings that differ by more than 15%)

g. Checklists covering typical items to review as part of QA procedures.

    i. Bridge file.

    ii. Field inspection.

    iii. Load rating analysis.

h. Others.

3. Document disqualification procedures for team leaders and consultant inspection firms that have continued record of poor performance.

4. Document re-qualification procedures for previously disqualified team leaders and consultant inspection firms that demonstrate they have acceptable performance.

5. Document procedures for conducting inspections on a "control" bridge.

6. Document procedures to validate the QC procedures.

Examples of Commendable State practices and additional resources regarding QC/QA programs are available at the following link: http://www.fhwa.dot.gov/bridge/nbis/nbisframework.cfm

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Chapter 2

Safety Fundamentals for Bridge Inspectors

Chapter 2
Safety Fundamentals for Bridge Inspectors

Topic 2.1 Duties of the Bridge Inspection Team

2.1.1 Introduction

Bridge inspection plays an important role in providing a safe infrastructure for the nation. As the 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 seven basic types of inspection:

- Initial (inventory)
- Routine (periodic)
- Damage
- In-depth
- Fracture critical
- Underwater
- Special (interim)

These inspection types are presented in Article 4.2 of the AASHTO *Manual for Bridge Evaluation*. Although this topic is organized for "in-depth" inspections, it applies to any inspection type. However, the amount of time and effort required for performing each duty vary with the type of inspection performed.

This topic 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.

2.1.2 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
- Communicate the need for immediate follow-up for critical findings

2.1.3 Planning the Inspection

Planning is necessary for a safe, efficient, cost-effective inspection effort which results in a thorough and complete inspection of in-service bridges.

Basic activities include:

- Determination of the type of inspection
- Selection of the inspection team, which includes a qualified team leader on site for all initial, routine, in-depth, fracture critical and underwater inspections. Although not required by NBIS, it is a good practice to provide a team leader for damage and special inspections.
- Evaluation of required activities (e.g., nondestructive evaluation, traffic control including use of flaggers, utilities, confined spaces, permits, hazardous materials such as pigeon droppings, lead paint and asbestos removal, etc.)
- Establishment of a schedule which includes the inspection duration

2.1.4 Preparing for 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:

- Review the bridge structure file
- Identify the components and elements
- Develop an inspection sequence
- Prepare and organize notes, forms, and sketches
- Arrange for temporary traffic control
- Arrange staging areas and access locations
- Reviewing safety precautions
- Organizing tools and equipment
- Arranging for subcontracting special activities
- Account for other special considerations

Review the Bridge Structure File

The first step in preparing for a bridge inspection is to review the available sources of information about the bridge, such as:

- Plans, including construction plans, shop and working drawings, and as-built drawings
- Specifications
- Correspondence
- Photographs
- Materials and tests, including material certification, material test data, and load test data
- Maintenance and repair history
- Coating history
- Accident records
- Posting
- Permit loads
- Flood and scour data
- Traffic data
- Inspection history
- Inspection requirements
- Structure Inventory and Appraisal sheets
- Inventories and inspections
- Rating records

Each of these sections of the bridge structure file is presented in detail in Topic 4.4.2.

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 2.1.1). If drawings or previous inspection reports are available, use the same identification system during the inspection as those used in these sources.

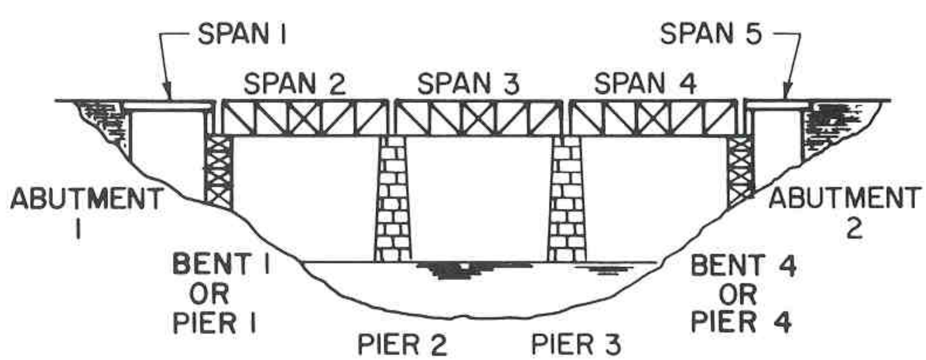

Figure 2.1.1 Sample Bridge Numbering Sequence

Establish an identification system if there are no previous records available. The numbering system presented in this topic is one possible system, but some states may use a different numbering system.

This route direction information can be used to identify the location of the bridge.

Route direction would be north, south, east or west. Mile markers, stationing or segments are the locations along the route. Location of the bridge can be identified by the route direction along with mile marker, stationing or segment information. The route direction can be determined based on mile markers, stationing, or segments, and use this direction to identify the location of the bridge.

Deck Element Numbering System

The deck sections (between construction joints), expansion joints, railing, parapets, and light standards are included in the deck element numbering system. Number these elements consecutively, from the beginning to the end of the bridge.

Superstructure Element Numbering System

The spans, the beams, and, in the case of a truss or arch, the panel points are included in the superstructure element numbering system. Number the spans consecutively, with Span 1 located at the beginning of the bridge. Multiple beams are to be numbered consecutively from left to right facing in the route direction. Similar to spans, floorbeams are also numbered consecutively from the beginning of the bridge, with the first floorbeam labeled as Floorbeam 0. This coordinates the floorbeam and the bay numbers such that a given floorbeam number is located at the end of its corresponding bay.

For trusses, number the panels 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 skip a number (see Figure 2.1.2). Some design plans number to midspan on the truss and then number backwards to zero using prime numbers (U9'). However, this numbering system is not recommended for field inspection use since the prime designations in the field notes may be obscured by dirt.

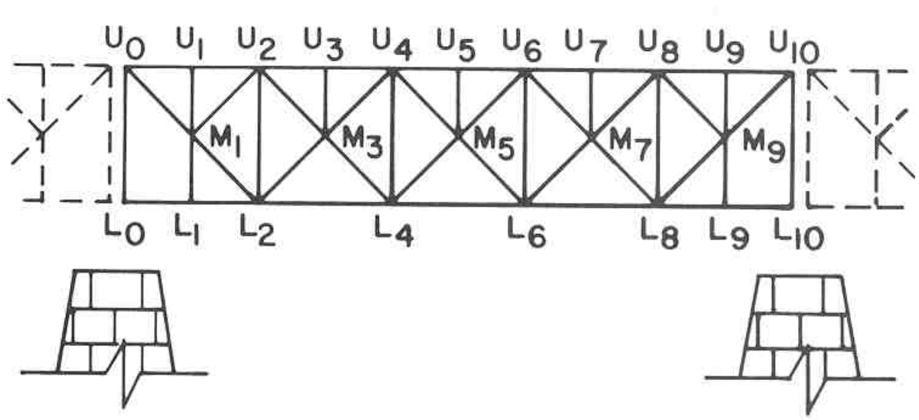

Figure 2.1.2 Sample Truss Numbering Scheme

Substructure Element Numbering System

The abutments and the piers are included in the substructure element numbering system. Abutment 1 is located at the beginning of the bridge, and Abutment 2 is located at the end. Number the piers consecutively, with Pier 1 located closest to the beginning of the bridge (see Figure 2.1.2). Alternatively, the substructure units may be numbered consecutively without noting abutments or piers.

AASHTO Bridge Elements

The *AASHTO Guide Manual for Bridge Element Inspection* provides a comprehensive set of bridge elements, designed to be flexible in nature to satisfy needs of all agencies. This set of elements capture the components necessary for any agency to manage the aspects of the bridge inventory and allows the full utilization of a Bridge Management System (BMS).

There are two different element types included in the element set which are identified as National Bridge Elements (NBEs) or Bridge Management Elements (BMEs). These two element sets combined comprise the full AASHTO element set.

Develop Inspection Sequence

An inspection normally begins with the deck and superstructure elements and proceeds to the substructure. However, there are many factors to 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 considerations

A sample inspection sequence for a bridge of average length and complexity is presented in Figure 2.1.3. While developing an inspection sequence is important, it is of value only if following it ensures a safe, 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: top and bottom
- Expansion joints
- Sidewalks and railings
- Drainage
- Signing
- Electrical-lighting
- Barriers, gates, and other traffic control devices

3) Superstructure Elements

- Primary load-carrying members
- Secondary members and bracings
- Utilities and their attachments
- Anchorages
- Bearings

4) Substructure Elements

- Abutments
- Piers
- Footings
- Piles
- Curtain walls
- Skewbacks (arches)
- Slope protection

5) Channel and Waterway Elements

- Channel profile and alignment
- Channel streambed
- Channel embankment
- Channel embankment protection
- Hydraulic opening Fenders
- Water depth scales
- Navigational lights and aids
- Dolphins
- Hydraulic control devices

Figure 2.1.3 Sample Inspection Sequence

Prepare and Organize Notes, Forms, and Sketches

Preparing notes, forms, and sketches prior to the on-site inspection reduces work in the field. Obtain copies of the agency's standard inspection form for use in recordkeeping and as a checklist to ensure that the condition of all elements is noted.

Create copies of 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 inspection sketches or design drawings are not available, then pre-made, generic sketches may be used for repetitive features or members. Possible applications of this timesaving method 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.

Arrange for Temporary Traffic Control

Bridge inspection, like construction and maintenance activities on bridges, often presents motorists with unexpected and unusual situations (see Figure 2.1.4). 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, check and follow the governing standards. These standards prescribe the minimum methods for a number of typical applications and the proper use of standard traffic control devices, such as cones, signs, and flashing arrow boards.

Figure 2.1.4 Temporary Traffic Control Operation

Principles and methods, which enhance the safety of motorists and bridge inspectors in work areas, include the following:

- Traffic safety is a high priority element on every bridge inspection project where the inspectors' activities are exposed to traffic or likely to affect normal traffic movements.
- Route traffic through work areas with geometrics and traffic control devices comparable to those employed for other highway situations.
- Inhibit traffic and pedestrian movement as little as practicable.
- Guide approaching motorists in a clear and positive manner throughout the bridge inspection site.
- On long duration inspections, perform routine inspection of temporary traffic control devices.
- Adequately train personnel responsible for the performance of temporary traffic control operations.

In addition, schedules may have to be adjusted to accommodate temporary 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. Some agencies require inspections to be performed during low tow traffic (i.e. at night).

Special Considerations

Time Requirements

The total time required to complete an inspection can vary from what may be documented on a previous inspection report or separately in the bridge file due to the various tasks for completing the inspection. Breaking down and recording the time to complete the various tasks (office preparation, travel, on-site, report preparation) separately benefits future planning and preparation efforts. Break down the inspection time requirements in to office preparation, travel time, field time, and report preparation. The overall condition of the bridge plays a major role in determining how long an inspection takes. 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, so adjust the schedules accordingly.

Set-up Time

Consider set-up time 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. Provide adequate time in the schedule for set-up and take-down time requirements. Also, consider the time to install and remove temporary traffic control devices.

Access

Consider access requirements 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, avoid climbing on the bridge structure. An increased awareness of safety hazards is required, and keeping notes dry can be difficult. During seasons of poor weather, adopt a less aggressive schedule 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. Review general safety guidelines for inspection and any agency or bridge specific safety precautions such as for hazardous material and confined space entry. Confined space entry methods are in accordance with OSHA and the owners' requirements. For climbing inspections, the three basic requirements covered in topic 2.2.5 for safe climbing are to be followed. For additional information about safety precautions, refer to Topic 2.2.5.

Permits

When inspecting a bridge crossing a railroad, obtain an access permit before proceeding with the field inspection. Also obtain a permit when inspecting bridges passing over navigable waterways. Environmental permits and permits to work around endangered species may be required for some bridges and bridge sites.

Tools

To perform a complete and accurate inspection, use the proper tools and equipment. Bridge location and type are two main factors in determining required tools and equipment. Refer to Topic 2.4 for a complete list of inspection tools and equipment.

Subcontract Special Activities

Give consideration to time requirements when special activities are 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
- Non-destructive evaluation/testing

2.1.5 Performing the Inspection

This duty is the on-site work of accessing and examining bridge components and waterway, if present.

Perform inspections in accordance with the *National Bridge Inspection Standards (NBIS) and AASHTO Manual for Bridge Evaluation (MBE)*.

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, 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, 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. Include the compass directions, the direction of waterway flow, and the direction of the inventory route in the orientation. Also record inspection team members, air temperature, weather conditions, and time.

After the site orientation has been established, the inspector is ready to begin the on-site inspection. Be careful and attentive to the work at hand, and do not overlook any portion of the bridge. Give special attention to those portions that are most critical to the structural integrity of the bridge. (Refer to Topic 6.4 for a description of fracture critical members in steel bridges.)

Combine the prudence used during the inspection with thorough and complete recordkeeping. Careful and attentive observations are to be made, and record every deficiency. A very careful inspection is worth no more than the records kept during that inspection.

Place numbers or letters on the bridge by using crayon or paint 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.

Note the general approach roadway alignment, and sight along the railing and edge of the deck or girder to detect any misalignment or settlement.

Approaches and Decks

Check the approach pavement for unevenness, settlement, or roughness. Also check the condition of the shoulders, slopes, drainage, and approach guardrail.

Examine the deck and any sidewalks for various deficiencies, noting size, type, extent, and location of each deficiency. Reference the location 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 (load restrictions), and lighting are present, and note their condition.

Superstructures

Inspect the superstructure thoroughly, since the failure of a primary load-carrying member could result in the collapse of the bridge. The primary method of bridge inspection is visual, requiring the removal of dirt, leaves, animal waste, and debris to allow close observation and evaluation of the primary load-carrying members. The most common forms of primary load-carrying members are:

- Beams and girders
- Floorbeams and stringers
- Trusses
- Cables (suspension, stay, suspender)
- Eyebar chains
- Arches
- Frames
- Pins and hanger assemblies

Bearings

Inspect the bearings thoroughly, since they provide the critical link between the superstructure and the substructure. The primary method of bearing inspection is a visual inspection, which requires removing dirt, leaves, animal waste, and debris to allow close observation and evaluation of the bearings. Record the difference between the rocker tilt and a fixed reference line, noting the direction of tilt, the air or bearing material 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 "design" or "as-built" plans are available, compare the dimensions of the substructure units with those presented on the plans. Since the primary method of bridge inspection is visual, remove the dirt, leaves, animal waste, and debris to allow close observation and evaluation. Check the substructure units for settlement by sighting along the superstructure and noting any tilting of vertical faces. In conjunction with the scour inspection of the waterway, check the substructure units for undermining, noting both its extent and location.

Culverts

Inspect culverts regularly to identify any potential safety problems and maintenance needs. Examine the culvert for various deficiencies, noting size, type, extent, and location of each deficiency. Reference the location using the centerline. In addition to the inspection of the culvert and its components, look for high-water marks, changes in drainage area, scour, and settlement of the roadway.

Waterways

Waterways are dynamic in nature, with their volume of flow and their path continually changing. Therefore, carefully inspect bridges passing over waterways for the effects of these changes.

Maintain a historical record of the channel profile and cross-sections. Record and compare current measures to initial (base line) measures, 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 is the removal of material from the streambed or streambank as a result of the erosive action of streamflow. Scour is the primary concern when evaluating the effects of waterways on bridges (see Figure 2.1.5). Determine the existence and extent of scour using a grid system and noting the depth of the channel bottom at each grid point.

Figure 2.1.5 Inspection for Scour and Undermining

Note the embankment erosion both upstream and downstream of the bridge, as well as any 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

There are several general terms used to describe bridge deficiencies:

- Corrosion – section loss
- Cracking - breaking away without separating in to parts
- Splitting - separating in to 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 Chapter 6 for a more detailed list and description of types and causes of deterioration for specific materials. As described in Chapter 6, each material is subject to unique deficiencies. Therefore, be familiar with the different inspection methods 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 deficiencies, avoid speculation as to the cause and be factual. 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. Also record any rust or efflorescence stains. Concrete scaling can occur on any exposed face of the concrete surface, so record its area, location, depth, and general characteristics. Inspect concrete surfaces for delamination or hollow zones, which are areas of incipient spalling, using a hammer or a chain drag. Carefully document any delamination using sketches showing the location and pertinent dimensions.

Unlike delamination, spalling is readily visible. Document any spalling 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 reinforcement.

Metal Inspection: Steel, Iron and Others

When inspecting metal structures, determine the extent and severity of corrosion, carefully measuring the amount of cross section remaining. Note all cracks, recording their length, size, and location. Document all bent or damaged members, 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 can be felt if it is loose. In addition, note any missing rivets or bolts.

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, rocker bearings lean towards the fixed end of the bridge, while in hot weather, they lean away from the fixed end. A locked bearing is generally caused by heavy rust on the bearing elements.

For the evaluation to be substantiated, document and record all inspection findings. Documentation is referred to as the “condition remarks” on the inspection form or in the inspection report.

Masonry Inspection

The examination of stone masonry and mortar is similar to that of concrete. Carefully inspect the joints for cracks and other forms of mortar deterioration. Inspection techniques are generally the same as for concrete.

Check masonry arches or masonry-faced concrete arches for mortar cracks, vegetation, water seepage through cracks, loose or missing stones or blocks, weathering, and spalled or split blocks and stones.

Fiber Reinforced Polymer Inspection

When inspecting Fiber Reinforced Polymers (FRP), note any blistering, voids and delaminations, discoloration, wrinkling, fiber exposure and any scratches. Document all visible cracks, recording their width, length, and location.

Critical Findings

Critical findings are any structural or safety-related deficiency that requires immediate follow-up inspection or action. When a critical finding is discovered, immediately communicate and document the critical finding according to agency procedures.

Refer to Topic 4.5 for a detailed description of critical findings and the methods required to address any critical findings discovered.

2.1.6 Preparing the Report

Documentation is essential for any type of inspection. Gather enough information to ensure a comprehensive and complete report. Report preparation is a duty, which reflects the effort that the inspector puts in to performing the inspection. Both documentation and preparation are to 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 the bridge, roadway and waterway components and elements
- Recommendations and cost estimates (refer to Topic 2.1.7 for further details)
- Summary

A sample bridge inspection report can be found in Appendix B of this manual. Follow the procedures of the agency responsible for the bridge.

2.1.7 Identifying Items for Preservation and Follow-up for Critical Findings

Another common duty is to identify work recommendations for bridge preservation and follow-up to critical findings. Recommend work items that promote public safety and maximize useful bridge life. Refer to Topic 4.5 for details on follow-up to critical findings.

Work recommendations are commonly aligned with an agency's bridge preservation program and are included in preservation work plans. These work recommendations are condition driven or cyclical. Examples of preservation activities include: deck or bridge washing, flushing the scuppers and down spouts, lubricating the bearings and painting the structure.

Carefully consider the benefits to be derived from completing the work recommendation and the consequences if the work is not completed. Also, check the previous report recommendations to see what work was recommended and the priority of such items. If work was scheduled to be completed before the next inspection, note if the work was completed and the need for any follow-up work.

The NBIS regulation requires the establishment of a statewide or Federal agency wide procedure to assure that critical finds are addressed in a timely manner. Additionally, the NBIS requires that FHWA be periodically notified of actions taken to resolve or monitor critical findings. The duty of the inspection team is to follow statewide or Federal agency-wide procedures for the follow-up on critical findings. It is the responsibility of Bridge Owners to implement procedures for addressing critical deficiencies, including:

- Immediate critical deficiency reporting steps
- Emergency notification of police and the public
- Rapid evaluation of the deficiencies
- Rapid implementation of corrective or protective actions
- A tracking system to ensure adequate follow-up
- Provisions for identifying other bridges with similar structural details for follow-up inspections

Critical findings are presented in detail in Topic 4.5.

2.1.8 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 seven types of inspections identified in the AASHTO Manual for Bridge Evaluation are described below and 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. The purpose of this inspection is two-fold. First, an initial 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 scour and 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. Follow the plan of action for scour critical bridges.

According to the NBIS, inspect each bridge at regular intervals not to exceed 24 months. However, certain bridges require inspection at less than the 24-month interval. Establish criteria to determine inspection frequency and intensity based on such factors as age, traffic characteristics, and known deficiencies. Certain bridges may be inspected at greater than 24-month intervals, not to exceed 48 months, with prior FHWA-approval. This may be appropriate when past inspection findings and analysis justifies the increased inspection interval.

Damage

A damage inspection is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions. The scope of inspection is sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic and to assess the level of effort necessary for an effective repair.

In-Depth

An in-depth inspection is a close-up, inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures. Hands-on inspection may be necessary at some locations. 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. This type of inspection can be scheduled independently of a routine inspection, though generally at a longer interval, or it may be a follow-up for other inspection types. For small bridges, the in-depth inspection includes 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.

According to the NBIS, establish criteria to determine the level and frequency of this type of inspection.

Fracture Critical

A fracture critical member (FCM) inspection is performed within arm's length of steel members in tension, or with a tension element, who failure would probably cause a portion of or the entire bridge to collapse. The FCM inspection uses visual methods that may be supplemented by nondestructive testing. A very detailed visual hands-on inspection is the primary method of detecting cracks. This may require that critical areas be specially cleaned prior to the inspection and additional lighting and magnification be used. Other nondestructive methods may be used at the discretion of the Bridge Owner. Where the fracture toughness of the steel is not documented, some tests may be necessary to determine the threat of brittle fracture at low temperatures.

According to the NBIS, fracture critical members (FCMs) are to be inspected at regular intervals not to exceed 24 months. However, certain FCMs require inspection at less than 24-month intervals. Establish criteria to determine the inspection level and frequency to which these members are inspected considering such factors as age, traffic characteristics, and known deficiencies.

Underwater

An underwater inspection is the inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate procedures. Underwater inspections are an integral part of a total bridge inspection plan. Scour evaluations are conducted for all bridges over water. Determine the severity and extent of scour, immediately communicating and documenting critical findings. Follow the plan of action for scour critical bridges.

Structural damage, scour and erosion due to water movement, drift, streambed load, ice loading, navigation traffic collision, and deleterious effects of water movement or of elements, are typical occurrences that could result in the decision to conduct underwater inspections at shorter intervals.

According to the NBIS, underwater structural elements are inspected at regular intervals not to exceed 60 months. However, certain underwater structural elements require inspection at less than the 60-month intervals. Establish criteria to determine the level and frequency to which these members are inspected considering such factors as construction material, environment, age, scour characteristics, condition rating from past inspections and known deficiencies. Certain underwater structural elements may be inspected at greater than 60-month intervals, not to exceed 72 months, with written FHWA-approval. This may be appropriate when past inspection findings and analysis justifies the increased inspection interval.

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. These inspections are not usually comprehensive enough to meet NBIS requirements for routine inspections.

According to the NBIS, establish criteria to determine the level and frequency of this type of inspection. Guidelines and procedures on what to observe and/or measure are provided, and a timely process to interpret the field results is in place.

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Chapter 2

Safety Fundamentals for Bridge Inspectors

Topic 2.2 Safety Fundamentals for Bridge Inspectors

2.2.1 Importance of 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, bridge inspectors 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. In the case of major disability, the burden of caring for the injured person falls on family members.

- Equipment damage - The repair or replacement of damaged equipment can be very costly. There is also a cost associated with the loss of time while the equipment is not available for use.

- Lost production - The employer loses revenues associated with the employee's work, and also loses time and money spent on safety training and equipment. Training additional inspectors to replace the injured worker contributes to lost production. Lost production also affects the bridge owner in terms of loses in revenue, time and money if a bridge is closed longer than expected after an inspection accident.

- Medical expenses - Whether coverage is an employee benefit, personal insurance, or out of pocket, someone has to pay for medical expenses. Ultimately, everyone is impacted by accidents through higher insurance premiums.

Constantly be aware of safety concerns. Spending the effort to be safe pays big dividends in avoided expenses and grief.

2.2.2 Safety Responsibilities

The employer is responsible for providing a safe working environment, including:

- Clear safety regulations and procedures
- Safety training
- Proper tools and equipment

The supervisor is responsible for maintaining a safe working environment, including:

- Supervision of established job procedures
- Training in application of safety procedures
- Training 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 – Recognize your limitations and communicate them to your supervisor and inspection team members.
- Knowledge of rules and requirements of job – Verify that you understand a particular task and that you are qualified to perform that task. If a procedure appears to be unsafe, question it and constructively try to develop a safer procedure.
- Safety of fellow workers – Do not act in a manner that endangers fellow inspectors. Warn co-workers if they are 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. Promptly report any injury in order to assure coverage, if necessary, under workmen's compensation or other insurance.

2.2.3 Personal Protection

Proper Inspection Attire

It is important to dress properly for the job. Be sure to wear field clothes that are properly sized and appropriate for the climate. For general inspection activities, wear boots with traction lug soles. For climbing of bridge components, wear boots with a steel shank (with non-slip soles without heavy lugs), as well as 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. Use the equipment correctly in order for it to provide protection. The following are some common pieces of safety equipment:

Hard Hat

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.

It is a good practice to always wear a hard hat (see Figure 2.2.1). Also, if the inspector is free climbing, it is a good practice to wear a chinstrap with the hard hat.

Figure 2.2.1 Inspector Wearing a Hard Hat

Reflective Safety Vest

When performing activities near traffic, the inspector is required to wear a safety vest. Be sure the vest conforms to current OSHA and MUTCD standards. The combination of bright color and reflectivity makes the inspector more visible to passing motorists. Safety is improved when the motorist is aware of the inspector's presence (see Figure 2.2.2).

Figure 2.2.2 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 be worn when climbing (no bifocals).

Wear eye protection during activities such as:

- Using a hammer
- Using a scraper or wire brush
- Grinding
- Shot or sand blasting
- Power tools

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 protect the inspector's hands from harmful effects of deteriorated members (see Figure 2.2.3). 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.

Figure 2.2.3 Inspector Wearing Safety Goggles and Gloves

Life Jacket

Always wear a life jacket when working over water or in a boat (see Figure 2.2.4). 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 keeps the inspector afloat. Also wear a life jacket when wearing hip or chest waders. If an inspector slips or steps in an area that is too deep, their waders can fill with water and drag them under, making swimming impossible.

Figure 2.2.4 Inspector Wearing a Life Jacket

Dust Mask / Respirator

A respirator or dust mask can protect the inspector from harmful airborne contaminants and pollutants (see Figure 2.2.5). Consult agency or OSHA regulations 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 histoplasmosis, a potentially very serious illness)
- Work in closed or constricted areas
- Hammering, scraping or wire brushing steel members with lead based paints.

Figure 2.2.5 Inspector Wearing a Respirator

Safety Harness and Lanyard

The safety harness and lanyard is the inspector's lifeline in the event of a fall (see Figure 2.2.6). Use this equipment as required by conditions. Make sure you satisfy agency and OSHA requirements.

For example, some agencies require a safety harness be worn in the following situations:

- At heights over 20 feet
- Above water
- Above traffic

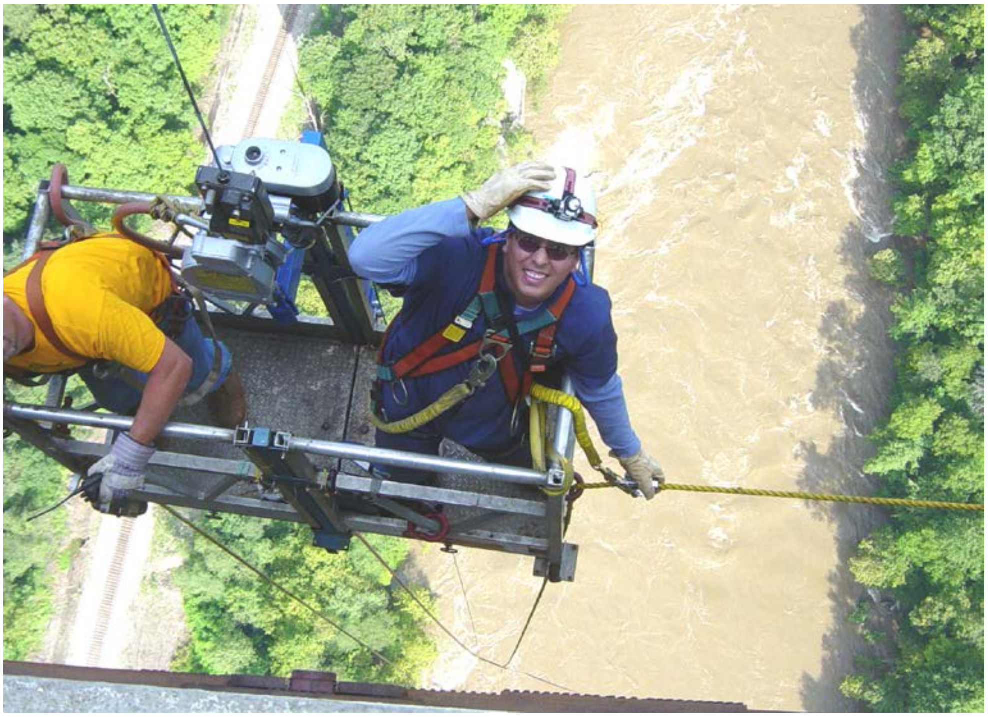

Figure 2.2.6 Inspector with Safety Harness with a Lanyard

To reduce the possibility of injury, the maximum lanyard length limits a fall to 6 feet per OSHA regulations. Further protection can be achieved using a shock absorber between the lanyard and the safety harness. The shock absorber reduces g-forces through the controlled extension of nylon webbing, which is pre-folded and sewn together. Two lanyards are required with one lanyard being tied off to a solid structural member or to a safety line rigged always for this purpose. Use the second lanyard to allow safe movement around obstacles connecting the second lanyard before disconnecting the first lanyard in order to safely move along the structure.

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 under bridge inspection vehicle or bucket truck, tie off to the structure if possible. Exercise extreme caution 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.

Boats/Skiff

There must be one rescue person present and specifically assigned to respond to water emergencies at all times when the inspection over water is active. Whenever possible, a manned boat/skiff should be in the water. Whenever possible, a manned boat/skiff should be in the water. In the event of an accident in which someone was to fall in to 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. In situations where the use of a boat/skiff is not practical, a rescue person should be stationed on the river bank with a life ring.

2.2.4 Causes of Accidents

General Causes

Accidents are usually caused by human error or equipment failure. Part of safety awareness is acknowledging this and planning ahead to minimize the effects of those errors or failures.

Accidents caused by equipment failure can often be traced to inadequate or improper maintenance. Inspection, maintenance, and update of equipment can minimize failures. Accidents caused by people are usually caused by an error in judgment, thoughtlessness, or trying to take shortcuts.

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, frayed cables or access equipment not inspected regularly.
- Inappropriate or loose fitting clothing.

2.2.5 Safety Precautions

Safety precautions can be divided in to several categories: General Precautions, Climbing Safety, Confined Spaces, Vegetation, Night Work, Working Around Water, and Culverts.

General

Mental Attitude

The inspector has to be mentally prepared to do a climbing inspection. A good safety attitude is of foremost importance. Address the following three precautions:

- Avoid emotional distress – Do not climb when emotionally upset. The inspector who climbs needs to have complete control; otherwise the chances of falling increase.
- Awareness of surroundings – Always be aware of dangers associated with inspection location when climbing. Do not become as engrossed in the job as to step into mid-air.
- Realize limitations – An inspector is to be confident the job can be performed safely. If there is a feature that cannot safely be inspected with the equipment available, do not inspect it. Highlight this fact in the notes so that appropriate equipment can be scheduled if necessary. Do not hide the fact that a particular bridge member was not inspected.

General Guidelines

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 is to 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 methods – Establish methods early in the job and utilize 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 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.
- Do not use of intoxicants or drugs – Intoxicants impair judgment, 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. Assume cables and wires to be hot (live) even if they appear to be only telephone cables. The conditions encountered on many bridges are conducive to electric shock. These conditions include steel members, humidity, perspiration, and damp clothing. Identify transmission lines on a structure prior to the inspection. Shut down power lines. In rural areas, avoid electric fences since they can be a hazard. Be aware that fiberglass posts eliminate the need for the distinctive porcelain insulation, which once identified electric fences.
- Inspection over water – A safety boat is to be provided when working over water. Be sure the boat is equipped with a life ring and radio communication with the inspection crew.
- Waders – Use caution 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, tie off equipment, such as hand tools and clip boards.
- Entering dark areas – Use a flashlight to illuminate dark areas prior to entering as a precaution against falls, snakebites, and stinging insects.
- Vagrant people – Exercise caution when approaching a bridge where homeless people are present. Explain to them an inspection of the bridge is taking place, and the inspection team leaves the site as soon as possible. Leave the bridge site immediately if there are any illegal activities or perceived danger.

Working in Teams

Work in pairs. Do not take any action without someone else there to help in case of an accident. Make sure someone else knows where you are. If someone seems to be missing, locate that person immediately.

First Aid Training is recommended for bridge inspectors and is available through organizations such as OSHA or the American Red Cross.

If an inspector is injured during an inspection, it is important to know First Aid and/or cardiopulmonary resuscitation (CPR). The American Red Cross offers training for First Aid, CPR and AED (automatic external defibrillator). Local fire departments and the American Heart Association (AHA) can also provide training for CPR.

Climbing Safety

There are two primary areas of preparation necessary for a safe climbing inspection:

- Organization
- Inspection Equipment

Figure 2.2.7 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 needs to have an inspection plan so the inspection team knows where to go, what to do, and what tools are needed to perform the inspection. An organized inspection reduces the chance of the inspectors falling or getting stuck in a position in which they are 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, be sure that your boots are free of mud, and use extreme caution in areas where debris accumulation may cause a slippery surface.

Inspection Equipment

The inspection team needs to be well equipped to properly complete their inspection.

Check personal attire for suitability to the job:

- Clothing – proper for climbing activities and temperature.

- Jewelry – Avoid wearing rings, bracelets, and necklaces. In an accident, jewelry can become snagged and cause additional injury.
- Eyeglasses – wear only single lens glasses; do not wear bifocals because split vision impairs ability to climb safely.

Check inspection equipment for proper use and condition.

Ladders

Accidents involving ladders are the most common type of inspection-related accident. Refer to and follow OSHA for rules applicable to stairways and ladders.

In order to use a ladder properly, consider the following:

- Proper ladder length for the job.
- 4 to 1 tilt with blocked and secured bottom (see Figure 2.2.8).
- An assistant for ladders over 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 the inspector's center of gravity or belt buckle over the rungs.
- Using a hand line to lift equipment or tools.

Figure 2.2.8 Proper Use of Ladder

Scaffolding

Refer to and follow OSHA for rules applicable to scaffolding. Check scaffolding 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. Perform a daily inspection for cracks, loose connections, and buckled or weak areas prior to use.

Timber Planks

Never use single planks. Use two or more planks securely cleated together. Securely attach plank ends to their supports. Inspect planks 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 elements during an inspection (see Figure 2.2.9). Confirm that they are in safe operating condition. Only use such equipment when placed on a firm surface at a slope not exceeding the manufacturer’s recommendations. Use extreme caution when operating near traffic.

Figure 2.2.9 Bucket Truck

Catwalks and Travelers

Permanent inspection access devices are ideal. However, be on guard for misalignment and deterioration of elements, such as flooring, hand-hold rods, and cables (see Figure 2.2.10).

Figure 2.2.10 Inspection Catwalk

Rigging

Be familiar with proper rigging techniques. The support cables need to be at least one-half inch in diameter. The working platform or "stage" need to be at least 20 inches wide. Use a line or tie-off cable separate from the primary rigging.

Use common sense with regard to rigging. Do not blindly trust the people arranging the rigging. Mistakes by riggers can cause life threatening accidents. If a method 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 2.2.11 Inspection Rigging

Confined Spaces Precautions

Safety Concerns

Inspection of box girder bridges, steel box pier caps, steel arch rings, arch ties, cellular concrete structures, and long culverts is often categorized as confined spaces. Confined space entry is regulated by Occupational Safety and Health Administration (OSHA) and requires proper training, equipment, and permitting.

There are four major concerns when inspecting a confined space:

- Lack of oxygen – an oxygen content above 19% is needed for the inspectors 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 inspecting a confined area, use the safety standards prescribed by OSHA and any additional agency or employer requirements. 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 – Check oxygen and gas levels and verify acceptability for a minimum prescribed time prior to entry.
- During occupancy – Regardless of activity, use continuous ventilation. 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.
- Perform operations that produce toxic gases "down-wind" of the operator and the inspection team.
- Position gasoline powered generators "down-wind" of operations.
- 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.
- Perform the inspection in pairs, with a third inspector remaining outside of dark or confined areas with means to communicate with inspectors.

Vegetation

Be aware of any vegetation located around any substructures. Poison ivy, oak and sumac are examples of vegetation which can cause skin irritations if touched by someone. Also, it is important to be aware of any tall vegetation which could hide holes in the ground and lead to possible injury if not found. Tall vegetation can also hide other tripping hazards.

Night work

When working at night, it is important to be properly dressed. This is necessary so the inspectors can be more visible by passing motorists. It can be accomplished by wearing a safety vest which has both bright colors and reflectivity. The use of proper temporary traffic control also helps motorists be aware that there are workers ahead.

Working Around Water

Wading

When wading in water, it is important to be aware of any scour holes and be careful not slip or fall on objects in the water. If an inspector slips or steps into a scour hole, their waders can fill with water and drag them under, making swimming impossible. It is also important to wear a life vest while wading to help prevent the inspector from being pulled down if the waders were to fill up with water. It is beneficial for the inspector to carry and use probing rod to locate scour holes and soft stream bed material. Be mindful of potentially dangerous aquatic life.

Drowning

Extensive streambed scour may result in channel depressions. 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. Be cautious whenever storms appear imminent.

Underwater

When performing an underwater inspection, particularly in low visibility and/or high current situations, use extreme care and be sure to watch for drift and debris at any height in the water. See Topic 13.3.2, for additional safety concerns.

Culverts

There are several hazards that can be encountered when performing a culvert inspection. Being aware of these situations and exercising proper precautions 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
- Quick Sand Conditions at the Outlet
- Potentially dangerous wildlife

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, perform tests 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.

Observe confined space entry requirements when inspecting a long culvert or any culvert with restricted ventilation.

Quicksand Conditions at the Outlet

Quicksand conditions can occur in sandy streambeds, especially at the outlet end of the culvert. Be aware of these conditions and proceed with caution in geographical areas known to have these problems.

Working Around Traffic

Do not obstruct traffic during bad weather. Avoid the inspection of the top of concrete decks during or just after it rains (see Figure 2.2.12).

Figure 2.2.12 Inclement Weather Causing Slippery Bridge Members and Poor Visibility for Motorists

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Chapter 2
Safety Fundamentals for Bridge Inspectors

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Topic 2.3 Temporary Traffic Control

2.3.1 Introduction

Bridge inspection usually only requires traffic control procedures for a relatively short term closure Long term closures for construction activity which use concrete barriers are not included in this topic.

Figure 2.3.1 Temporary 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 standard manuals or drawings.

When working in an area exposed to traffic, check and follow the existing agency standards. These standards prescribe the minimum procedures for a number of typical applications and the proper use of standard temporary traffic control devices such as cones, signs, and flashing arrow-boards (see Figure 2.3.1). Sometimes after initial installation, temporary traffic control may need revised to provide adequate protection to motorists, pedestrians or inspectors.

2.3.2 Philosophy and Fundamental Principles

Temporary traffic control devices used on street and highway construction or maintenance work need to conform to the applicable standards of the *MUTCD* and the agency.

Minimize inspection time to reduce exposure to potential hazards without compromising the thoroughness of the inspection. Principles and procedures which have been shown to enhance the safety of motorists, pedestrians, and bridge inspectors in the vicinity of work areas include the following:

Inform the Motorists

Traffic safety in work zones is an integral and high priority element of every inspection project, from the planning stage to performance of the inspection. Keep in mind the safety of the motorist, pedestrian, and worker.

The basic safety principles governing the design of temporary traffic control for roadways and roadsides, govern the design of inspection sites. The goal is to route traffic through such areas with geometrics and temporary traffic control devices comparable to those for normal highway situations. Clearly communicate to the driver the notice of work site locations and guidance through these sites.

A temporary traffic control plan, in detail appropriate to the complexity of the work project, is prepared and understood by the responsible parties before the site is occupied. The official trained in safe traffic control practices approves any changes in the temporary traffic control plan.

Control The Motorists

Inhibit traffic movement as little as practical. Design temporary traffic control in work sites on the assumption that motorists only reduce their speeds if they clearly perceive a need to do so. Avoid reducing the speed zoning as much as practical.

The objective is a traffic control plan that uses a variety of temporary 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. Avoid frequent and abrupt changes in geometrics, such as lane narrowing, dropped lanes, or main roadway transitions that require rapid maneuvers.

Make provisions 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 the entire floor system from left to right, temporary traffic control may dictate working partial width, a few stringers at a time.

Provide a Clearly Marked Path

A good traffic control plan provides safe and efficient movement of motorists and pedestrians and the protection of bridge inspectors at work areas.

Provide adequate warning, delineation, and channelization to assure the motorist positive guidance in advance of and through the work area. Use proper signing and other devices which are effective under varying conditions of light and weather.

The maintenance of roadside safety requires constant attention during the life of the work because of the potential increase in hazards. Remove temporary traffic control devices immediately when no longer needed.

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.

Accomplish the channelization of traffic by the use of cones, barricades, and other lightweight devices which yield when hit by errant vehicles.

Store equipment and materials in such a manner as not to be vulnerable to run-off-the-road vehicle impact, whenever practical. Also, provide adequate attenuation devices when safe storage is not available.

2.3.3 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 (see Figure 2.3.2). To minimize traffic disruption, the work zone needs to be as compact as possible, but 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 is to be clearly signed as a courtesy to the motorist.

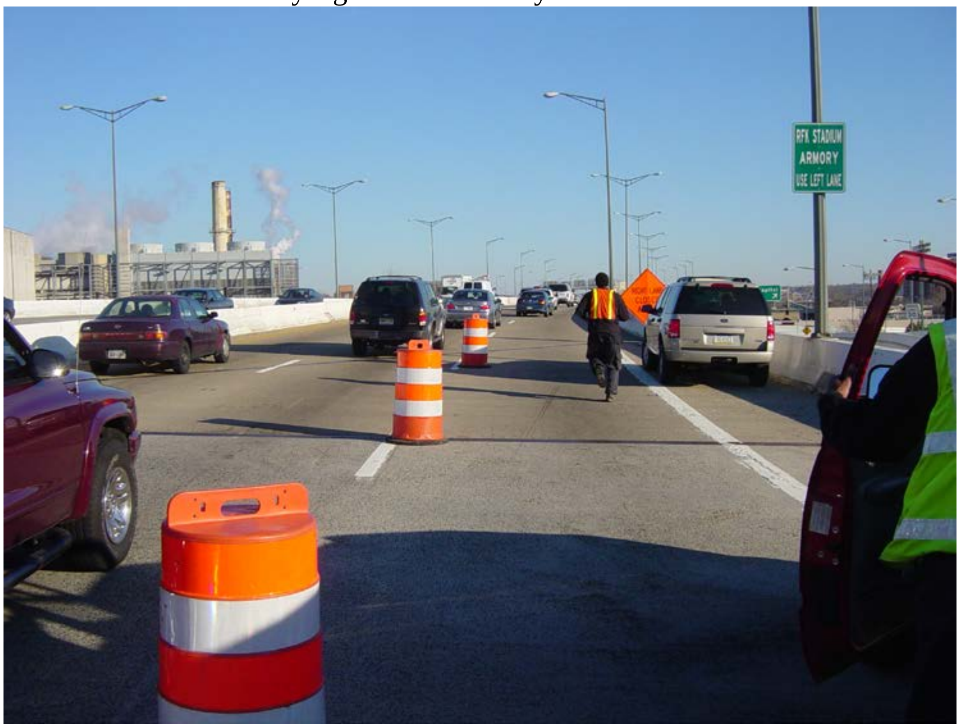

Figure 2.3.2 Work Zone

Vehicles and Equipment

Inspection vehicles and equipment need to be made visible to the motorists with flashing marker lights or arrow boards as appropriate (see Figure 2.3.3).

Use roof mounted flashing lights or flashers on vehicles entering and exiting the work zone to distinguish them from other motorists' vehicles. Also, vehicles are to use extreme caution when moving in and out of the work zone. Allow motorists ample time to react to the vehicle's movements.

Figure 2.3.3 Inspection Vehicles with Flashing Light

Workers

Individuals in a work zone are to wear approved safety vests and hard hats for visibility and identification. They also help make the inspector look "official" to the public. Also, it is important for the inspector to stay within the work zone for their own safety.

2.3.4 Principles of Temporary Traffic Control Devices

Each bridge inspection project is different and has traffic concerns that are unique to that location. Selection of the proper temporary traffic control devices for each location is dependent upon many factors. Though there are several different types of temporary traffic control devices, there are some basic principles for efficient temporary traffic control devices:

1. Temporary traffic control devices are to be visible and attention getting. Devices in good condition are preferred.

   - Bright colors make devices easier for motorists to see. Standard colors are orange and white (*MUTCD*).
   - Signs that are 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, or illumination.
   - Properly sized for the roadway so they can be seen by the motorists.

2. Temporary traffic control devices are to give clear direction.

3. Temporary traffic control devices are to command respect and be official (*MUTCD*). These devices need to look professional and be geared to the class of highway, speeds and traffic involved. Haphazard traffic control gives the public a bad perception of the rest of the project as well.

   State agencies have been mandated to adopt the Federal *MUTCD*. When working in an area exposed to traffic, check and follow the agency standards. These standards prescribe the minimum methods for a number of typical applications and the proper use of standard traffic control devices.

4. Temporary traffic control devices are to 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, the 2.5 seconds translates to 220 feet. Additional time and distance is required for a specific action taken such as “hitting the brakes”.
   - Temporary traffic control accommodates a wide range of vehicles (from small compact cars to large combination tractor-trailers) and driver skills, which may be impaired by alcohol, drugs, drowsiness, or use of cell phones.

   Advance warning is essential to get the right response from drivers. The *MUTCD* provides guidance on the positioning of advance warning signs for specific traffic control applications.

These basic principles for temporary traffic control devices have been factored into the various agencies' procedures for work area traffic control. These procedures represent efforts by trained people. Do not change traffic patterns without consulting the *MUTCD*, agency standards or traffic control personnel.

2.3.5 Types of Temporary Traffic Control Devices

Signs

Types of temporary traffic control signs include the following:

- Regulatory – Inform motorists of traffic laws or regulations and indicate the applicability of legal requirements that are not apparent. These signs are authorized by the public agency or official having jurisdiction. Examples include "Speed Limit", "DO NOT PASS", which may require special authority (see Figure 2.3.4).
- Warning – Notify road users of specific situations or conditions on or adjacent to a roadway that might not be apparent. They may be used by themselves or in combination with other advance warning signs. Examples include "Bridge Inspection", "Work Area Ahead", and "Slow" messages (see Figure 2.3.5).
- Guide Signs - Directional and destination signs that provide motorists with information to help them through a temporary traffic control zone. They are not used for bridge inspection traffic control unless a detour is established (see Figure 2.3.6).
- Arrow boards – Used to advise approaching motorists of a lane closure along major multi-lane roadways in situations involving heavy traffic volumes, and/or limited sight distances, or at other location and under other conditions where road users are less likely to expect such lane closures. Use them in combination with the appropriate signing, channelization devices and other temporary traffic control devices (see Figure 2.3.7)
- Changeable message signs – Provide motorists with the notice of unexpected situations. They may present the motorist with complex messages, important information, and real time conditions.(see Figure 2.3.8)

Figure 2.3.4 Regulatory Sign

Figure 2.3.5 Warning Sign

Figure 2.3.6 Examples of Guide Signs

Figure 2.3.7 Arrow Board

Figure 2.3.8 Changeable Message Sign

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 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. They need to 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 are 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 – Used to channelize motorists, divide opposing traffic lanes, divide lanes when two or more lanes are kept open in the same direction, and delineate short duration maintenance and utility work including bridge inspections. Predominately orange and made of material that can be struck without causing damage to the impacting vehicle and are primarily used during the day. Consult the appropriate governing agency and *MUTCD* to determine the specific requirements for cones, such as size and features, which depend on the application. (see Figure 2.3.9) Drums – Used for road user warning or channelization and are constructed from lightweight, deformable materials. They provide the motorist a highly visible and respectable warning of upcoming conditions. For the bridge inspector, drums are portable enough to be shifted place to place within a work zone to accommodate changing conditions (see Figure 2.3.10).

- Tubular markers – Predominately orange and made of a material that can be struck without causing damage to an impacting vehicle. Consult the appropriate governing agency and *MUTCD* to determine the specific requirements for tubular markers, which depends on the application. These devices are not as common for bridge inspection as cones (see Figure 2.3.11).

- Vertical panels – May be used to channelize vehicular traffic, divide opposing lanes, or replace portable lightweight barricades. The diagonal orange and white stripes pointing downward indicate the direction motorists are to pass (see Figure 2.3.12).

- Temporary traffic barrier – Not considered temporary traffic barriers by themselves. When placed in a position identical to a line of channelizing devices and marked and/or equipped with appropriate channelization features to provide guidance, they serve as traffic control devices. Not only do they serve to direct motorists, but they also protect the workers. These are seldom applicable to bridge inspection due to the short duration of the work (see Figure 2.3.13).

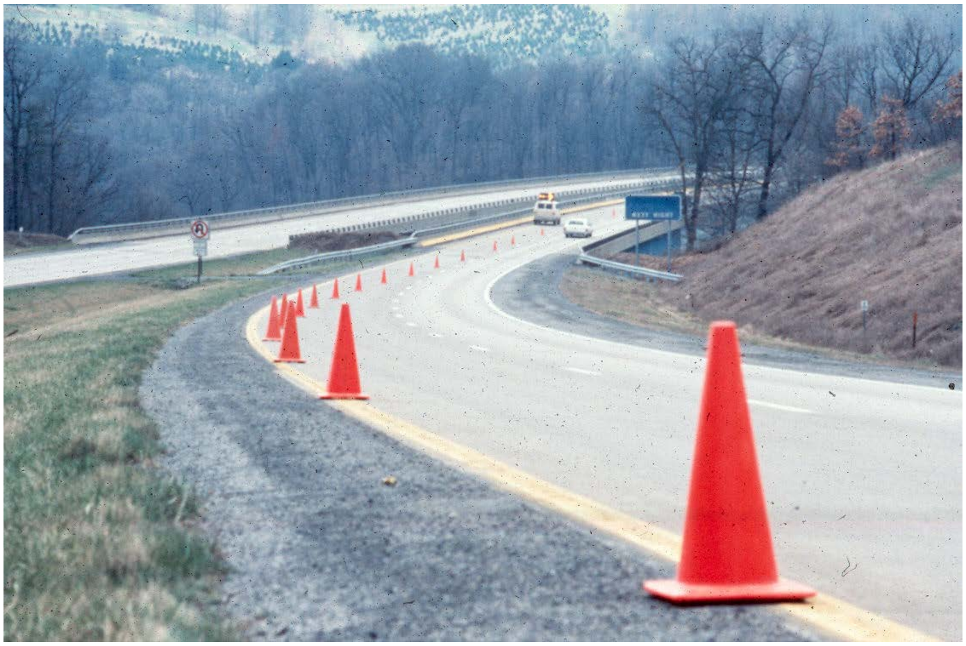

Figure 2.3.9 Cones

Figure 2.3.10 Drums

Figure 2.3.11 Tubular Marker

Figure 2.3.12 Vertical Panel

Figure 2.3.13 Temporary Traffic Barriers

Lighting Devices

Another type of control device is lighting. Lighting devices are used to supplement retroreflectorized signs, barriers, and channelizing devices. Examples of lighting include the following:

- Warning lights **-** Attached to signs or other devices to attract attention or for night visibility. Flashers are commonly placed on maintenance and inspection vehicles, as well as drums, vertical posts, and other channelization devices (see Figure 2.3.14).
- Floodlights – Inspection, utility, maintenance, or construction activities are sometimes conducted during nighttime periods when vehicular traffic volumes are lower. Bridge inspections may be conducted during these hours on high volume roadways to avoid additional congestion from daytime traffic. During these periods, floodlights used to illuminate the work area, equipment crossings, and other areas.

Figure 2.3.14 Warning Lights

Flaggers

A number of hand signalizing devices, such as STOP/SLOW paddles, flashing lights, flashlights, 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. If permitted by the agency, limit flag use to emergency situations and at spot locations that can best be controlled by a single flagger.

Since flaggers are responsible for human safety and make the greatest number of public contacts of any inspection personnel, it is important that qualified personnel be selected. The following are qualifications for a flagger:

- Ability to receive and communicate specific instructions clearly, firmly, and courteously
- Ability to move and maneuver quickly in order to avoid danger from errant vehicles
- Ability to control signaling devices (such as paddles and flags) in order to provide clear and positive guidance to drivers approaching a TTC zone in frequently changing situations
- Ability to understand and apply safe traffic control practices, sometimes in stressful or emergency situations
- Ability to recognize dangerous traffic situations and warn workers in sufficient time to avoid injury

For daytime and nighttime activity, flaggers shall wear high-visibility safety apparel that meets Performance Class 2 or 3 requirements of the ANSI/ISEA 107-2004 publication titled *American National Standard for High-Visibility Apparel and Headwear*. The apparel background (outer) material shall be fluorescent orange-red, fluorescent yellow-green, or a combination of the two as defined in the ANSI standard. The retroreflective material shall be orange, yellow, white, silver, yellow-green, or a fluorescent version of these colors, and shall be visible at a minimum distance of 1,000 feet. The retroreflective safety apparel shall be designed to clearly identify the wearer as a person.

For nighttime activity, high-visibility safety apparel that meets the Performance Class 3 requirements of the ANSI/ISEA 107-2004 publication should be considered for flagger wear.

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 is always 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. In positioning flaggers, consideration is given to maintaining color contrast between the work area background and the flagger's protective garments.

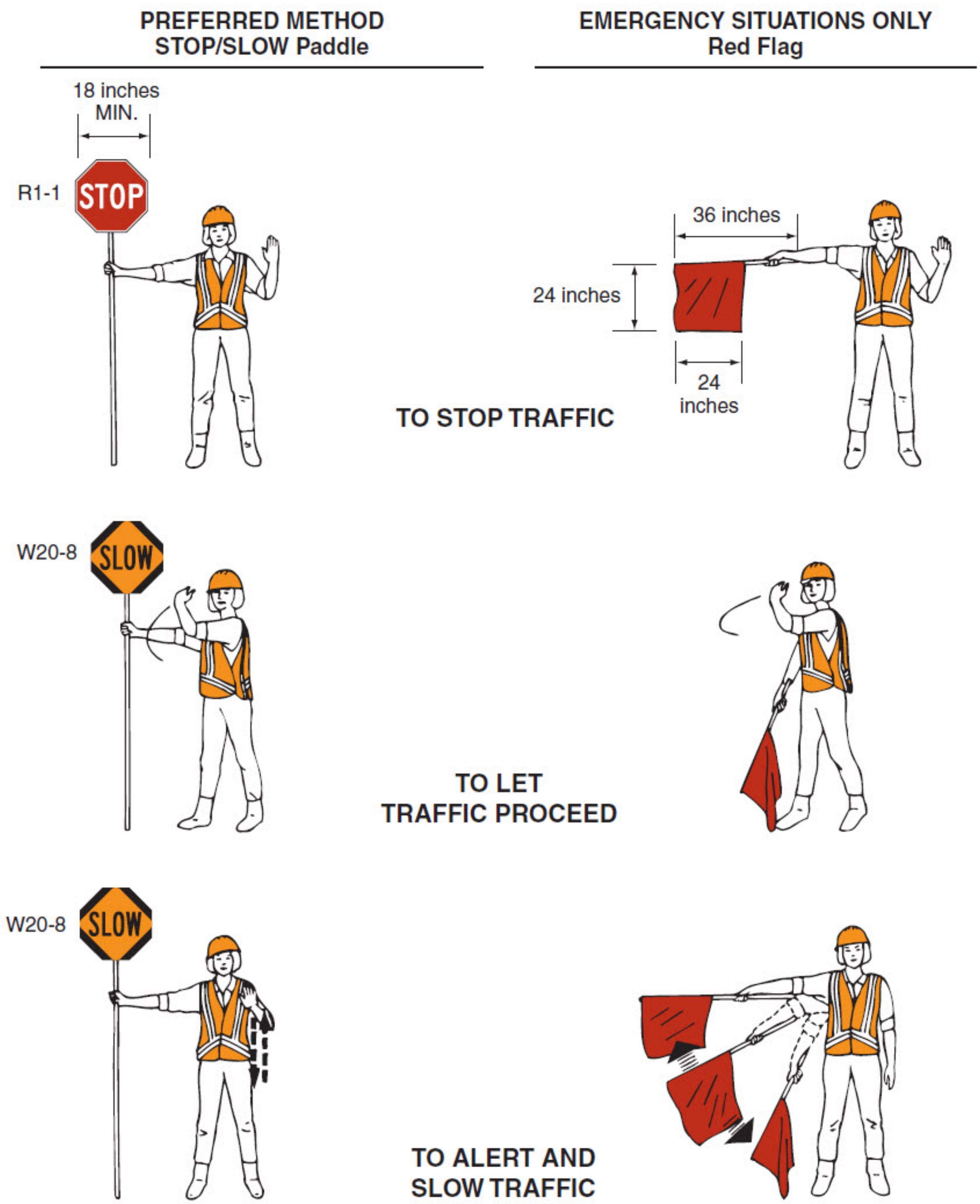

Figure 2.3.15 Use of Hand Signaling Devices by Flagger (from *Manual on Uniform Traffic Control Devices (MUTCD)*)

Use the following methods of signaling with sign paddles (see Figure 2.3.15):

- To stop traffic, the flagger faces the motorists and aims the STOP paddle face toward the traffic in a stationary position with the arm extended horizontally away from the body. The free arm is held with the palm of the hand above shoulder level toward approaching traffic.

- To direct traffic to proceed, the flagger faces the motorists with the SLOW paddle face aimed toward the traffic in a stationary position with the arm extended horizontally away from the body. The flagger motions with the free hand for traffic to proceed.
- To alert or slow traffic, the flagger faces the motorists with the SLOW paddle face aimed toward the motorists in a stationary position with the arm extended horizontally away from the body.

Use the following methods of signaling with a flag (see Figure 2.3.15):

- To stop traffic, the flagger faces the motorists and extends the flag staff horizontally across the traffic lane in a stationary position so that the full area of the flag is visibly hanging below the staff. The free arm may be held with the palm of the hand above shoulder level toward approaching traffic.
- To direct stopped traffic to proceed, the flagger faces the motorists with the flag and arm lowered from the view of the drivers, and motions with the free hand for road users to proceed. Flags are not to be used to signal road users to proceed.
- To alert or slow traffic, the flagger faces the motorists and slowly waves the flag in a sweeping motion of the extended arm from shoulder level to straight down without raising the arm above a horizontal position. The flagger keeps the free hand down.

For flagging traffic at night, lights approved by the appropriate highway authority or reflectorized sign paddles or reflectorized flags are used.

Whenever practicable, the flagger advises the motorist of the reason for the delay and the approximate period that traffic is halted. Flaggers and operators of machinery or trucks are made to understand that every reasonable effort is to be made to allow the driving public the right-of-way and prevent excessive delays.

Locate flagger stations far enough in advance of the work site so that approaching traffic have sufficient distance to reduce speed before entering the project. This distance is related to the approach speed and physical conditions at the site (see Figure 2.3.16). In urban areas, where speeds are low and streets are closely spaced, the distance is decreased.

Speed*	Distance
20 mph	115 feet
25 mph	155 feet
30 mph	200 feet
35 mph	250 feet
40 mph	305 feet
45 mph	360 feet
50 mph	425 feet
55 mph	495 feet
60 mph	570 feet
65 mph	645 feet
70 mph	730 feet
75 mph	820 feet

* Posted speed, off-peak 85th-percentile speed prior to work starting, or the anticipated operating speed

Figure 2.3.16 Stopping Sight Distance as a Function of Speed (from *Manual on Uniform Traffic Control Devices (MUTCD)*)

The flaggers stand either on the shoulder adjacent to the traffic being controlled or in the barricaded lane (see Figure 2.3.17). At a spot obstruction, a position may have to be taken on the shoulder opposite the barricaded section to operate effectively. Under no circumstances is a flagger to stand in the lane being used by moving traffic. The flagger always is clearly visible to approaching traffic. For this reason, the flagger has to stand alone, never permitting a group of workers to congregate around the flagger station. The flagger is stationed sufficiently in advance of the work force to warn them of approaching danger, such as out-of-control vehicles.

Figure 2.3.17 Flagger with Stop/Slow Paddle

Adequately protect flagger stations and precede them by proper advance warning signs. At night, adequately illuminate flagger stations.

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-lane, Two-way Traffic Control

Where traffic in both directions use a single lane for a limited distance, make provisions for alternate one-way movement to pass traffic through the constricted work zone. 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 needs to be some means of coordinating movements at each end so that vehicles are not simultaneously moving in opposite directions in the work zone and so that delays are not excessive at either end. Chose control points at each end of the route so as to permit easy passing of opposing lines of vehicles.

Alternate one-lane, two-way temporary traffic control may be facilitated by the following means:

- Flagger control
- Flag transfer
- Pilot car
- Temporary traffic signals
- Stop or yield control

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. Designate one of the two as the chief flagger to coordinate movement. They are able to communicate with each other verbally or by means of signals. These signals are not such as to be mistaken for flagging signals.

Where the end of a one-lane, two-way section is not visible from the other end, the flaggers may maintain contact by means of radio or cell 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 motorist drifts into the lane closure. Each agency has its own specific requirements, but a shadow vehicle is generally employed any time a shoulder or travel lane is occupied by workers or equipment. Shadow vehicles are equipped with appropriate lights and warning signs which may be used for stationary operations for additional protection of occupants and vehicles within the work zone.

- 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 is generally required.
- Some agencies recommend the use of truck or trailer mounted attenuators (see Figure 2.3.18). This protects the motorist, as well as the inspectors.

Figure 2.3.18 Shadow Vehicle with Attenuator

Police Assistance

On some inspection projects, police assistance may be helpful and even required. The presence of a patrol car aids in slowing and controlling the motorists. At a signalized intersection near a job site, a police officer may be required to ensure traffic flows properly and smoothly.

Specialized Traffic Crews

Some states have specialized traffic crews for high traffic roads. They are used due to their specialized training, allowing for a safer work environment.

2.3.6 Public Safety

Since the fundamental goal of bridge inspection is to enhance public safety, it makes little sense to endanger that same public by inadequate traffic control measures. Temporary 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 by inconveniencing them as little as possible and protecting their safety. This means providing well thought out, clear, and effective traffic control measures.

Also consider pedestrians. If a walkway is to be closed, be sure it is properly signed and barricaded. Indicate an alternate route for the pedestrian, if necessary through or preferably around the work zone.

Training

Each person whose actions affect inspection, maintenance and construction zone safety (from the upper-level management personnel to construction and maintenance field personnel) need 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 supervise the selection, placement, and maintenance of temporary 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, motorists, and pedestrians. If there is an accident, the secondary goal is to be able to defend yourself and your employer. Accidents bring lawsuits. Lawsuits bring inquiries about who is responsible. Temporary traffic control is one thing that is investigated. Anything not done in accordance with published standards, regulations, and directives could bring blame upon whoever violated them. Being blamed for an accident is expensive and damaging.

Table of Contents

Chapter 2

Safety Fundamentals for Bridge Inspectors

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Topic 2.4 Inspection Equipment

2.4.1 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 necessary to have. Also, if the bridge is made of timber, then specific pieces of equipment like timber boring tools and ice picks are needed, whereas they are not 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 beginning 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.

2.4.2 Standard Tools

In order for the inspector to perform an accurate and comprehensive inspection, the proper tools are to be used. Standard tools that an inspector uses at the bridge site can be grouped into seven basic categories:

- Tools for cleaning (see Figure 2.4.1)
- Tools for inspection (see Figure 2.4.2)
- Tools for visual aid (see Figure 2.4.3)
- Tools for measuring (see Figure 2.4.4)
- Tools for documentation
- Tools for access
- Miscellaneous equipment

Figure 2.4.1 Tools for Cleaning

Figure 2.4.2 Tools for Inspection

Figure 2.4.3 Tools for Visual Aid

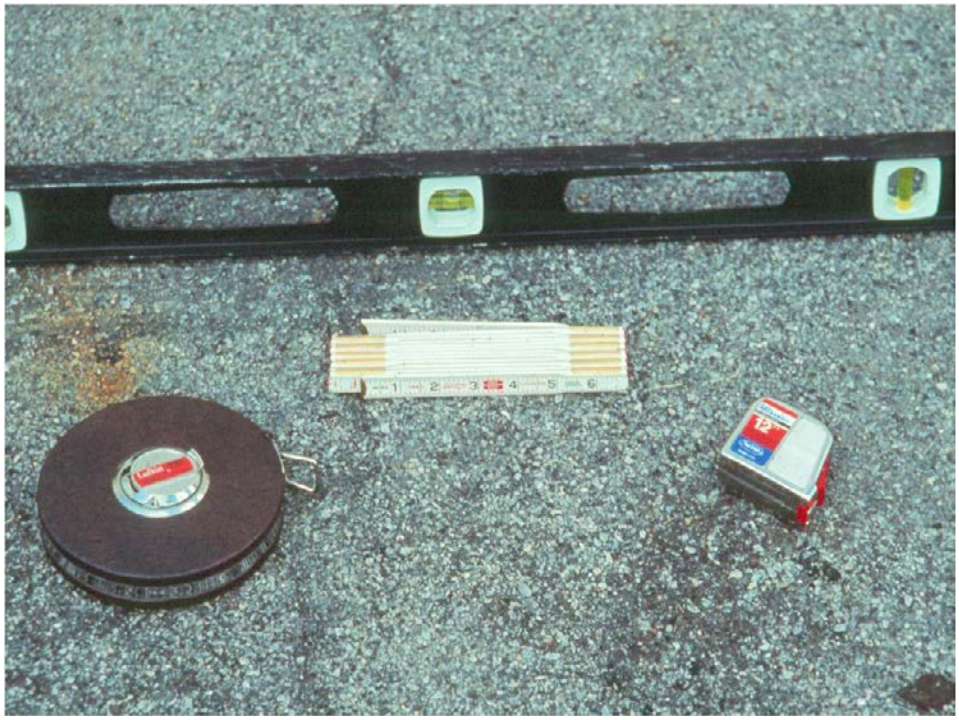

Figure 2.4.4 Tools for Measuring

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 members
- Scrapers - used for removing corrosion or growth from member 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 members
- Hand brace and bits - used for boring suspect areas of timber members
- Timber boring tools - used for internal examination of timber members
- 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 member
- 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 (six foot rule) - used to measure deficiencies and member and joint dimensions
- 25 foot and 100 foot tape - used for measuring component dimensions
- Calipers - used for measuring the thickness of a member 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
- Four foot carpenter's level - used for measuring deck cross-slopes, approach pavement settlement and substructure alignment
- 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
- Note books - used for additional record keeping for complex structures
- Straight edge - used for drawing readable sketches
- Digital camera - used to provide digital images of deficiencies which can be downloaded and e-mailed for instant assessment
- Chalk, kiel, paint sticks, or markers - used for member and defect identification for improved organization and photo documentation
- Center punch - used for applying reference marks to steel members 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 2.5.2.

Miscellaneous Equipment

Miscellaneous equipment includes:

- "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)

2.4.3 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. These inspectors are familiar with the 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.

Non-destructive Evaluation Equipment

Non-destructive evaluation (NDE) is the in-place examination of a material for structural integrity without damaging the material. NDE equipment allows the inspector to "see" inside a bridge member and assess deficiencies that may not be visible with the naked eye. Generally, a trained technician is necessary to conduct NDE and interpret their results. For a more detailed description of NDE, refer to Topics 15.1.2, 15.2.2, and 15.3.2.

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 range pole, piece of reinforcing steel, a survey rod, a folding rule, or even a tree limb.

When the waterway is deep, an underwater inspection is performed by trained divers. This requires special diving equipment that includes a working platform, fathometer, air supply systems, radio communication, and sounding equipment. Refer to Topic 13.3 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

2.4.4 Recent Developments in Equipment

In addition to the standard and special equipment listed previously, there are new equipment and technology available to aid in bridge inspection. The developments in various types of advanced testing methods are described in Topics 15.1, 15.2 and 15.3. The following information represents some of the advances in inspection tools and data collection.

Rotary Percussion

Rotary percussion is a method whereby a uniform tapping is produced by rolling a gear-toothed wheel on a concrete member to detect the presence of concrete deficiencies. This allows for the inspection of overhead and vertical surfaces to be done quickly, and is similar to using a chain drag for the inspection of horizontal surfaces. Advantages of rotary percussion testing tools include the ability to detect near-surface delaminations, quickness of testing, low equipment cost, relatively low level of user's skill required, and low sensitivity to the surroundings.

Figure 2.4.5 Rotary Percussion

Scour Measurement

There is a specialized device used to measure the depth of scour during flood flows. It consists of a depth finder mounted on a water ski. The use of a water ski allows for depth readings to be taken in extremely fast flowing water and also allows for excellent maneuverability of the depth finder into locations under a bridge.

Scour Monitoring

Side Scan Sonar

Side scan sonar is a specialized application of basic sonar theory. Although common for oceanographic and hydrographic survey work, side scan sonar has not been widely utilized for portable scour monitoring. Side scan sonar transmits a specially shaped acoustic beam to either side of the support craft, which allows for one of the most accurate systems for imaging large areas of channel bottom. A disadvantage to this method is that most side scan systems do not provide depth information.

Multi-beam Sonar

Multi-beam systems provide similar fan-shaped coverage to side scan systems, but output depths instead of images. Multi-beam sonar is typically attached to the surface vessel rather than being towed.

Scanning Sonar

Scanning sonar operates by rotating the transducer assembly, emitting a beam while the assembly (or "head") moves in an arc. Scanning sonar is performed by moving the transducer assembly, which allows it to be used from a fixed, stationary position.

Web-based Scour Monitoring

Scour monitoring software allows transportation engineers to predict, identify, prepare for, and record potentially destructive flooding events through a secure internet connection. This type of system identifies the occurrence of a flood event and collects and processes relevant bridge information, several sources of real-time hydrological data and any bridge scour monitoring device data. Transportation officials are able to efficiently dispatch emergency personnel, bridge safety inspectors, and maintenance workers before, during, and after a flood event affects a state's bridge inventory.

Portable Depth Sounders with Transducers

Portable depth sounders with transducers have been used to monitor real time scour at substructure units during major flood events. The deck elevations and scour depths of concern are indicated on the Scour Action Plan. If the scour reaches the critical depth specified, the bridge is closed.

Scour Monitoring Collar

The Magnetic Sliding Collar (MSC) is a scour monitoring device. The magnetic sliding collar device consists of a stainless steel pipe driven into the channel bottom with a sliding collar that drops down the pipe as the scour progresses. The location of the collar is detected by the magnetic field created by magnets on the collar. Installations conducted in cooperation with state highway agencies demonstrated that this simple, low-cost instrument is adaptable to various field situations, and can be installed with the equipment and technical skills normally available at the district level of a state highway agency.

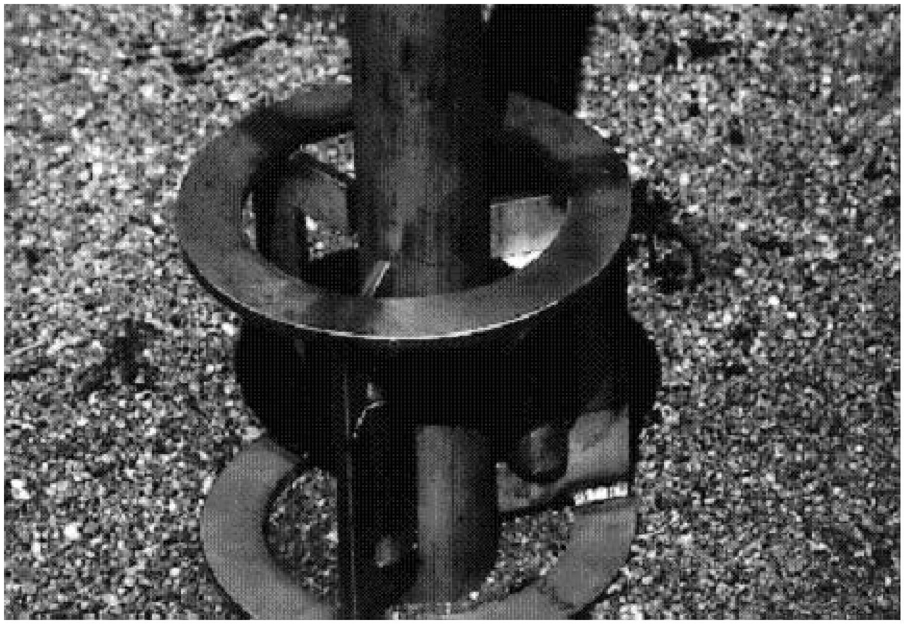

Figure 2.4.6 Scour Monitoring Collar

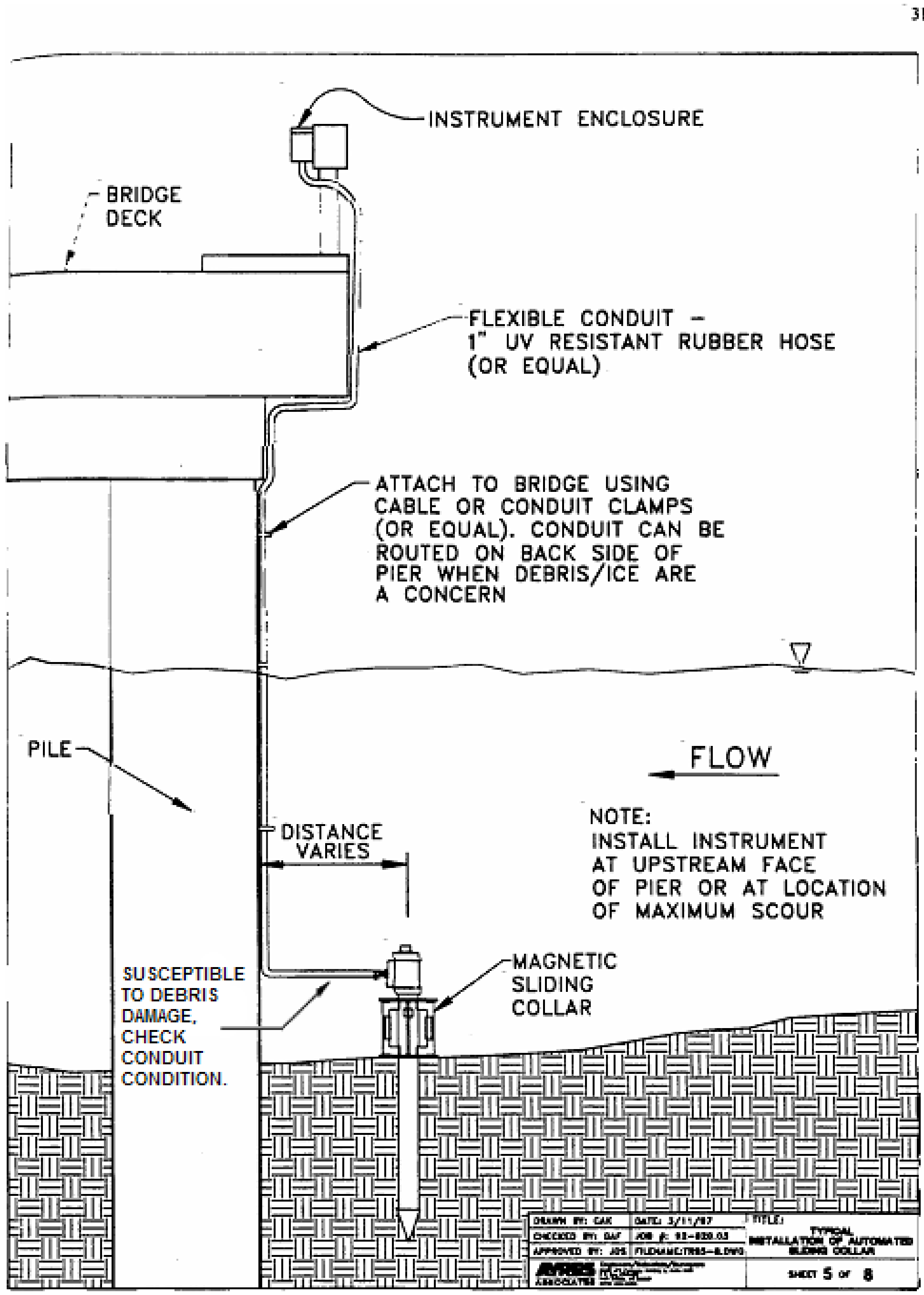

Figure 2.4.7 Scour Monitoring Collar Schematic

Remote Camera

The basic components of a computer-based image system include: an imaging sensor, most commonly a solid-state camera; the image acquisition boards, which convert optical images into an array of digital information, representing the brightness values of the surface; and dedicated processor. The computer-based imaging system can provide two main types of information: spatial measurements and surface analysis. Spatial information encompasses two-dimensional or three-dimensional analysis, measurements and recognition. The surface analysis provides information regarding the color or gray-scale attributes of the target. For example, imaging systems are able to distinguish a flaw from the rest of the surface, and determine size, shape, location and even smallest color attributes of the deficiency. The field of view can be processed in a fraction of a second and can be on the order of 200 to 500 times the size of the smallest feature of interest.

This system works well in a clean environment, however if the item/member is very dirty or has debris surrounding it, cleaning may need to be performed prior to using a remote camera.

Figure 2.4.8 Remote Camera

High Speed Underclearance Measurement System

The high speed underclearance measurement system can mount on any vehicle with a trailer hitch receiver. The system measures the underclearance of a bridge at normal highway speeds. Along with the underclearance data, the GPS location is gathered. Software is used for the data acquisition, display and analysis. The bridge beam height is read to the nearest tenth of a foot. The GPS information can be pasted into a map program to obtain the structure location for future reference.

Figure 2.4.9 High Speed Underclearance Measurement System

Robots

Robotic devices for many applications are being developed by university researchers. High level and underwater bridge inspection are among these applicatons.

One example is the serpentine robot being developed that possesses multiple joints that give it a superior ability to flex, reach, and approach every point on the bridge. This robot is under development.

Other developments in robotic devices are presented in more detail in Topic 15.3.

Laser Scanning

Laser scanning technology can create accurate and complete 3D as-built models quickly and safely. These digital models are automatically combined with CAD design models to allow generation of “as-built” drawings for existing structures. This method can replace tedious field measurements for rehabilitation projects.

Data Recording

The majority of bridge inspectors use pencil and paper to record deficiencies on a bridge. They usually take a copy of the last inspection notes or report and "mark-up" changes since the last inspection. The inspectors input the current findings into the bridge owner's software and the inspection is updated.

Many State agencies are using Electronic Data Collection for bridge inspection.

Hardware

Data recording hardware can include regular office computers, notebook computers or tablet PCs (see Figure 2.4.10) Some versions of these devices have been made to be more rugged and even “wearable” for use in the field.

Software

Specialized software packages can provide a comprehensive set of solutions to manage, inspect, maintain and repair bridges. They allow the user to maintain a comprehensive asset inventory database, collect inspection data from electronic devices, keep history of inspection and maintenance records, assign inspection and maintenance requirements to each structural component, automatically generate inspection reports, and offer decision support.

AASHTO Pontis suports databases on bridge inspection and management. Many bridge owners use AASHTO Pontis based software and have developed programs to address their specific needs.

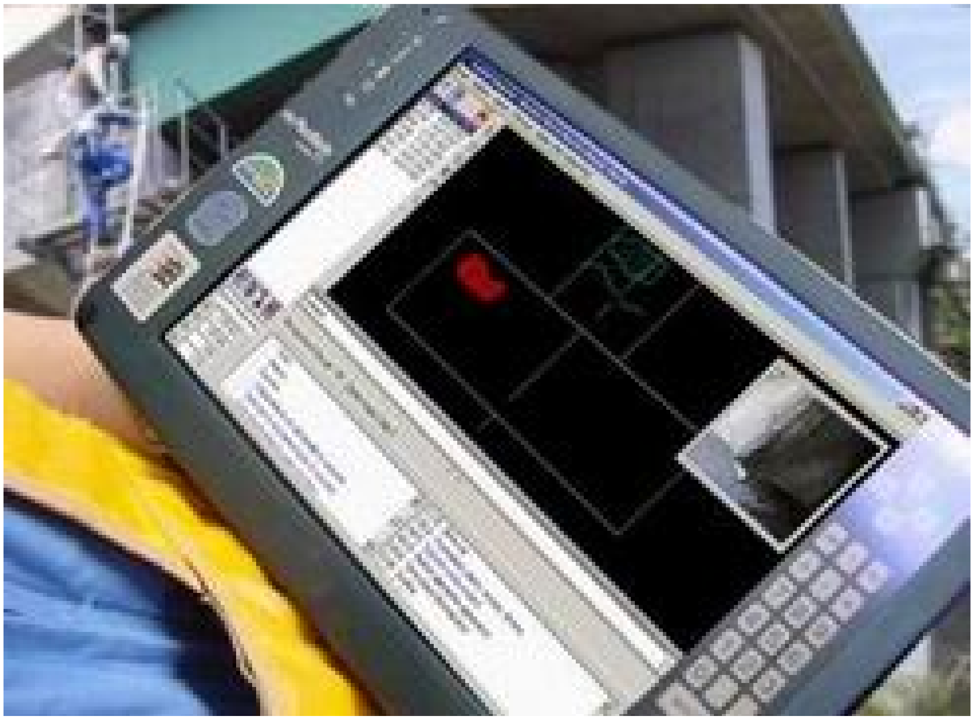

Figure 2.4.10 Tablet PC Used to Collect Inspection Data

2.4.5 Primary Safety Concerns

Proper inspection equipment plays a key role in maintaining the safety of the traveling public and the inspectors. Inspectors who do not have the right equipment, may attempt to use an alternate piece of equipment that is not really designed for the job. Using whatever equipment is at hand, in an attempt to save time and money, can prove dangerous for the inspection team as well as the public. The best way to avoid these circumstances is to ensure the inspectors have the proper equipment for the job and that the equipment is serviced or replaced periodically. This responsibility lies not only with the inspector or team leader but also their employer. It is important that the employer make every effort to properly equip their inspection teams. Also, the inspector needs to be familiar with every piece of equipment and how to use and operate it properly and safely.

Safety fundamentals for bridge inspectors is presented in detail in Topic 2.2.

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Chapter 2
Safety Fundamentals
for Bridge Inspectors

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Topic 2.5 Methods of Access

2.5.1 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 under bridge inspection vehicles. 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, is normally offset by the higher costs associated with operating access vehicles.

2.5.2 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 used in bridge inspection.

Ladders

Ladders are used for inspecting the underside of a bridge or for inspecting substructure units. However, a ladder is used only for those portions of the bridge that can be reached safely, without undue leaning or reaching. The proper length of the ladder is determined by using it at a four vertical to one horizontal angle. When set up at the proper angle (1 horizontal to 4 vertical), the inspector is able to reach out horizontally, grasp the rung while keeping his or her feet at the base of the ladder (see Figure 2.5.1).

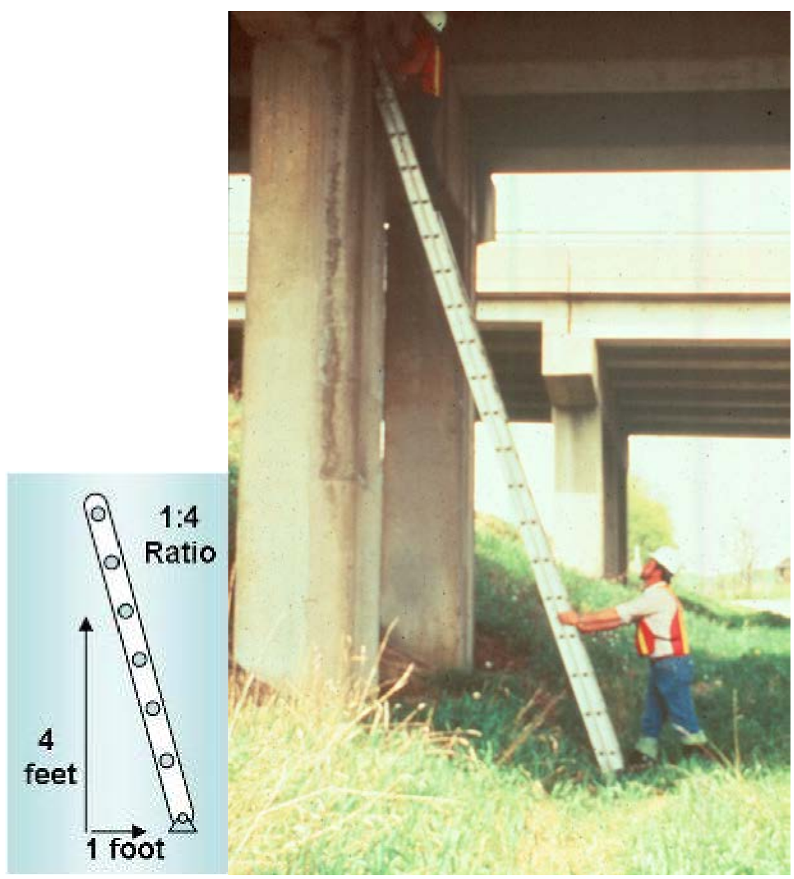

Figure 2.5.1 Inspectors using a Ladder with the Proper 1H to 4V Ratio

Ladders are also used to climb down to access members of the bridge. The hook-ladder, as it is commonly referred to, is fastened securely to the bridge framing (see Figure 2.5.2).

Figure 2.5.2 Inspector using a Hook-ladder

When using a hook-ladder, the inspector is 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 main load-carrying members in areas where access by other means is not feasible or where special inspection procedures are required (e.g., NDE of pins). Rigging is often used when ladders or other access equipment cannot reach a given location (see Figures 2.5.3 and 2.5.4). Rigging is a good choice for a load-posted bridge that does not have the capacity to support an inspection vehicle.

Rigging does not interfere with traffic on the bridge and can be used in high traffic situations where lane closures are intolerable, and on toll facilities to avoid loss of revenue. Rigging may not be an option if there is not enough clearance to avoid interfering with passing features below the bridge.

Figure 2.5.3 Rigging for Substructure Inspection

Figure 2.5.4 Rigging for Superstructure Inspection

Scaffolds

Scaffolds provide an efficient access alternative for structures that are less than 40 feet high and over level ground with little or no traffic nearby (see Figure 2.5.5). The Occupational Safety and Health Administration (OSHA) has specific requirements when working on scaffolding. Scaffolds may take longer to set up than it takes to inspect the bridge. For this reason, scaffolding is not normally used for bridge inspections.

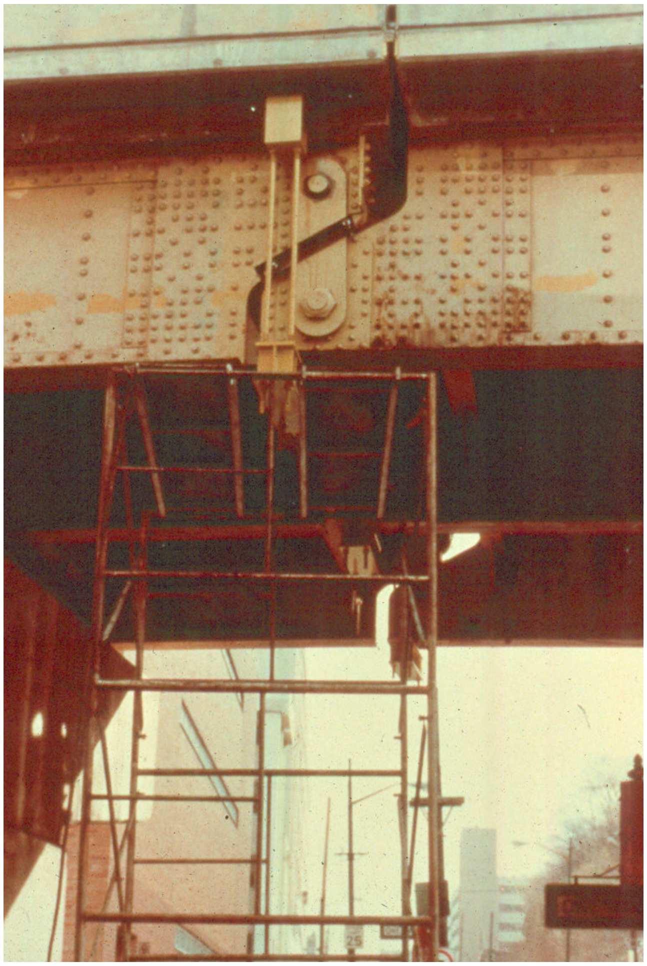

Figure 2.5.5 Scaffold

Boats or Barges

A boat or barge may be needed to gain access to structures over water. A boat can be used for inspection, as well as providing access to areas for taking photographs. Also, a safety boat is required when performing an inspection over water (see Figure 2.5.6).

Figure 2.5.6 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 mobilization for underwater inspections.

Climbers

Climbers are mobile inspection platforms or cages that "climb" steel cables or truss members (see Figure 2.5.7). They are well suited for the inspection of high piers and other long vertical faces of bridge members.

Figure 2.5.7 Climber

Floats

A float is a wood plank work platform hung by ropes (see Figure 2.5.8). Floats are generally used for access in situations where the inspector will be at a particular location for a relatively long period of time.

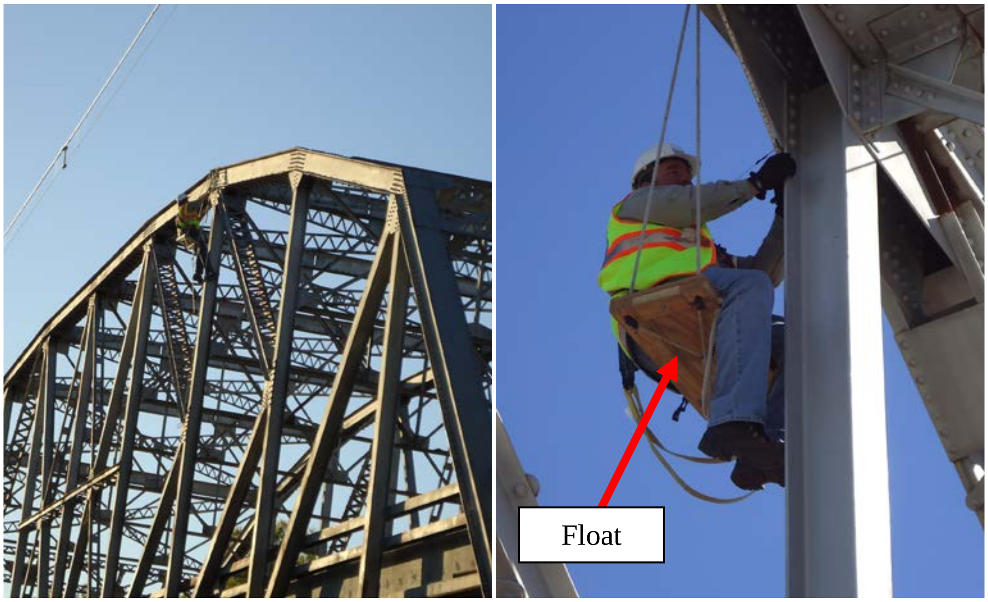

Figure 2.5.8 Inspector Using Float

Bosun (or Boatswain) Chairs / Rappelling

Bosun (or boatswain) chairs are suspended with a rope and can carry 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 (see Figure 2.5.9). However, both methods require the use of independent safety lines.

Figure 2.5.9 Inspector Rappelling Substructure Unit

Free Climbing

On structures, where other methods of access are not practical, inspectors climb on the bridge members to gain access (see Figure 2.5.10). Safety awareness is of the utmost importance when utilizing this technique. When using this method, the inspector is tied off to the bridge using an independent safety harness and lanyard.

Figure 2.5.10 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 under the superstructure (see Figure 2.5.11). Catwalks can be used to inspect parts of the deck, superstructure and some portions of the substructure. The range of inspection area is limited to those locations near the catwalk.

Figure 2.5.11 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 (see Figure 2.5.12). Having the platform perpendicular to the girders allows the inspectors a wider range of movement and enables them to see more of the superstructure elements.

Figure 2.5.12 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 girder web are just a few locations where handrails may be built (see Figures 2.5.13 and 2.5.14). 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 2.5.13 Handrail on Girder Bridge

Figure 2.5.14 Handrail on Suspension Bridge

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.

2.5.3 Types of Access Vehicles

There are many types of vehicles available to assist the inspector in gaining access for "hands-on" inspection of bridge members. The following are some of the most common types of access vehicles used in bridge inspection.

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 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 40 feet to over 170 feet (see Figure 2.5.15).

Figure 2.5.15 Manlift

Scissors Lift

Scissor lifts may be used for bridge inspections with low clearance between the bridge and underpassing roadway. Scissor lifts have a typical maximum vertical reach of 20 feet. These lifts are designed for use on relatively level ground (see Figure 2.5.16).

Figure 2.5.16 Scissor Lift

Bucket Truck

A bucket truck is similar to a manlift. However, a bucket truck can be driven on a highway, and the inspector controls bucket movement (see Figure 2.5.17). As with the manlift, a bucket truck needs to be used on fairly level terrain. Bucket trucks have a number of different features and variations:

- Lift capability - varies 25 to 50 feet.
- Rotating turret - turning range (i.e., the rotational capability of the turret) varies with each vehicle.
- Telescoping boom - some booms may be capable of extending and retracting, providing a greater flexibility to reach an area from a given truck location.
- Multiple booms - some bucket trucks have more than one boom, and provide reach up to 50 feet.
- Outriggers - bucket trucks that offer extended reach and turning range have outriggers or supports that are lowered from the chassis of the vehicle to help maintain stability.
- 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.

Figure 2.5.17 Bucket Truck

A track-mounted man-lift provides access to areas with rough terrain that a conventional bucket truck would not be able to navigate (see Figures 2.5.18 and 2.5.19). By utilizing rubber tracks, track-mounted man-lifts can be operated in water, climb 35 degree slopes, traverse 25 degree side slopes, and navigate wet and muddy terrain.

Figure 2.5.18 Track-mounted Man-lift in a Stream

Figure 2.5.19 Track-mounted Man-lift on a Slope

Under Bridge Inspection Vehicle

An under bridge inspection vehicle is a specialized bucket truck with an articulated boom designed to reach under the superstructure while parked on the bridge deck. Usually the third boom has the capacity for extending and retracting, allowing for greater reach under a structure. Some of the larger under bridge inspection vehicles have four booms, allowing an even greater reach (see Figure 2.5.20).

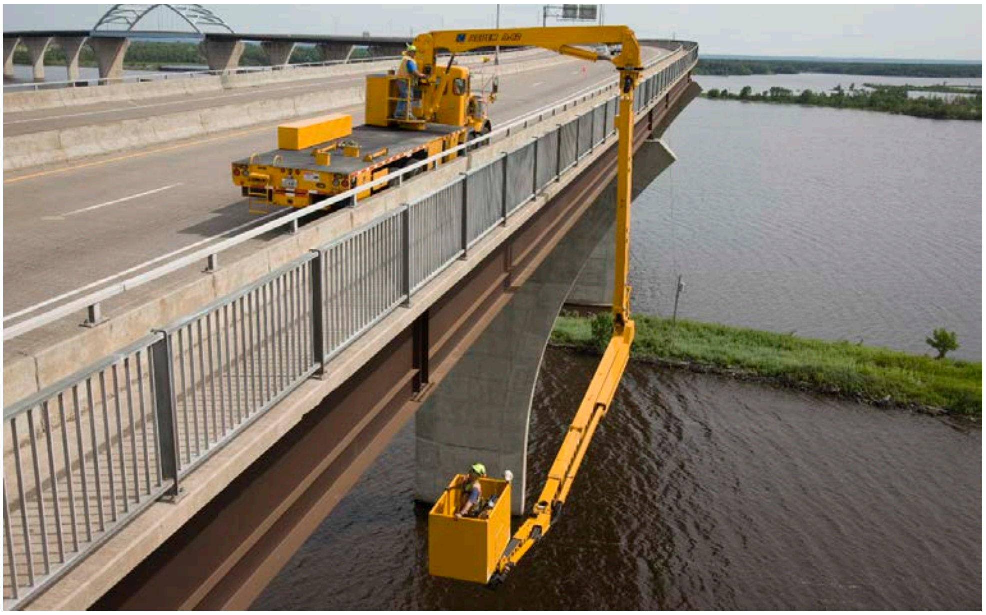

Figure 2.5.20 Under Bridge Inspection Vehicle with Bucket

Variations and options available on different models include:

- Capacity - Some under bridge 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.
- Platform – The platform is lowered by an articulated boom and can then telescope out to provide inspection access to a wide range of the superstructure and substructure. The inspector is now free to walk from beam to beam without having to reposition the platform (see Figure 2.5.21). This combination allows for an efficient and thorough inspection.
- Telescoping second boom - Some under bridge 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 under bridge 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.

Figure 2.5.21 Under Bridge Inspection Vehicle with Platform

2.5.4 Method of Access and Cost Efficiency

In most cases, even the most sluggish lift device will be quicker than using a ladder, rigging or free climbing to inspect a structure. The time saved, however, needs to offset the higher costs associated with obtaining and operating an access vehicle.

In assessing the time-saving effectiveness of an access vehicle, the following questions need to be answered:

- Can the bridge be safely inspected by other reasonable methods?
- What types of access vehicle or access equipment are available?
- How much of the bridge can be inspected using the access vehicle?
- How much of the bridge can be inspected from one setup of the access vehicle?
- 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 independent operator or driver other than the inspector?
- Will the use of the access vehicle require special traffic control?
- Can the bridge carry the weight of an inspection vehicle?
- What are the associated costs of using a bridge inspection access vehicle?

The inspection time, safety and vehicle costs can then be compared to standard access equipment.

2.5.5 Safety Considerations

Safety is the 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 is performed. As a minimum, inspect access equipment as per the manufacture's guidelines. Using faulty equipment can lead to serious accidents and even death. Check the equipment and verify that it is in good working condition with no defects or problems. If rigging or scaffolding is being used, check to ensure that it was installed properly and that the cables and planks are secured tightly. Use OSHA-approved safety harnesses with shock absorbing lanyards when using access equipment.

Access Vehicles

If the inspector is not familiar with the inspection vehicle being used, then take the time required to become accustomed to its 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 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. Always be alert to your location. Do not extend the boom out into unsafe areas such as unprotected traffic lanes or near electrical lines. Use OSHA-approved safety harnesses with shock absorbing lanyards when using access vehicles.

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Chapter 3
Basic Bridge Terminology

Chapter 3
Basic Bridge Terminology

Topic 3.1 Basic Bridge Terminology

3.1.1 Introduction

It is important to 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.

3.1.2 NBIS Structure Length

According to the *FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges*, the structure length is measured in accordance with Item 49 as shown on Figure 3.1.1. The structure length is the length of the roadway that is supported by the bridge structure. To determine the length, measure back to back of back-walls of abutments or from paving notch to paving notch. If the location of the backs of backwalls cannot be exactly determined, inspectors can then measure the distance between the paving notches to determine structure length.

To measure the length of culverts, measure along the center line of the roadway regardless of their depth below grade. The measurements will be between the inside faces of the exterior walls. Tunnels should be measured along the center of the roadway.

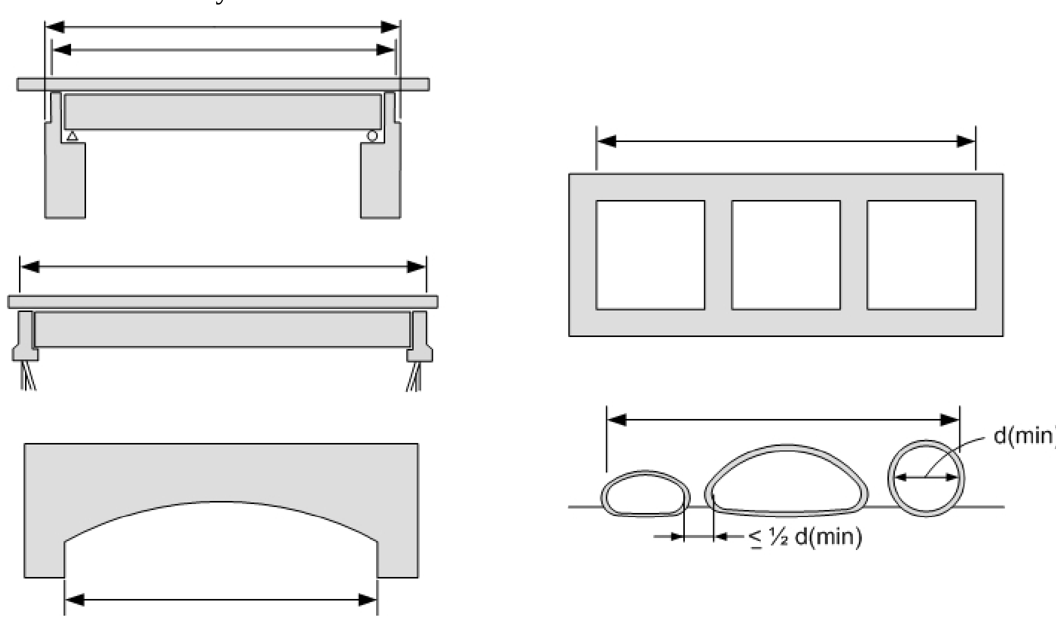


Figure 3.1.1 NBIS Structure Length

3.1.3 NBIS Bridge Length

The *FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* also states, in accordance with Item 112 – NBIS Bridge Length, that the minimum length for a structure to be considered a bridge for National Bridge Inspection Standards purposes, is to be 20 feet (see Figure 3.1.2).

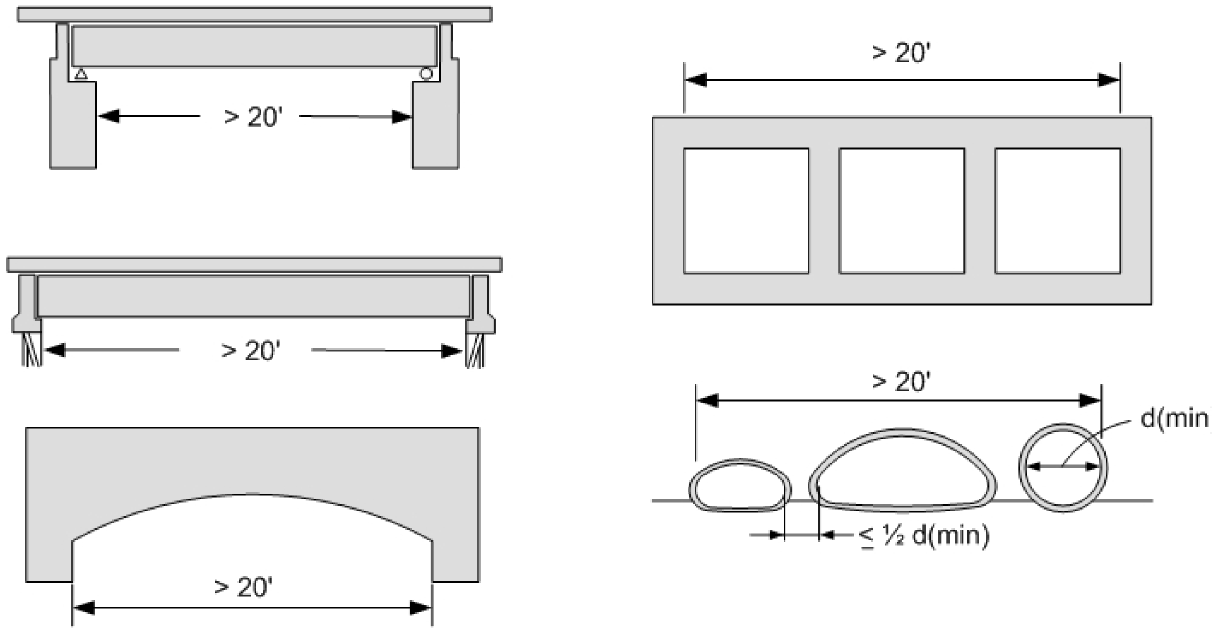


Figure 3.1.2 NBIS Bridge Length (Coding Guide Item 112)

23 CFR Part 650.305 Definitions gives the definition of a bridge as it applies to the NBIS regulations: A bridge is a structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

3.1.4 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 3.1.3):

- Deck
- Superstructure
- Substructure

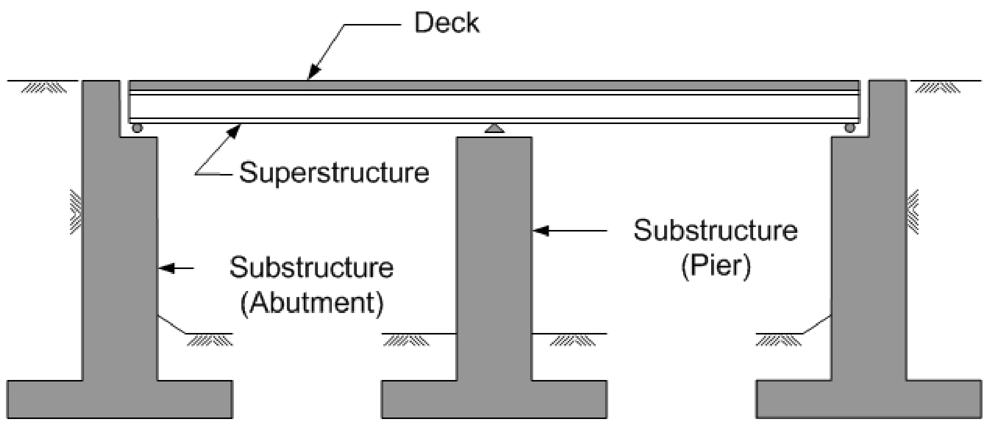


Figure 3.1.3 Major Bridge Components

3.1.5 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 5.1.

Timber Shapes

Basic shapes, properties, gradings, deficiencies, protective systems, and examination of timber are covered in detail in Topic 6.1.

Timber members are found in a variety of shapes (see Figure 3.1.4). The sizes of timber members are generally given in nominal dimensions (such as in Figures 3.1.4 and 3.1.5). However, sawn timber members are generally seasoned and surfaced from the rough sawn condition, making the actual dimension about 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:

- Round – piles, columns, posts
- Rectangular – planks, beams, columns, piles
- Built-up shapes - beams

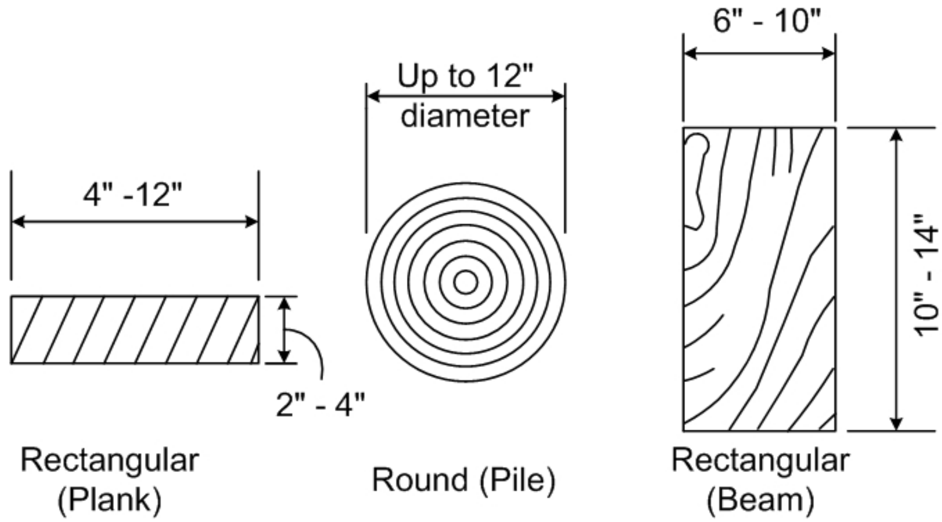


Figure 3.1.4 Timber Members

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. Common nominal or rough sawn dimensions for timber planks are 2 to 4 inches thick and 6 to 12 inches wide. Dressed lumber dimensions would be 1 ½ inches x 11 ¼ inches (see Figure 3.1.4).

Planks are most often used for bridge decks on bridges carrying light or infrequent truck traffic. Timber plank decks have been used for centuries. Timber planks are advantageous in that they are economical, lightweight, readily available, and easy to install.

Beams

Timber beams have more equal rectangular dimensions than do planks, and they are sometimes square. Common dimensions include 10 inch by 10 inch square timbers, and 6 inch by 14 inch rectangular timbers. Beams generally are installed with the larger dimension vertical.

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 built-up glued-laminated (see Figure 3.1.5). 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 deficiencies such as knots. Glued-laminated timbers are normally manufactured from well-seasoned wood and display very little shrinkage after they are fabricated.

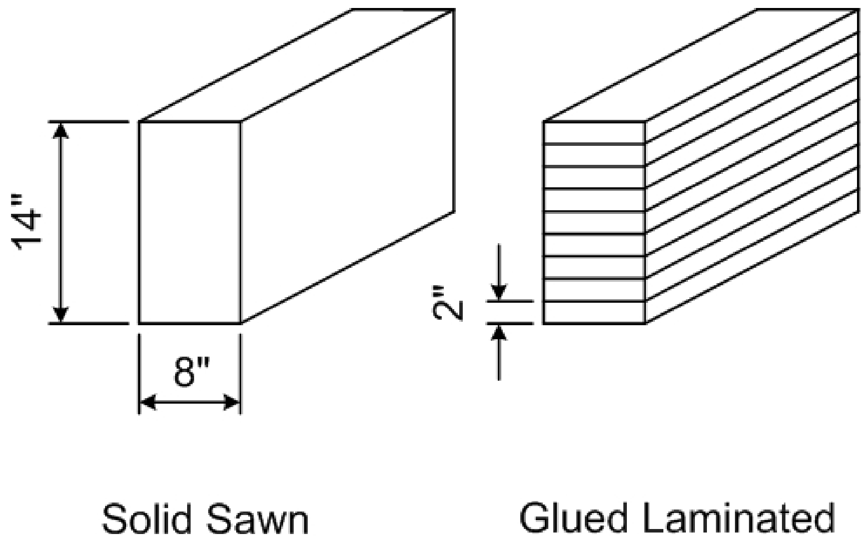


Figure 3.1.5 Timber Beams

Piles/Columns

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

Concrete Shapes

Basic ingredients, properties, reinforcement, deficiency, protective systems, and examination of concrete are covered in detail in Topic 6.2.

Figure 3.1.6 Unusual Concrete Shapes

Concrete is a unique material for bridge members because it can be formed into an infinite variety of shapes (see Figure 3.1.6). Concrete members are used to carry axial and bending loads. Since bending results in a combination of compressive and tensile stresses, concrete bending members are typically reinforced with either reinforcing steel bars (producing conventionally reinforced concrete) or with prestressing steel (producing prestressed concrete) in order to carry the tensile stresses in the member. Reinforcing steel is also added to increase the shear and torsion capacity of concrete members.

Cast-in-Place Flexural Shapes

The most common shapes of reinforced concrete members are (see Figure 3.1.7):

- Slabs/Decks
- 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.

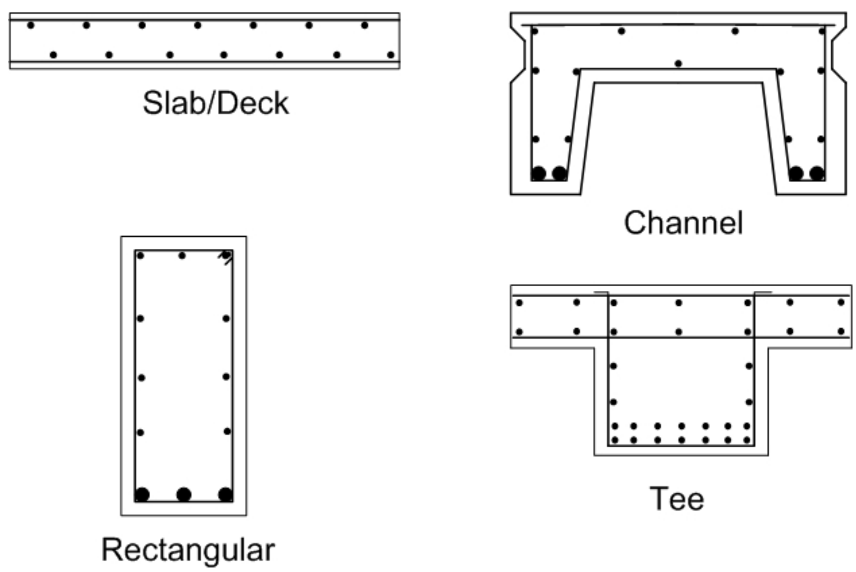


Figure 3.1.7 Reinforced Concrete Shapes

On concrete decks, the concrete spans the distance between superstructure members and is generally 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 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.

Tee beams are generally limited to superstructure elements. Distinguished by a "T" shape, tee beams combine the functions of a rectangular stem and flange to form an integral deck and superstructure.

Channel beams are generally limited to superstructure elements. This particular shape can be precast or 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.

Precast Flexural Shapes

The most common shapes of prestressed concrete members are (see Figure 3.1.8):

- I-beams
- Bulb-tees
- Voided or solid slabs
- Box beams
- Box girders

These shapes are used for superstructure members.

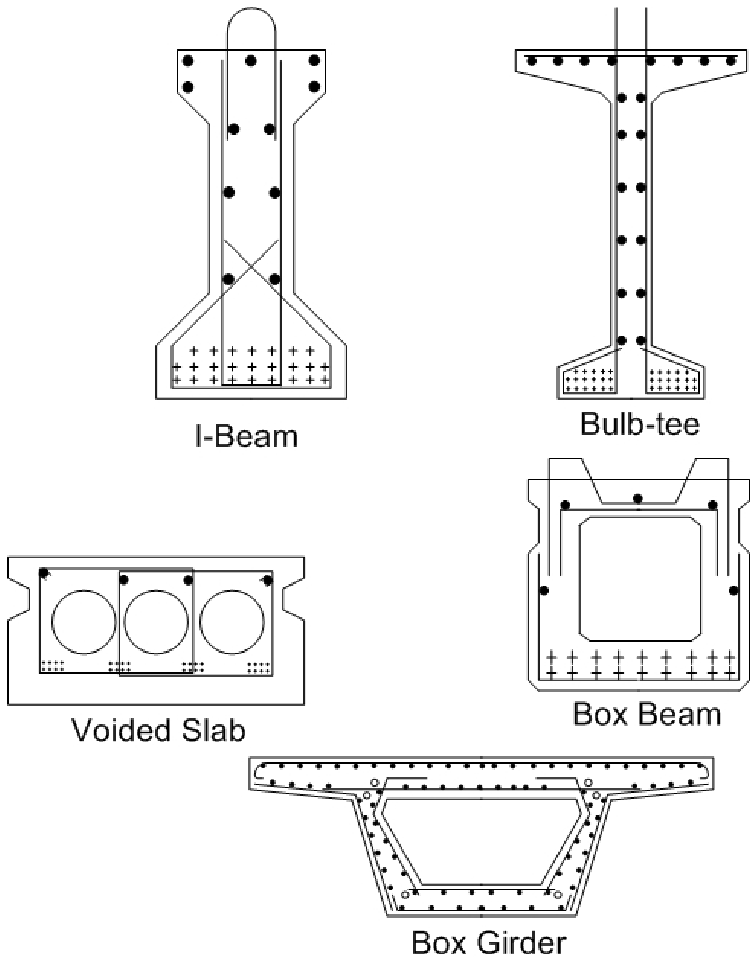


Figure 3.1.8 Prestressed Concrete Shapes

Prestressed concrete beams can be precast at a fabricator's plant using high compressive strength concrete. Increased material strengths, more efficient shapes, the prestress forces and closely controlled fabrication 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 3.1.9).

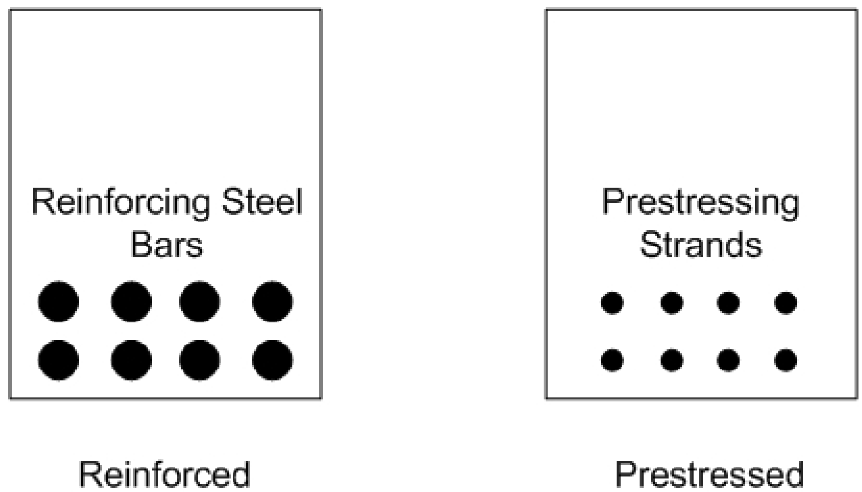


Figure 3.1.9 Non-prestressed Mild Steel Reinforced Concrete vs. Precast Prestressed Concrete

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 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 200 feet.

Box beams, distinguished by a square or rectangular shape, usually have a beam depth greater than 17 inches. Box beams can be adjacent or spread, and they are typically used for short and medium span bridges. Adjacent box beams have practical span lengths that range 40 to 130 feet and spread box beams have practical span lengths that range up to 130 feet.

Box girders, distinguished by their trapezoidal or rectangular box shapes, function as both deck and superstructure. Box girders are used for long span or curved bridges and can be precast and erected in segments or cast in place. Spans lengths can range from 130 to 1000 feet.

Voided slabs, distinguished by their rectangular shape and their interior voids, are generally precast units supported by the substructure. The interior voids are used to reduce the dead load. Voided slabs can be used for spans up to 60 feet.

Axially-Loaded Compression Members

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

- Columns
- Arches
- Piles

These members are conventionally reinforced to carry bending forces and to augment their compression load capacity.

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 3.1.10). Concrete piles may be conventionally reinforced or prestressed.

Figure 3.1.10 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 forging 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
- I-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 3.1.11). 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, A588, and, more recently, HPS steels have replaced early grades of steel, such as A7.

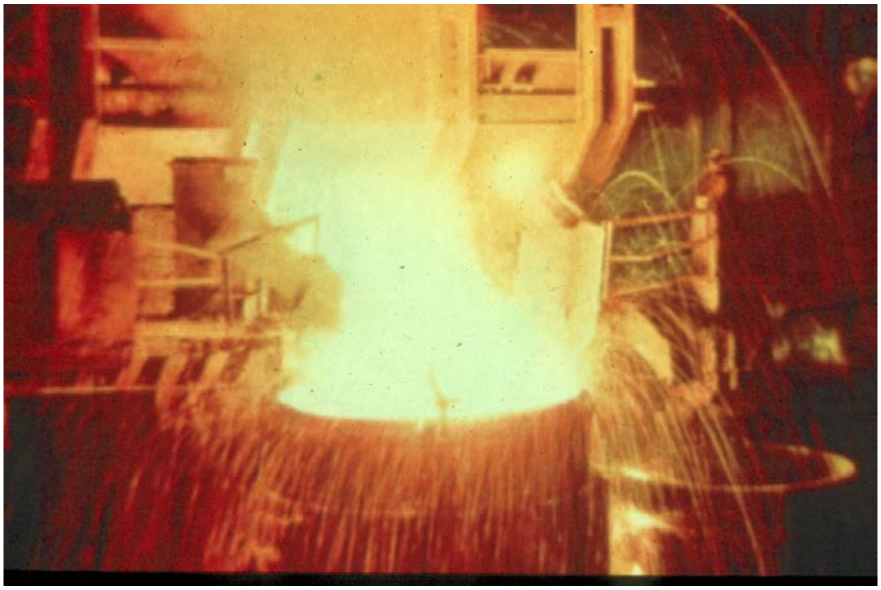


Figure 3.1.11 Steel Making Operation

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 3.1.12):

- Bars and plates
- Angles
- Channels
- S Beams (American standard “I” beams)
- W Beams (Wide flange “I” beams

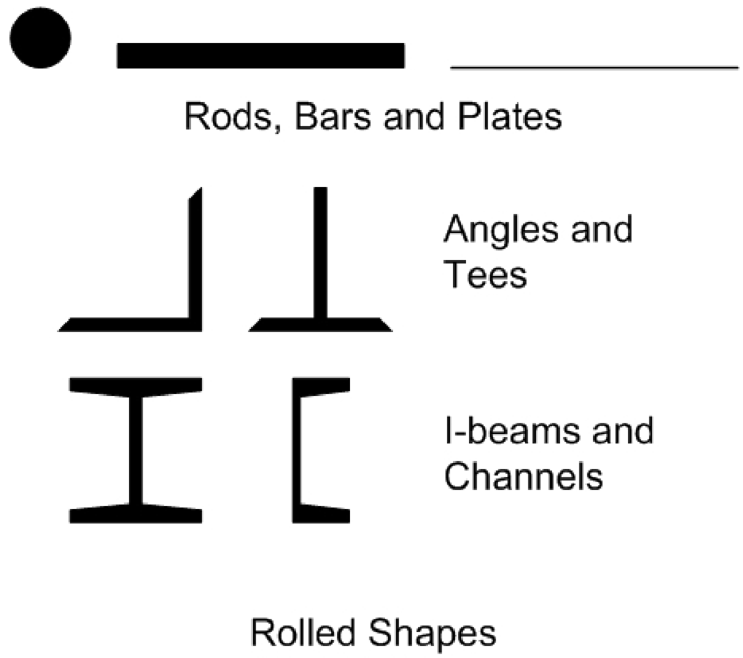


Figure 3.1.12 Common Rolled Steel Shapes

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 flat pieces of steel. Bars are normally considered to be up to 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 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 x thickness x length. Examples of bar and plate dimensions include:

- Lacing bar: 2" x 3/8" x 1'-3"
- Gusset plate: 21" x 1/2" x 4'-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"
- 2L's 5" x 3" x 3/8" x 1'-1"

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

Angles, bars, and plates are commonly connected to form bracing members (see Figure 3.1.13).

Figure 3.1.13 Bracing Members Made from Angles, Bars, and Plates

Channels are squared-off "C"-shaped members and are used as diaphragms, struts, or other bracing 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 inches, the weight in pounds per foot, and the length in inches. Examples of channel dimensions include:

- C 9 x 15 x 9'-6"
- C 12 x 20.7 x 11'-2-1/2"

When measuring a channel, it is not possible for the inspector to know how much the channel section weighs. In order to identify a channel, measurements of the average thickness, flange width, the web depth, and the thickness are needed. From this information, the inspector can then determine the true channel designation through the use of reference books such as American Institute of Steel Construction (AISC) *Manual of Steel Construction*.

Standard channels range in depth from 3 inches to 15 inches, and weights range from less than 5 pounds per foot to 50 pounds per foot. Nonstandard sections (called miscellaneous channels or MC) are rolled to depths of up to 18 inches, weighing up to 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 and weight per unit length.

Examples of beam dimensions include:

- S15x50 - an American Standard (hence the "S") beam with a depth of 15 inches and a weight of 50 pounds per foot
- W18x76 - 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

To identify an I-beam, measurements of the depth, the flange width and thickness, and the web thickness (if possible) are needed. With this information, the inspector can then determine the beam designation from reference books such as American Institute of Steel Construction (AISC) *Manual of Steel Construction*.

These beams normally range in depth from 3 to 36 inches and range in weight from 6 to over 300 pounds per foot. There are some steel mills that can roll beams up to 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 for their particular need. Built-up shapes are fabricated by either riveting, bolting or welding techniques.

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

Riveted girders are large I-beam members fabricated from plates and angles. These girders were used when the largest rolled beams were not large enough as required by design (see Figure 3.1.14).

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 3.1.15).

As technology improved, riveting was replaced by high strength bolting and welding. Popular since the early 1960's, welded steel shapes 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 3.1.16).

Welded boxes are large, rectangular-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 3.1.17).

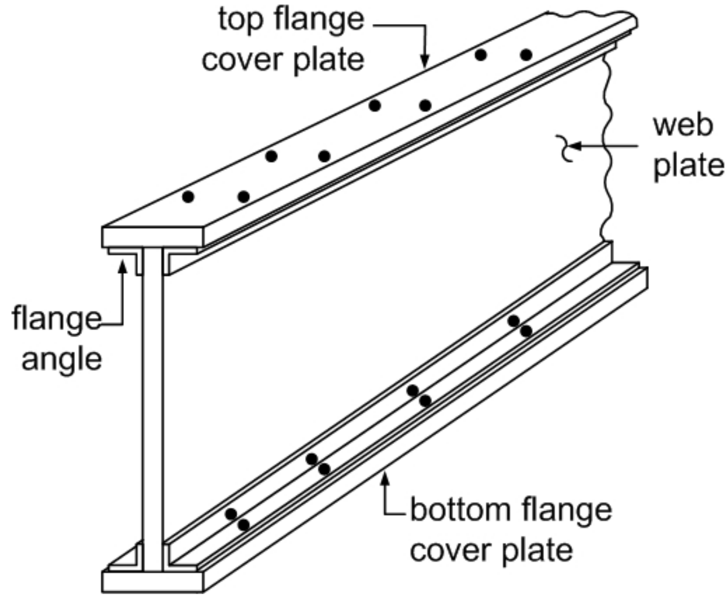


Figure 3.1.14 Riveted Plate Girder

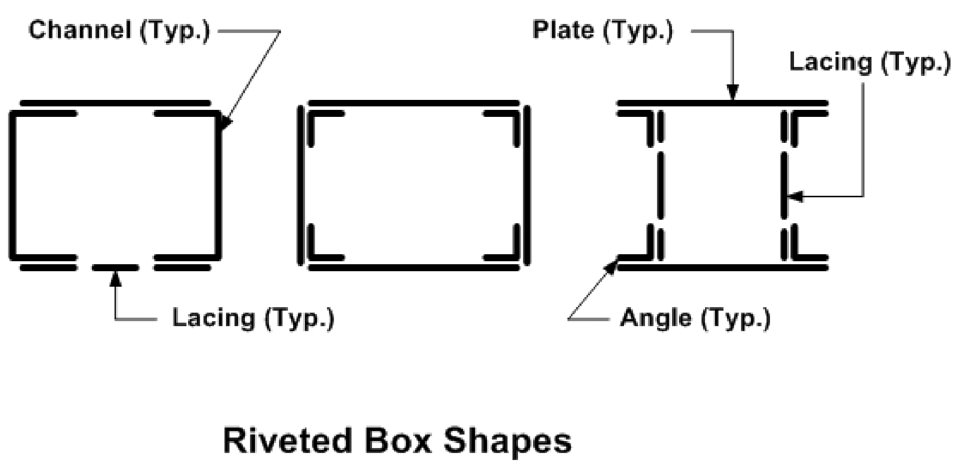


Figure 3.1.15 Riveted Box Shapes

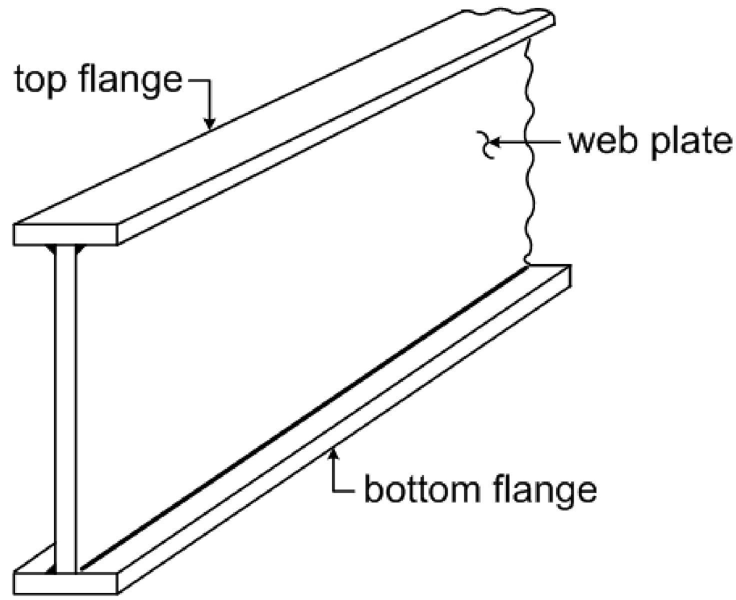


Figure 3.1.16 Welded I-Beam

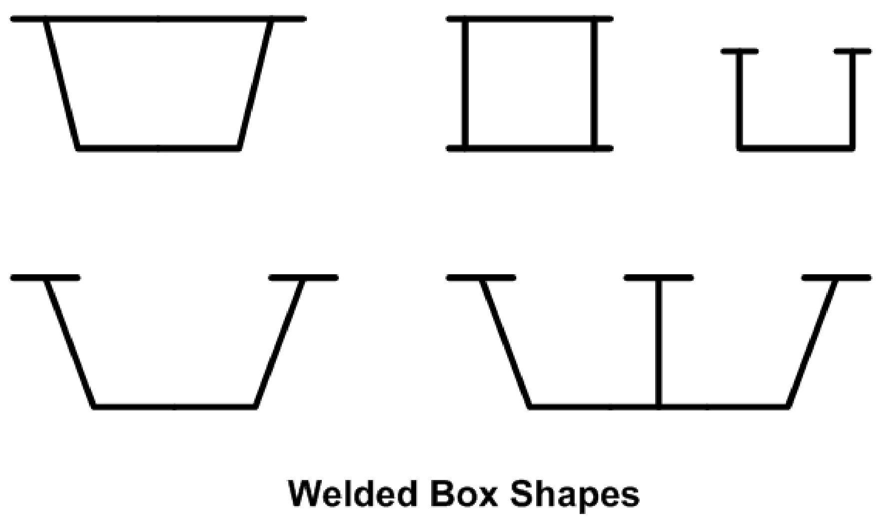


Figure 3.1.17 Welded Box Shapes

Cables

Steel cables (see Figure 3.1.18) 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 3.1.19 and 3.1.20). Refer to Topic 16.1 for a more detailed description of cable-supported bridges.

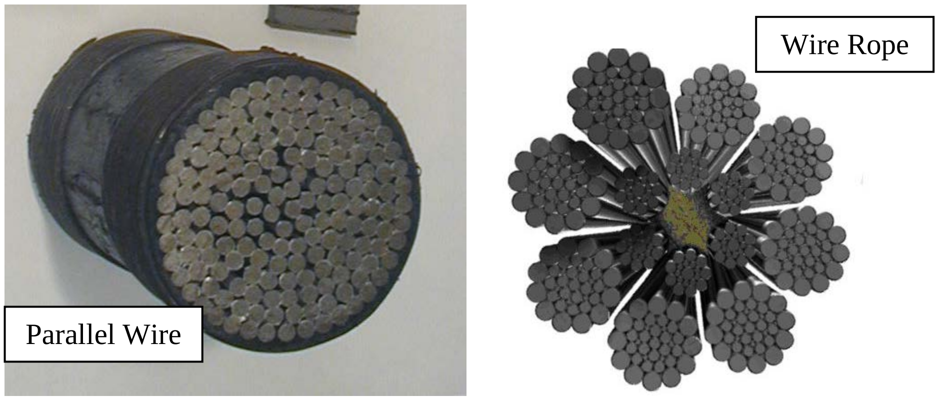


Figure 3.1.18 Cable Cross-Sections

Figure 3.1.19 Cable-Supported Bridge: Suspension Cables and Hangers

Figure 3.1.20 Cable-Supported Bridge: Cable Stayed

3.1.6 Connections

Rolled and built-up steel shapes are used to make stringers, floorbeams, girders, trusses, frames, arches and other bridge 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 assemblies
- Splice connections

Pin Connections

Pins are cylindrical bars produced by forging, casting, or cold-rolling. The pin sizes and configurations are as follows (see Figure 3.1.21):

- A small pin, 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 10 inches in diameter, usually has threaded end projections for recessed retainer nuts
- A large pin, over 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 member elements. Pin connections are commonly used in eyebar trusses, hinged arches, pin and hanger assemblies, and bearing supports (see Figure 3.1.22).

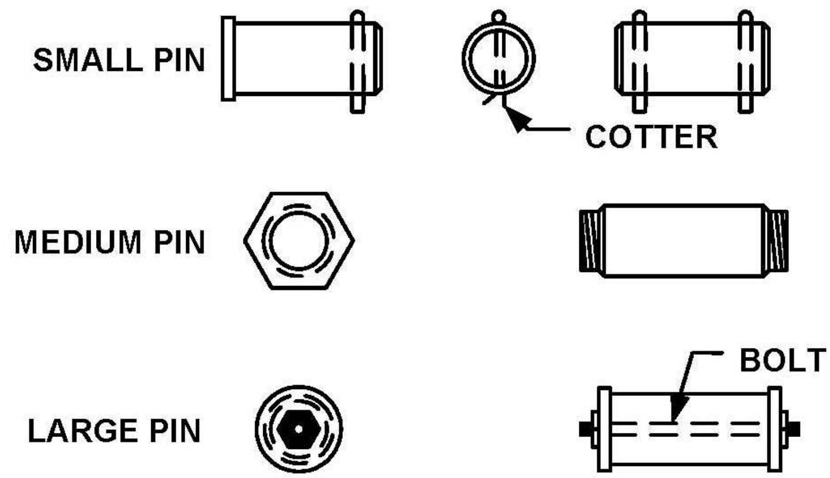


Figure 3.1.21 Sizes of Bridge Pins

Figure 3.1.22 Pin-Connected Eyebars

The major advantages of using pin connection details are the design simplicity and to facilitate rotation. The design simplicity afforded by pin connections reduces the amount and complexity of design calculations. By allowing for 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 were commonly used in trusses, suspended girder spans and some bearings. These pin connections are susceptible to freezing due to corrosion. This results in changes in structural behavior and undesirable stresses when axially-loaded members must resist bending.

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 may be needed (e.g., ultrasonic testing or pin removal).

Riveted Connections

The rivet was the primary fastener used in the early days of iron and steel bridges. 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 a hanger connection (see Figure 3.1.23).

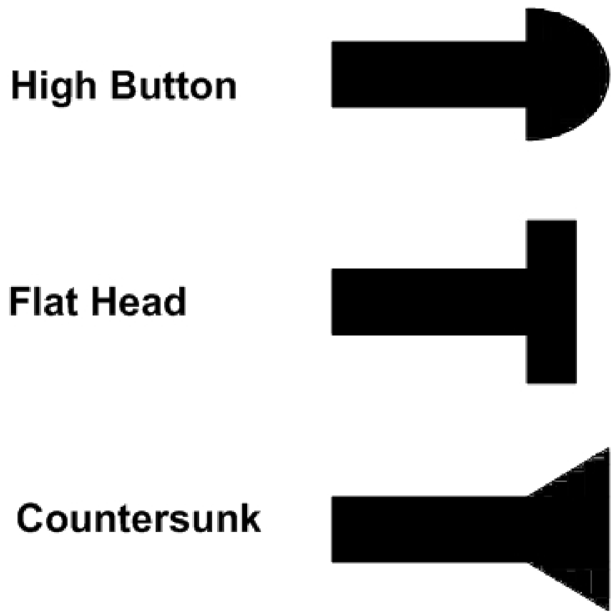


Figure 3.1.23 Types of Rivet Heads

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 rivet 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 such bolts were 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 3.1.24).

Structural bolts consist of three basic material designations:

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

For further information on the bolts listed above or any other material properties visit the American Society for Testing and Materials International website at: www.astm.org .

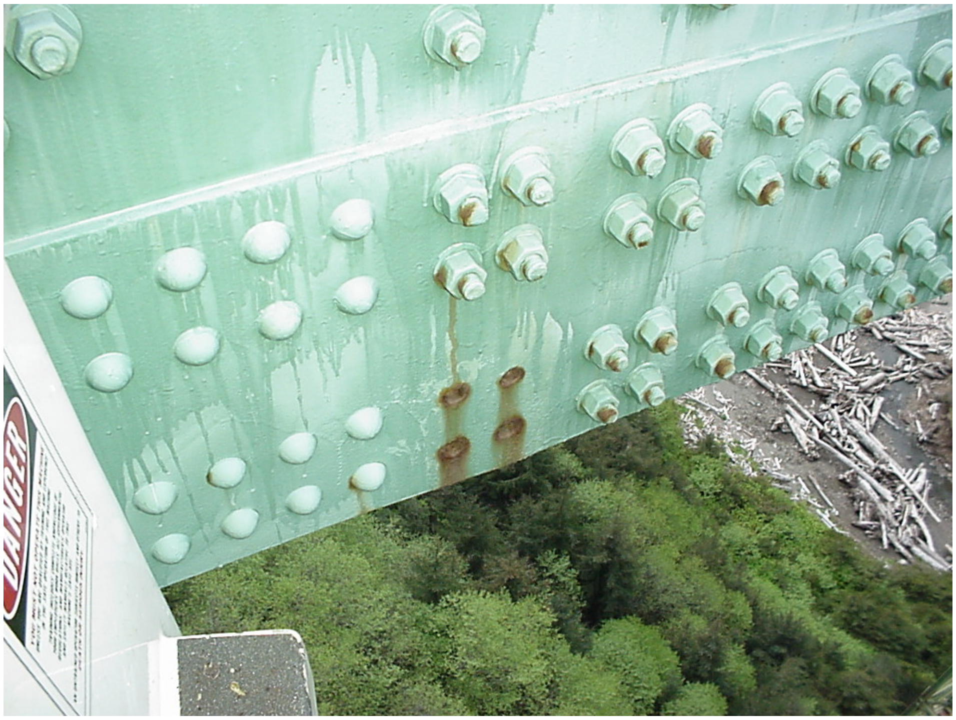


Figure 3.1.24 Shop Rivets and Field Bolts

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 if properly detailed.

Tightening high strength bolts puts them in tension, which clamps the member elements together. Although proper installation of new high strength bolts can be verified 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, and corrosion. Simple methods, such as visual observation, striking with a hammer and listening or feeling for loose bolts, are the most common methods used by inspectors when inspecting bolts.

Welded Connections

Pins, rivets, and bolts are examples of mechanical fasteners. A welded connection is not mechanical but rather is a rigid one-piece construction. A properly designed and executed 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 fatigue problems to certain bridge members (see Figure 3.1.25). Fatigue and fracture of steel bridge members are discussed in detail in Topic 6.4 (refer to 6.4.3 for factors affecting fatigue crack initiation). Welding is also used as a means of sealing joints and seams from moisture.

Figure 3.1.25 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 Assemblies

A pin and hanger assembly is a type of hinge consisting of two pins and two hangers. Pin and hanger assemblies are used in an articulated (continuous bridge with hinges) or a suspended span configuration. The location of the assembly varies depending on the type of bridge. In I-beam bridges, a hanger is located on either side of the webs (see Figure 3.1.26). In suspended span truss bridges, each assembly has a hanger which is similar in shape to the other connecting members (with the exception of the pinned ends). Pin and hangers were used to simply design before computer programs were developed to aid design of continuous bridges.

Figure 3.1.26 Pin and Hanger Assembly

Pin and hanger assemblies must be carefully inspected for signs of wear and corrosion. A potential problem can occur if corrosion of the pin and hanger causes the assembly to "freeze," inhibiting free rotation. This condition does not allow the pin to rotate and results in additional stresses in the pin and hanger and adjacent members. The failure of a pin and hanger assembly may 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 3.1.27). Shop welded flange splices are common in large welded plate girders and long truss members.

Figure 3.1.27 Bolted Field Splice

3.1.7 Decks

The deck is that component of a bridge to which the live load is directly applied. Refer to Chapter 7 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 3.1.28).

Figure 3.1.28 Bridge Deck with a Smooth Riding Surface

The function of the deck is to transfer live loads and dead loads of and on the deck to other bridge components commonly referred to as the superstructure (see Figure 3.1.29). However, on some bridges (e.g., a concrete slab bridge), the deck and the superstructure are one unit which distributes the live load directly to the substructure.

Figure 3.1.29 Underside View of a Bridge Deck

Deck Types

Decks function in one of two ways:

- Composite decks - act together with their supporting members and increase superstructure capacity (see Figures 3.1.30 and 3.1.31)
- Non-composite decks - are not integral with their supporting members and do not contribute to structural capacity of the superstructure

An inspector reviews the plans to determine if the deck is composite with the superstructure.

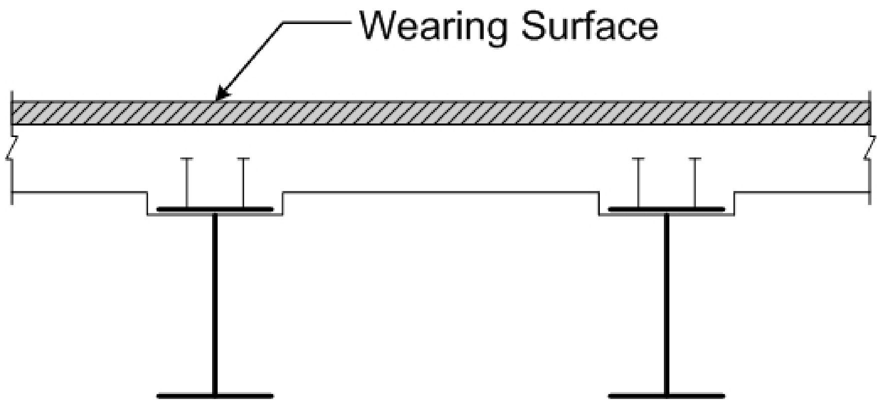


Figure 3.1.30 Composite Deck and Steel Superstructure

Figure 3.1.31 Shear Studs on Top Flange of Girder (before Concrete Deck is Placed)

Deck Materials

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

- Timber
- Concrete
- Steel

Fiber Reinforced Polymer (FRP) has been used, but are not as common.

Timber Decks

Timber decks are often referred to as decking or timber flooring, and the term is limited to the roadway portion which receives vehicular loads. Refer to Topic 7.1 for a detailed explanation on the inspection and evaluation of timber decks.

Five basic types of timber decks are:

- Plank deck (see Figure 3.1.32)
- Nailed laminated deck
- Glued-laminated deck planks
- Stressed-laminated decks
- Structural composite lumber decks

Figure 3.1.32 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 tensile stresses (see Figure 3.1.33). Refer to Topic 7.2 for a detailed explanation on the inspection and evaluation of concrete decks.

There are several common types of concrete decks:

- Conventionally reinforced cast-in-place - removable or stay-in-place forms
- Precast conventionally reinforced
- Precast prestressed
- Precast prestressed deck panels with cast-in-place topping

Figure 3.1.33 Concrete Deck

Steel Decks

Steel decks are decks composed of either solid steel plate or steel grids (see Figure 3.1.34). Refer to Topic 7.4 for a detailed explanation on the inspection and evaluation of steel decks.

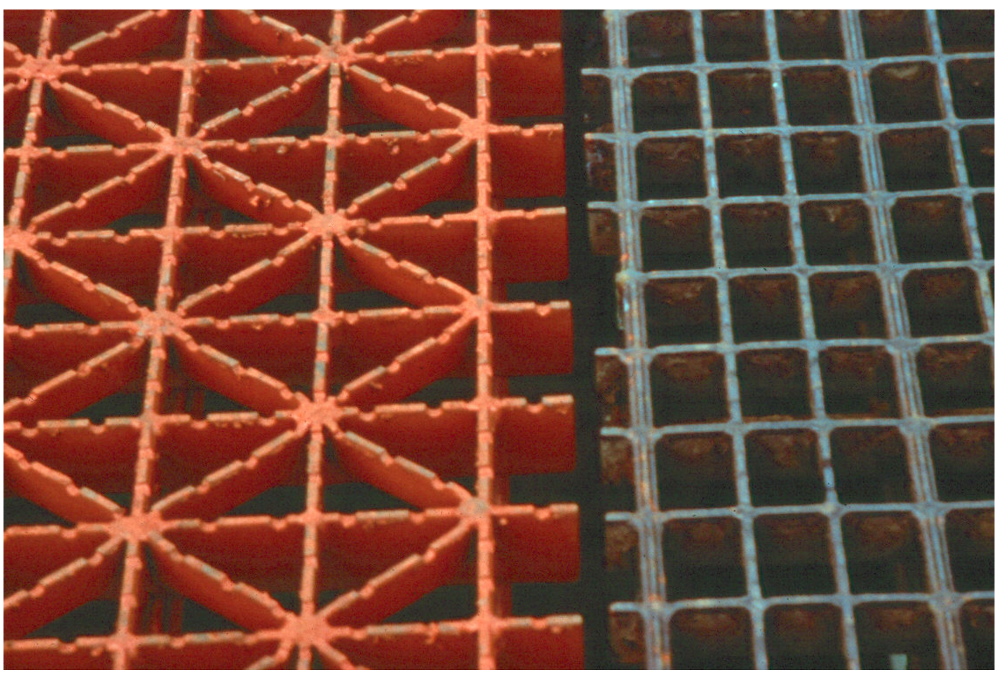


Figure 3.1.34 Steel Grid Deck

There are four common types of steel decks:

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

Fiber Reinforced Polymer (FRP) Decks

With the rise of technological development, innovative material such as fiber-reinforced polymer (FRP) bridge decking has begun replacing existing highway bridge decks. Though FRP material is more expensive than conventional bridge materials such as concrete, it has several advantages. These include lighter weight for efficient transport, better resistance to earthquakes, and easier installation. FRP bridge decking is also less affected by water and de-icing salts, which corrode steel and deteriorate concrete (see Figure 3.1.35). Refer to Topic 7.3 for a detailed explanation on the inspection and evaluation of FRP decks.

Figure 3.1.35 Fiber Reinforced Polymer (FRP) Deck

Wearing Surfaces

Constant exposure to the elements makes weathering a significant cause of deck deficiency. 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 to 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 – running boards
- Bituminous
- Concrete
- Gravel
- Polymers

Concrete decks may have wearing surfaces of:

- Concrete – latex modified concrete (LMC), low slump dense concrete (LSDC), lightweight concrete (LWC), fiber reinforced concrete (FRC), micro-silica modified concrete
- Bituminous (see Figure 3.1.36)
- Polymers - epoxy, polyester, methyl methacrylates

Figure 3.1.36 Asphalt Wearing Surface on a Concrete Deck

Steel decks may have wearing or riding surfaces of:

- Serrated steel
- Concrete
- Asphalt
- Polymers

Deck 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. Refer to Topic 7.5 for detailed explanation on the inspection and evaluation of deck joints.

There are six categories of deck joints:

- Strip seal expansion joints (see Figure 3.1.37)
- Pourable joint seals
- Compression joint seals (see Figure 3.1.38)
- Assembly joints with seal (Modular)
- Open expansion joints
- Assembly joints without seals (finger plate and sliding plate joints) (see Figure 3.1.39)

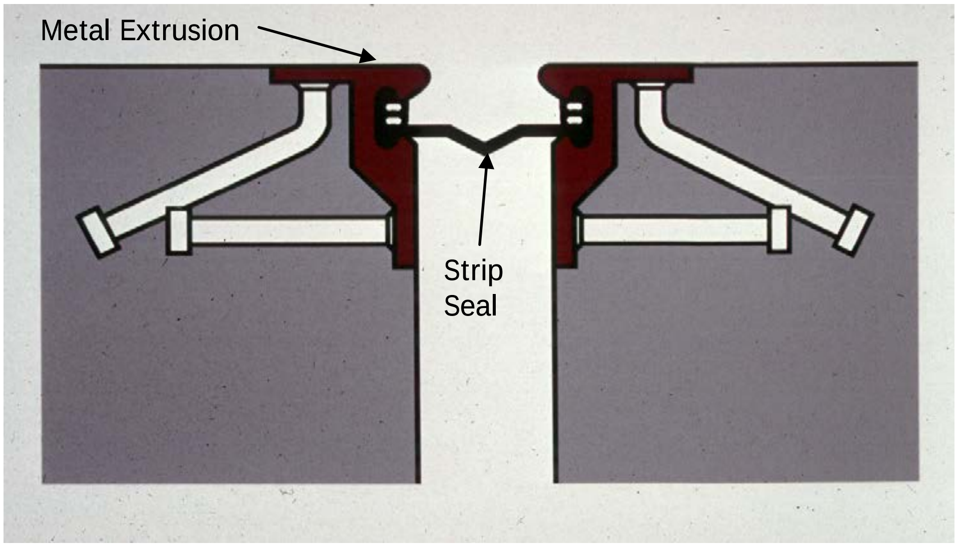


Figure 3.1.37 Strip Seal Expansion Joint

Figure 3.1.38 Top View of an Armored Compression Seal in Place

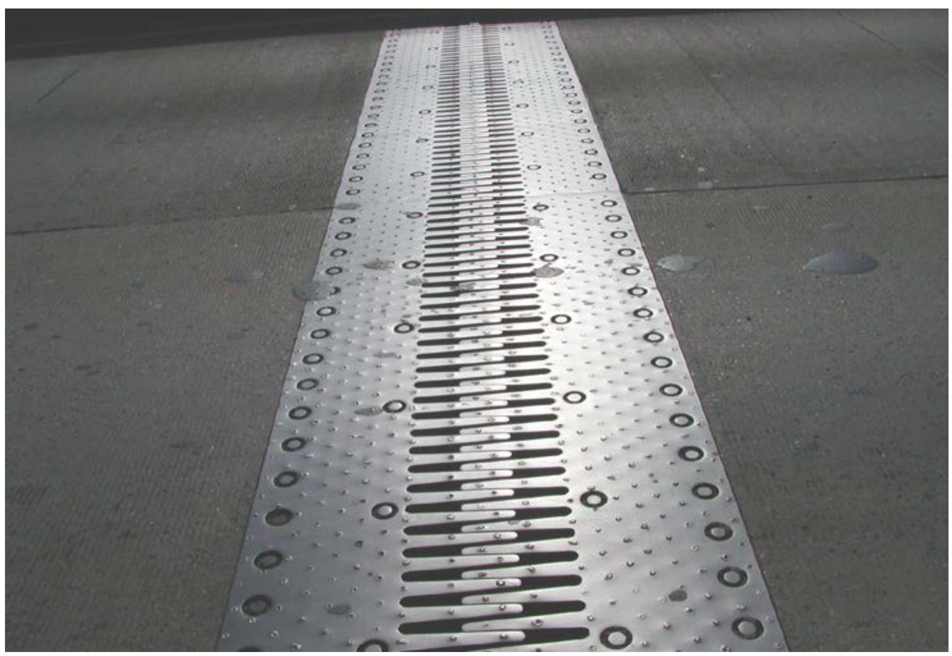


Figure 3.1.39 Top View of a Finger Plate Joint

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. Refer to Topic 7.5 for detailed explanation on the inspection and evaluation of drainage systems.

A deck drainage system has the following components:

- Grade and cross slope
- Inlets
- Outlet pipes
- Downspout pipes - to transport runoff to storm sewers
- Cleanout plugs - for maintenance
- Drainage troughs
- Support brackets/hardware

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 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.

Traffic Safety Features

The proper and effective use of traffic barriers minimizes hazards for traffic on the bridge, on the highways, and waterways beneath the bridge.

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
- Concrete pigeon hole parapet
- Combination bridge-pedestrian aluminum or steel railing
- New Jersey barrier - a very common concrete barrier (see Figure 3.1.40)

Refer to Topic 7.6 for detailed explanation on the inspection and evaluation of traffic safety features.

Figure 3.1.40 New Jersey Barrier

Sidewalks and Curbs

The function of sidewalks and curbs is to provide access to and maintain safety for pedestrians and to direct water to the drainage system. 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. Refer to Topic 7.5 for detailed explanation on the inspection and evaluation of signing.

Several signs likely to be encountered are:

- Weight limit and/or lane restrictions (see Figure 3.1.41)
- Speed traffic marker
- Vertical clearance
- Lateral clearance
- Narrow underpass
- Informational and directional
- Object markers

Figure 3.1.41 Weight Limit Sign and Object Marker Signs

Lighting

Types of lighting that may be encountered on a bridge include the following (see Figure 3.1.42):

- Highway lighting
- Traffic control lights
- Aerial obstruction lights
- Navigation lights
- Signing lights
- Illumination and drawbridge operation flashing lights

Refer to Topic 7.5 for detailed explanation on the inspection and evaluation of lighting systems.

Figure 3.1.42 Bridge Lighting

3.1.8 Superstructure

Superstructure Purpose

The basic purpose of the superstructure is to carry loads from the deck across the span and to the bridge supports commonly referred to as the substructure. 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.

The function of the superstructure is to span a feature and to transmit loads from the deck to the bridge supports commonly referred to as the substructure. Bridges are categorized by their superstructure type. 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.

Superstructure Types

There are many different superstructure types such as:

- Slabs
- Single web beams/girders
- Box beams/girders (multi-web)
- Trusses
- Arches
- Rigid frames
- Cable-supported bridges
- Movable bridges
- Floating bridges

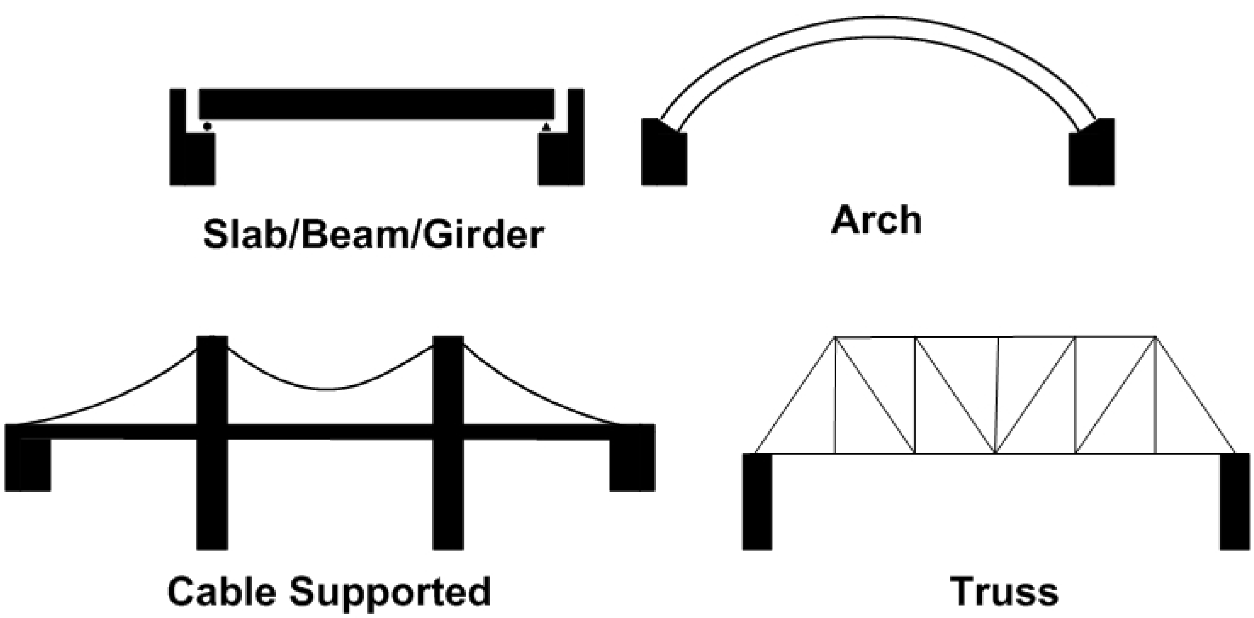


Figure 3.1.43 Four Basic Bridge Types

Slab Bridges

In slab bridges, loads from the slab are transmitted vertically to the substructure (see Figure 3.1.44).

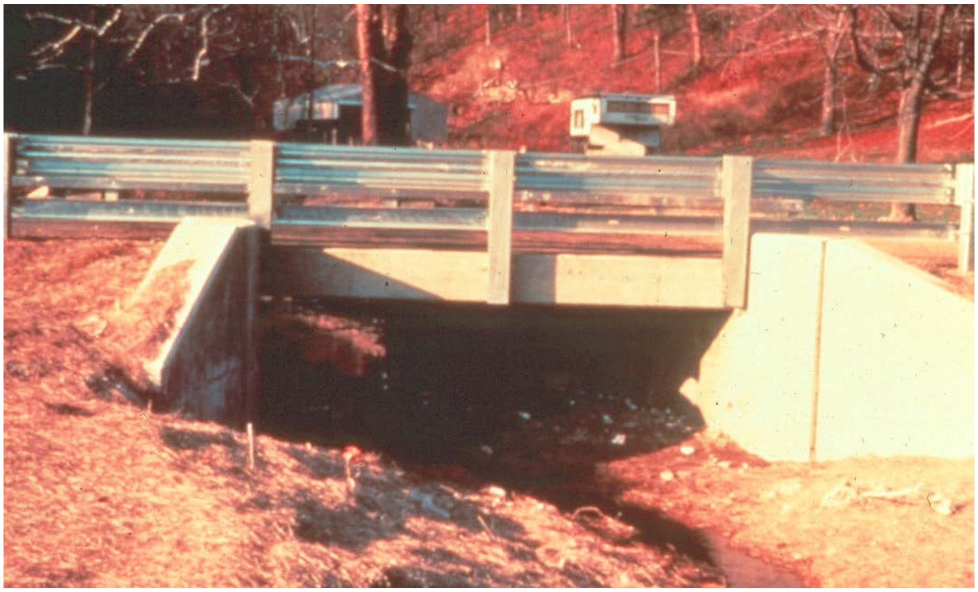


Figure 3.1.44 Slab Bridge

Single Web Beam/Girder Bridges

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

- Beams (timber, concrete, or steel) (see Figures 3.1.45, 3.1.49, 3.1.50)
- Girders (concrete or steel) (see Figures 3.1.46, 3.1.47, 3.1.48, 3.1.51)

Figure 3.1.45 Beam Bridge

Figure 3.1.46 Multi-Girder Bridge

Figure 3.1.47 Girder Floorbeam Stringer Bridge

Figure 3.1.48 Curved Girder Bridge

Figure 3.1.49 Tee Beam Bridge

Figure 3.1.50 Adjacent Box Beam Bridge

Figure 3.1.51 Box Girder Bridge

Trusses

Truss members including chords, verticals, and diagonals primarily carry axial tension and compression loads. Trusses can be constructed from timber or steel (see Figures 3.1.52 and 3.1.53).

Figure 3.1.52 Deck Truss Bridge

Figure 3.1.53 Through Truss Bridge

Arches

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 3.1.54 and 3.1.55).

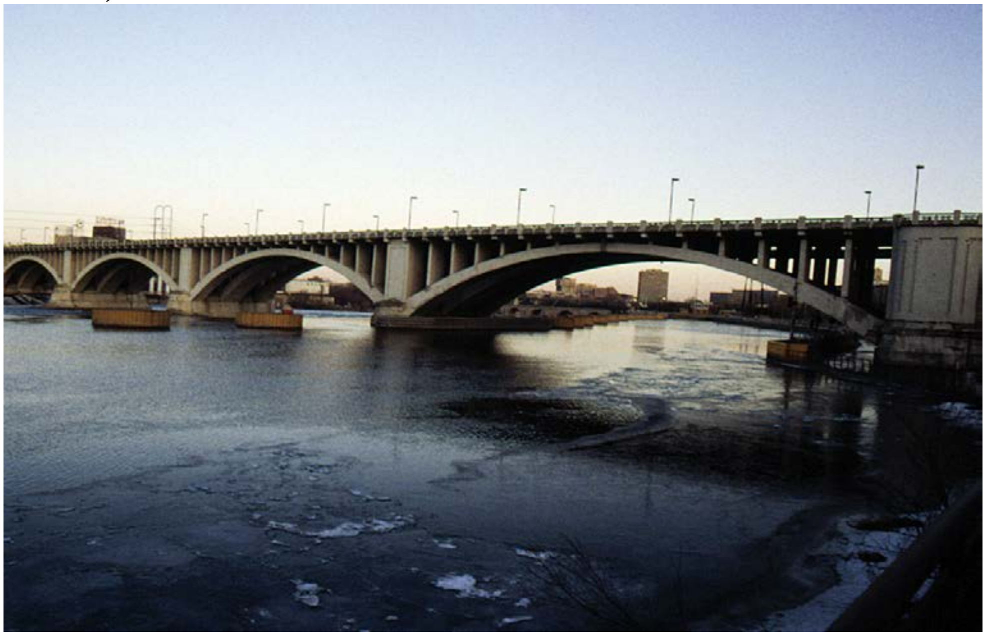


Figure 3.1.54 Deck Arch Bridge

Figure 3.1.55 Through Arch Bridge

Rigid Frames

Rigid frame superstructures are characterized by rigid (moment) connections between the horizontal girder and the legs. This connection allows the transfer of both axial forces and moments into vertical or sloping elements, which may be classified as superstructure or substructure elements depending on the exact configuration. Similar to beam/girder or slab configurations, rigid frame systems may be multiple parallel frames or may contain transverse floorbeams and longitudinal stringers to support the deck. (see Figure 3.1.56)

Figure 3.1.56 Rigid Frame

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 3.1.57 and 3.1.58). Refer to Topic 16.1 for a more detailed explanation on cable-supported bridges.

Figure 3.1.57 Suspension Bridge

Figure 3.1.58 Cable-stayed Bridge

Movable Bridges

Movable bridges are constructed across designated "Navigable Waters of the United States," in accordance with "Permit Drawings" approved by the U.S. Coast Guard or other agencies. 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 16.2 for a more detailed explanation on movable bridges.

Movable bridges can be classified into three general groups:

- Bascule (see Figure 3.1.59)
- Swing (see Figure 3.1.60)
- Lift (see Figure 3.1.61)

Figure 3.1.59 Bascule Bridge

Figure 3.1.60 Swing Bridge

Figure 3.1.61 Lift Bridge

Floating Bridges

Although uncommon, some states have bridges that are not supported by a substructure (see Figure 3.1.62). Instead, they are supported by water. The elevation of the bridge will change as the water level fluctuates.

Figure 3.1.62 Floating Bridge

Superstructure Materials

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

- Timber
- Concrete
- Steel

Primary Members

Typical primary members carry primary live load from trucks and typically consist of the following:

- Girders (see Figure 3.1.63)
- Floorbeams (see Figure 3.1.63)
- Stringers (see Figure 3.1.63)
- Trusses
- Spandrel girders (see Figure (3.1.64)
- Spandrel columns (see Figure (3.1.64) or bents
- Arch ribs
- Rib chord bracing
- Hangers
- Frame girder
- Frame leg
- Frame knee
- Pin and hanger links

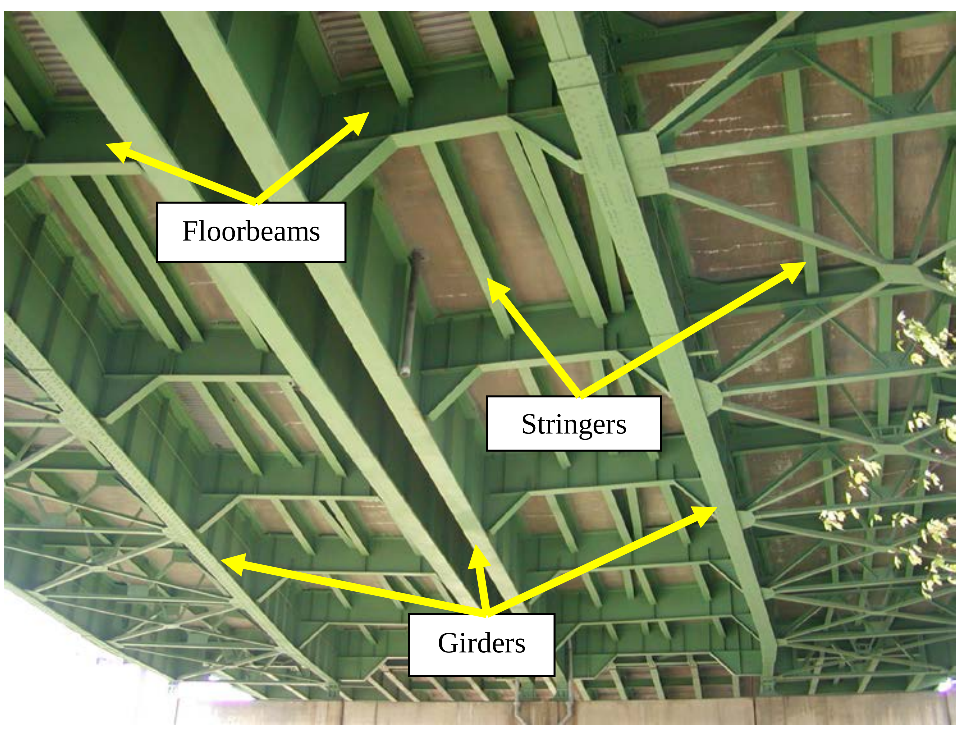


Figure 3.1.63 Floor System and Main Supporting Members

Additionally, diaphragms for curved girders may also be considered primary members. Vehicular live load is transmitted between the mains supporting members through the diaphragms in a curved multi-girder arrangement.

Figure 3.1.64 Main Supporting Members of Deck Arch

Secondary Members

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

- Diaphragms (see Figure 3.1.65)
- Cross or X-bracing (see Figure 3.1.66)
- Lateral bracing (see Figure 3.1.67)
- Sway-portal bracing (see Figure 3.1.67)
- Pin and hanger assemblies - Through bolts**,** pin caps, nuts, cotter pins on small assemblies, spacer washers, doubler plates

Figure 3.1.65 Diaphragms

Figure 3.1.66 Cross or X-Bracing

Figure 3.1.67 Top Lateral Bracing and Sway Bracing

3.1.9

Bearings

Bearing Purpose

A bridge bearing is an element which provides an interface between the superstructure and the bridge supports referred to as the substructure.

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 horizontal movement of the superstructure are referred to as fixed bearings. Bearings that allow for horizontal movement of the superstructure are known as expansion bearings. Both fixed and expansion bearings permit rotation. Refer to Topic 11.1 for more detailed explanation on expansion/fixed bearings.

Bearing Types

There are six bearing types that are utilized to accommodate superstructure movement and rotation:

- Elastomeric bearings
- Moveable bearings (roller, sliding, etc.)
- Enclosed/concealed bearings
- Fixed bearings
- Pot bearings
- Disk bearing

Refer to Topic 11.1 for detailed explanations on bridge bearing types.

Bearing Materials

There are two common materials used in the construction of bridge bearings:

- Steel
- Neoprene

Bearing Elements

A bridge bearing can be normally categorized into four basic elements (see Figure 3.1.68):

- Sole plate
- Bearing device
- Masonry plate
- Anchor bolts

Refer to Topic 11.1 for detailed explanations of these four bearing elements.

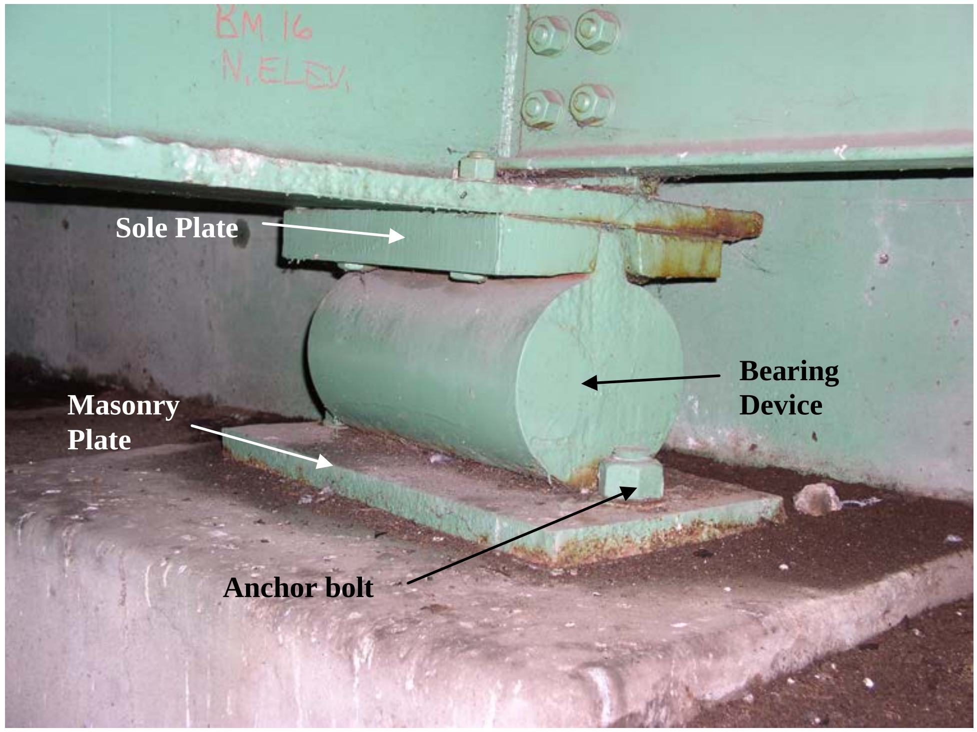


Figure 3.1.68 Steel Roller Bearing Showing Four Basic Elements

3.1.10 Substructure

The substructure is the 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.

Substructure units function as both axially-loaded and bending members. These units resist both vertical and horizontal loads applied from the superstructure and roadway embankment. Substructures are divided into two basic categories:

- Abutments
- Piers and bents

Abutments provide support for the ends of the superstructure and retain the roadway approach embankment (see Figure 3.1.69). Piers and bents provide support for the superstructure at intermediate points along the bridge spans (see Figure 3.1.70).

Figure 3.1.69 Abutment

Figure 3.1.70 Pier

Substructure Types

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 3.1.71).
- ➢ 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

abutment stem is visible than in the case of the full height abutment. Most new construction uses this type of abutment. These abutments may be supported on deep foundations (see Figure 3.1.72).

- 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 3.1.73).
- Integral abutment – superstructure and substructure are integral and act as one unit without an expansion joint or bearings. Relative movement of the abutment with respect to the backfill allows the structure to adjust to thermal expansions and contractions. Pavement relief joints at the ends of approach slabs are provided to accommodate the thermal movement between bridge deck and the approach roadway pavement (see Figure 3.1.74)

Figure 3.1.71 Cantilever Abutment (or Full Height Abutment)

Figure 3.1.72 Stub Abutment

Figure 3.1.73 Spill-Through or Open Abutment

Figure 3.1.74 Integral Abutment

Refer to Topic 12.1 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 12.2 for a more detailed explanation on bridge piers and bents.

There are four basic types of piers:

- Solid shaft pier (see Figures 3.1.75 and 3.1.76)
- Column pier (see Figure 3.1.77)
- Column pier with web wall (see Figure 3.1.78)
- Cantilever or hammerhead pier (see Figure 3.1.79)

Figure 3.1.75 Solid Shaft Pier

Figure 3.1.76 Solid Shaft Pier

Figure 3.1.77 Column Pier

Figure 3.1.78 Column Pier with Web Wall and Cantilevered Pier Caps

Figure 3.1.79 Cantilever or Hammerhead Pier

There are two basic types of bents:

- Column bent (see Figure 3.1.80)
- Pile bent (see Figure 3.1.81)

Figure 3.1.80 Column Bent

Figure 3.1.81 Pile Bent

Substructure Materials

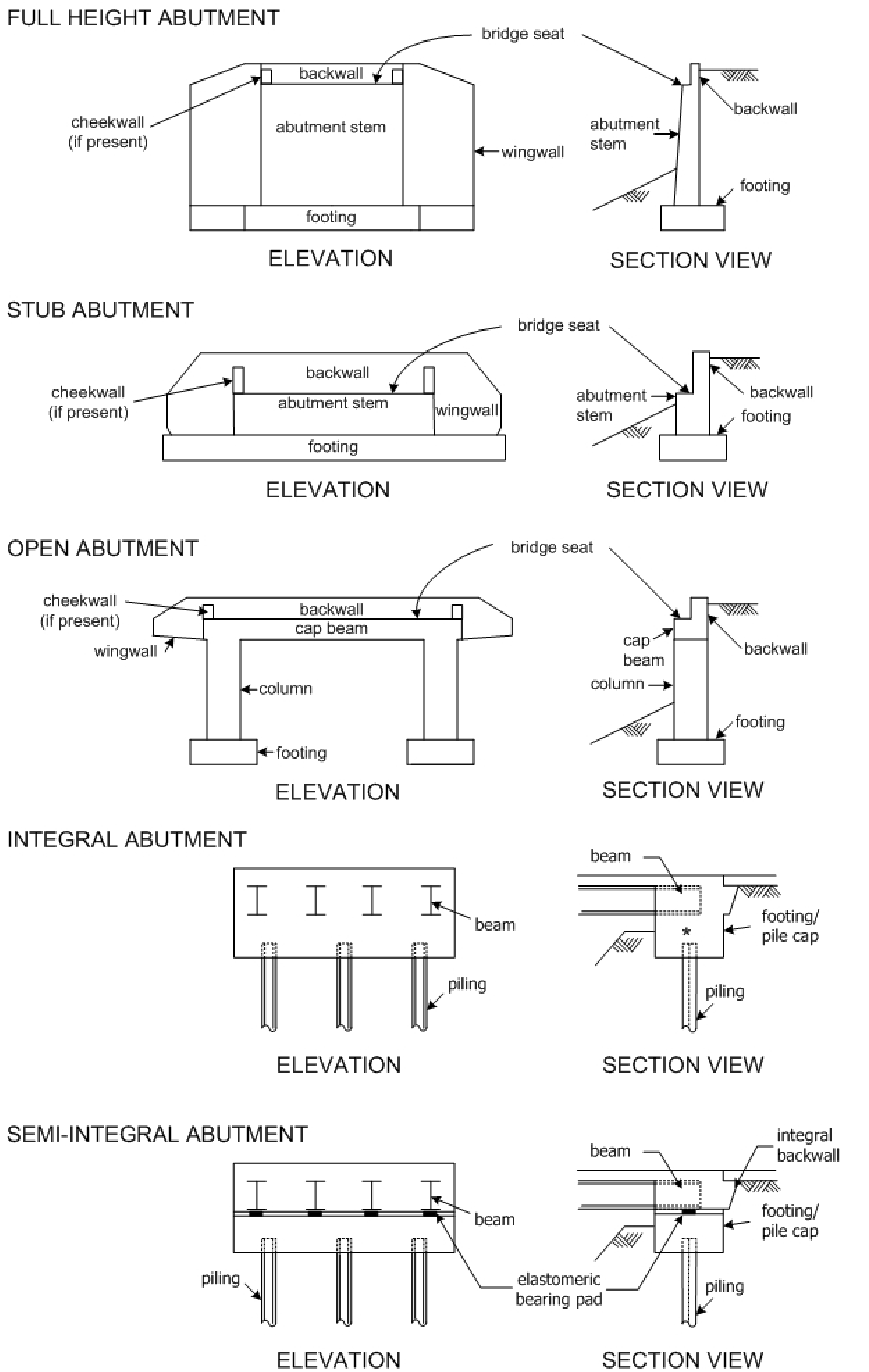
There are four common materials used in the construction of bridge substructures:

- Timber
- Concrete
- Steel
- Masonry

Substructure Elements

A bridge substructure can consist of several different elements (see Figure 3.1.82). Typical elements can include:

- Abutments
    - Backwall
    - Stem/bridge seat
    - Footing
    - Integral backwall



* Some states weld beam and piles prior to concrete placement

Figure 3.1.82 Schematic of Common Abutment Types

- Pier/Bents
  - Pier caps
  - Columns/Piles
  - Walls
  - Footing

Refer to Topics 12.1 and 12.2 for a detailed explanation of abutment, pier and bent elements.

3.1.11 Culverts

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. Culverts that are 20 feet or greater are defined as a bridge, according the NBIS definition for bridge length (see Topic 3.1.3).

Culvert Purpose

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

Culvert Materials

There are several common materials used in the construction of culverts:

- Concrete
- Masonry
- Steel
- Aluminum
- Timber
- Plastic

Refer to Topic 14.1 for a detailed explanation about culvert characteristics.

Culvert Types

Rigid Culverts

Rigid culverts can carry the load the same way a frame or an arch does by resisting the loads in bending and shear or frame an arch action (see Figure 3.1.83). Refer to Topic 14.2 for a detailed explanation of rigid culverts.

Figure 3.1.83 Rigid Culvert

Flexible Culverts

Flexible culverts will require lateral earth pressure to help maintain their shape. The loads are distributed through the flexible culvert and backfill. The backfill is critical to a flexible culverts performance (see Figure 3.1.84). Refer to Topic 14.3 for a detailed explanation of flexible culverts.

Figure 3.1.84 Flexible Culvert

Table of Contents

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Bridge Inspection Reporting

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Chapter 4
Bridge Inspection Reporting

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 are fulfilled in accordance with 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* (*FHWA Coding Guide*) is used for defining the bridge inventory and the items to be used to collect information on the overall condition of the deck, superstructure, substructure, and channel. The data is reported to FHWA in accordance with the *FHWA Coding Guide*. It is not an inspection guide. Each state may use its own coding scheme, provided that the data is directly translatable into the format of the *FHWA Coding Guide*. In other words, the states are responsible for having the capability to obtain, store, and report certain information about bridges, for collection by FHWA as requested.

The Structure Inventory and Appraisal (SI&A) sheet is a tabulation of information that is submitted for each individual structure (see Figure 4.1.1).

For the small structures and culverts that are less than or equal to 20 feet, some states still collect the inventory information and generate a "local" database.

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 Program.

Substitutes for the SI&A Sheet

There are suitable substitutes for the SI&A sheet. 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 through 4.1.5).

Data Entry Requirements

For routine, in-depth, fracture critical member, underwater, damage and special inspections, the NBIS requires entry of the SI&A data into the State or Federal agency inventory within 90 days of the date of inspection for State or Federal agency bridges and within 180 days of the date of inspection for all other bridges.

For existing bridge modifications that alter previously recorded data and for new bridges, the NBIS requires entry of the SI&A data into the State or Federal agency inventory within 90 days after the completion of the work for State or Federal agency bridges and within 180 days after the completion of the work for all other bridges.

For changes in load restriction or closure status, the NBIS requires entry of the SI&A data into the State or Federal agency inventory within 90 days after the change in status of the structure for State or Federal agency bridges and within 180 days after the change in status of the structure for all other bridges.

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

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

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

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

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

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

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 Wdth 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

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

Figure 4.1.1 Example SI&A Sheet with Element Level Data

Appendix A

OMB No. 2125-0501

Structure Inventory and Appraisal Sheet

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 Typical SI&A Sheet with NBI Data Only

Oregon Department of Transportation

Bridge Inspection Report

District	07	**Structure**	Coos Bay, Hwy 9 (McCullough)	**Bridge ID**	**01823**
Fac Crossed	COOS BAY (MCCULLOUGH BR)	**Owner**	State Highway Agency	**Fac Carried**	**US101(HWY009)**
Suff Rating	46.5	**County**	Coos	**Mile Point**	**233.99mi**
AC Depth	0.00	**Record Type**	1	**Insp Date**	06/11/2009
Bridge Length	5305.00ft	**Insp Freq**	24	**Inspector 1**	**Jeff Swanstrom (2010)**
		Bridge Width	33.80ft	**Inspector 2**	**JOHN MILCAREK (241)**

Signature:______________________________

Element Condition States

Elem	Description	Env	Qty	Units	1	2	3	4	5	status
18	Concrete Deck - Protected w/ Thin Overlay	Sev.	152100.00sqft	(SF)	0%	100%	0%	0%	0%	
110	Reinforced Conc Open Girder/Beam	Sev.	3332.00ft	(LF)	70%	20%	10%	0%	0%	
113	Painted Steel Stringer	Sev.	15372.00ft	(LF)	47%	48%	5%	0%	0%	
121	Painted Steel Bottom Chord Thru Truss	Sev.	3416.00ft	(LF)	47%	48%	5%	0%	0%	
126	Painted Steel Thru Truss (excl. bottom chord)	Sev.	3416.00ft	(LF)	45%	50%	5%	0%	0%	
144	Reinforced Conc Arch	Sev.	5522.00ft	(LF)	78%	20%	2%	0%	0%	
152	Painted Steel Floor Beam	Sev.	2090.00ft	(LF)	50%	48%	2%	0%	0%	
155	Reinforced Conc Floor Beam	Sev.	4862.00ft	(LF)	80%	15%	5%	0%	0%	
205	Reinforced Conc Column or Pile Extension	Sev.	64	(EA)	75%	20%	5%	0%	0%	
210	Reinforced Conc Pier Wall	Sev.	11	(EA)	20%	75%	5%	0%	0%	
215	Reinforced Conc Abutment	Sev.	2	(EA)	50%	50%	0%	0%	0%	
220	Reinforced Conc Submerged Pile Cap/Footing	Sev.	9	(EA)	100%	0%	0%	0%	0%	
221	Submerged Concrete Spread Footing	Sev.	2	(EA)	100%	0%	0%	0%	0%	
223	Submerged, Conc Footing Seal	Sev.	2	(EA)	100%	0%	0%	0%	0%	
234	Reinforced Conc Cap	Sev.	12	(EA)	80%	15%	5%	0%	0%	
304	Open Expansion Joint	Sev.	70.00ft	(LF)	50%	50%	0%	0%	0%	
305	Polychlorophrene Joint	Sev.	2552.00ft	(LF)	10%	20%	70%	0%	0%	
309	Other Joint	Sev.	3700.00ft	(LF)	100%	0%	0%	0%	0%	
310	Elastomeric Bearing	Sev.	8	(EA)	100%	0%	0%	0%	0%	
311	Moveable Bearing (roller, sliding, etc.)	Sev.	290	(EA)	35%	60%	5%	0%	0%	
313	Fixed Bearing	Sev.	4	(EA)	50%	50%	0%	0%	0%	
321	Reinforced Conc Approach Slab w/ or w/o AC Ovly	Sev.	2	(EA)	50%	50%	0%	0%	0%	
325	Traffic Impact Condition	Ben.	1	(EA)	0%	100%	0%	0%	0%	
326	Deck Wearing Surface	Ben.	1	(EA)	0%	100%	0%	0%	0%	
331	Reinforced Conc Bridge Railing	Sev.	7044.00ft	(LF)	90%	10%	0%	0%	0%	
334	Metal Bridge Railing - Coated	Sev.	1708.00ft	(LF)	70%	30%	0%	0%	0%	
357	Pack Rust	Sev.	1	(EA)	0%	0%	100%	0%	0%	
359	Soffit of Concrete Deck or Slab	Sev.	1	(EA)	38%	30%	30%	2%	0%	
363	Section Loss	Sev.	1	(EA)	100%	0%	0%	0%	0%	

Figure 4.1.3 Oregon Bridge Inspection Report with Element Level Data

390	Paint, Alkyd (incl red lead)	Sev.	3713.50sqft	(SF)	35%	60%	5%	0%	0%
990	Miscellaneous Items	Sev.	1	(EA)	100%	0%	0%	0%	0%
994	Miscellaneous Fender Sys Timber	Sev.	2	(EA)	0%	100%	0%	0%	0%

Appraisal

Appraisal	NBI #	Rating
Scour	113	5 Stable w/in footing
Bridge Rail	36A	0 Substandard
Transitions	36B	0 Substandard
Approach Rail	36C	0 Substandard
Rail Ends	36D	0 Substandard
Structural	67	5 Above Min Tolerable
Deck	68	3 Intolerable - Correct
Clearance	69	N Not applicable (NBI)
Waterway	71	9 Above Desirable
Approach Alignment	72	8 Equal Desirable Crit

NBI Category

Category	NBI #	Rating
Deck Condition	58	6 Satisfactory
Superstructure	59	5 Fair
Substructure	60	6 Satisfactory
Channel	61	7 Minor Damage
Culvert/Retaining Walls	62	N N/A (NBI)

Remarks

P Conc Deck/Thin Ovl (18)
(6/09) Thin overlay overtops one of the joints.

R/Conc Open Girder (110)
Bt 5 girder 1 has exposed stirrups (6/09)

P/Stl Thru Truss/Bot (121)
(6/9) Lots of garbage/materials (PVC) on steel joints below the deck.

P/Stl Thru Truss/Top (126)
(6/09) Missing rivets in SE spire at start of thru truss.

R/Conc Arch (144)
CONCRETE ARCH'S HAVE HORIZONTAL CRACKS - NEAR THE CENTER TOP... (6/09) Steel exposed in spandrel column as well as cracks with efflor. on arches. Cathodic Protection project underway @ South approach spans

R/Conc Floor Beam (155)
SOME OF THE CAPS, COLUMNS, HAVE CRACKS, SPALLS & EXPOSED REBAR

R/Conc Pier Wall (210)
(6/09) Bt 7 pier wall, S. side, has corrosion cracking @ bottom of columns and delamination.

R/Conc Cap (234)
MOST OF THE CAPS NEED WASHED... (6/09) Bent 7 cap has spalling w/exposed stirrup near column 2.

Open Expansion Joint (304)
MANY OF THE JOINTS EDGES ARE SPALLING. ....ALL JOINTS ARE LEAKING ....JOINT AT MIDSPAN HAS FAILED - PERCALL CORNER FAILED -

Other Joint (309)
[none]

Moveable Bearing (311)
(6/09) Verify total quantity of bearings after completion of cathodic protection.

Conc Bridge Railing (331)
Concrete rail being replaced in south approach spans (6/09)

Misc (990)
(6/09) Earthquake retrofit on S. end, bent 3, cables are tight

Fender System (994)
UW report states rating for elem. 994 as CS1-95%, CS2-3%, and CS3-2%

Notes

Inspection Notes

Reviewed for Item #113, stays a T, jrw, user #152, 09-02-08. Tidal hydraulics study needed to determine seriousness and extent of possible scour during the flood of maximum scour potential. Tidal hydraulics study done by West Consultants, changed item 113 from T to 5, 01-11-11, jrw.

Figure 4.1.3 Oregon Bridge Inspection Report with Element Level Data (cont.)

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

N19-Detour Length (miles):	20	
N20-Toll:	3	
N28-Lanes On / Under:	2 / 0	
		2nd Record
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

N38-Navigation Control:	0
N39-Nav Vert Clr (feet):	0
N40-Nav Horiz Clr (feet):	0
N111-Nav Pier/Abut Prot:	
N116-Nav Min Vert Clr (feet):	0

GENERAL DATA

N33-Bridge Median:	0
N34-Skew:	0
N35-Structure Flared:	0
N37-Historical Significance:	5
N107-Deck Str Type:	1
N108-Wear Surf Prot System:	6 0 0
A201-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 4.1.4 Arizona Structural Inventory and Appraisal Sheet

REPORT ID: INVT001A

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
COMPREHENSIVE INVENTORY DATA REPORT

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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 4.1.5 Florida Structural Inventory and Appraisal Sheet

REPORT ID: INVT001A

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
COMPREHENSIVE INVENTORY DATA REPORT

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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 4.1.5 Florida Structural Inventory and Appraisal Sheet (Continued)

REPORT ID: INVT001A

FLORIDA DEPARTMENT OF TRANSPORTATION
BRIDGE MANAGEMENT SYSTEM
COMPREHENSIVE INVENTORY DATA REPORT

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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 4.1.5 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

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 4.1.5 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 (see Figures 4.1.6 and 4.1.7) or wearable computers (see Figures 4.1.8 and 4.1.9).

Figure 4.1.6 Portable Computer

Figure 4.1.7 Inspector Using Portable Computer

Figure 4.1.8 Wearable Computer with Case

Figure 4.1.9 Inspector Using Wearable Computer

The data and information required of states by the FHWA is listed in the *FHWA Coding Guide* and AASHTO *Manual for Bridge Evaluation*. It is important to note that several items listed in the *FHWA Coding Guide* apply to both the field and office personnel responsible for bridge inspections. The bridge inspector is typically 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 geometric and other inventory items will remain unchanged. The inspector is responsible for spot checking to see if inventoried items are consistent with observations at 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 include the following SI&A items:

- 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. This item is subject to change as conditions change and is therefore not viewed as a "permanent" item.
- Proposed Improvements – Items for work proposed and estimated costs for all bridges eligible for funding from the Highway Bridge Program.
- 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 *FHWA Coding Guide*. Although inventory items are usually provided from previous reports, the inspector is responsible for verifying and updating the inventory data as needed. See Topic 4.2 for condition and appraisal rating items.

4.1.4 Condition and Appraisal Rating Items

Condition Rating Items

Condition ratings are used to describe the existing, in-place bridge as compared to the as-built condition. Condition ratings are typically coded by the inspector. Condition rating items include:

- Deck – Describes the overall condition rating of the deck. This condition of the surface/protective systems, joints, expansion devices, curbs, sidewalks, parapets, fascias, bridge rail and scuppers is not included in the rating, but the condition will be noted in the inspection form. Decks that are integral with the superstructure will be rated as a deck only and not influence the superstructure rating.
- Superstructure – Describes the physical condition of all the structural members. The condition of the bearings, joints, paint system, etc. will not be included in the rating except for extreme situations, but the condition will be noted in the inspection form. Superstructures that are integral with the deck will be rated as a superstructure only and not influence the deck rating.
- Substructure – Describes the physical condition of piers, abutments, piles, fenders, footings or other components.
- Channel and channel protection – Describes the physical condition that is associated with the flow of the water through the bridge which include the stream stability and the condition of the hydraulic countermeasures.
- Culvert – Evaluates the alignment, settlement, joints, structural condition, scour and any other of the items that may be associated with a culvert.

Appraisal Rating Items

Condition ratings are a judgment of a bridge component condition in comparison to current standards. Appraisal items are used to evaluate a bridge in relation to the level of service which it provides on the highway system of which it is a part. The structure will be compared to a new one which is built to current standards for that particular type of road. Appraisal rating items include:

- Structural Evaluation – Overall evaluation of the structure based on the lowest bridge component condition rating, excluding the deck, superstructure, substructure, channel and channel protection and culverts. This item is calculated by the FHWA Edit/Update program.
- Deck Geometry – Evaluates the curb-to-curb bridge roadway width and the minimum vertical clearance over the bridge roadway. This item is calculated by the FHWA Edit/Update program.
- Under-clearances, Vertical and Horizontal – The vertical and horizontal under-clearances from the through roadway under the structure to the superstructure or substructure units. This item is calculated by the FHWA Edit/Update program.
- 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 classify 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

Chapter 4

Bridge Inspection Reporting

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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. 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.

4.2.2 Condition Rating Items

Deck, Superstructure and Substructure

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, and substructure. 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 is responsible for evaluating each element of each component and assigning to it a descriptive condition rating of "good," "fair," or "poor," based on the physical deficiencies found on the individual element. The following guidelines are 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 is responsible for recording the location, type, size, quantity, and severity of deterioration and deficiencies for each element of a given component.

Evaluating Components

The following major components of bridges receive an overall Structure Inventory and Appraisal (SI&A) component condition rating:

- Item No. 58 – Deck
- Item No. 59 – Superstructure
- Item No. 60 – Substructure

Component Condition Rating Guidelines

NBI component condition ratings for deck, superstructure, or substructure components, in general, should reflect the overall condition of the component rather than localized conditions. This has been true for many years and is emphasized in the FHWA *Coding Guide* with the following wording:

> *Condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated. Conversely, they are improperly used if they attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition code must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated.*

Although the *FHWA Coding Guide* states that it is improper to use the condition codes to describe localized instances of deterioration or disrepair, it also states that the inspector must consider both the severity and extent of the deterioration. With this in mind, there are occasions when a severe, localized condition affects the structural capacity of a component member. It is important to recognize that the coding applies to all primary members of a component. Therefore, localized conditions that impact the structural capacity of just one member can impact the overall performance of the entire component. The affect on structural capacity is dependent upon several factors including the type and extent of the deterioration, as well as the location along the member. An inspector may need to discuss the observed condition with an engineer to make this determination. When these situations occur, it is appropriate to assign a lower component condition rating for that component from a safety perspective and is in keeping with the intent of the National Bridge Inspection Program.

When these localized conditions are determined to be such that prompt action is needed and/or the overall component condition rating is affected, the conditions should also be addressed through the "critical findings" process that is identified in the NBIS regulation. The NBI component condition rating should be reviewed and appropriately adjusted once the critical finding has been addressed. This adjustment will depend on how the critical finding was addressed and how that action relates to the original rating rationale.

The coding of NBI condition items should be viewed as important, but secondary, to the recognition of and follow-up on critical findings.

Currently, states employ two approaches to coding condition items when localized areas of severe deterioration are encountered. Some will account for the severity of a localized area of deterioration by lowering the condition rating of an entire component. The component condition rating is adjusted after the deteriorated area is improved (i.e., rating may rise if physical improvements are made, or may stay the same if the bridge is posted for load restrictions and/or supported with temporary shoring). FHWA recognizes this approach when the severity of the localized deterioration affects the structural capacity of the component.

Other states “rate to the average” regardless of the severity of a localized area of deterioration. This approach relies heavily on ensuring that critical findings are addressed in a timely manner regardless of the component condition rating value. If the localized area of severe deterioration is not improved following the critical finding follow-up process, the component condition rating may need to be lowered to account for the severity of the deterioration if structural capacity is affected.

Either approach to coding the condition items results in the same ultimate outcome, i.e. critical inspection findings are addressed to ensure continued safe use of the bridge and component condition ratings <u>eventually</u> reflect the overall condition of the component. If the approach is to consider both the severity and extent of a component's deterioration in rating each component at the time of inspection (or up to 90 days after the inspection as required by the NBIS), there cannot be any assumptions about future improvements made to a localized area. Only if an improvement is made, the rating should then be raised as appropriate. If the improvement is made within 90 days of the inspection, there is no need to consider the localized deterioration in the rating.

The following general component condition rating guidelines (obtained from the 1995 edition of the *FHWA Coding Guide*) are to be used in the evaluation of the deck (Item 58), superstructure (Item 59), and substructure (Item 60):

<u>Code</u>	<u>Description</u>
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 component condition rating guidelines presented above are general in nature and can be applied to all bridge components and material types.

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.

A bridge's load-carrying capacity is not to influence component condition ratings. The fact that a bridge was designed for less than current legal loads, and may even be posted, has no influence upon component condition ratings.

Component condition ratings are determined by applying condition descriptions, which are general in nature, covering a broad array of bridge components and material types. The inspector is responsible for being familiar with terminology concerning material types and associated deficiency to utilize condition descriptions for accurately assigning component condition ratings. The following illustrates several common deficiency 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

Establishing a link between material type and deficiency allows for accurate component condition ratings determined by utilizing condition descriptions for ratings 9 through 1 found in the general component condition rating guidelines.

Supplemental component condition rating guidelines, which may be developed by individual states, are intended to be used in addition to the *FHWA 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 *FHWA Coding Guide*) helps to clarify how each type of deficiency affects the component condition rating. Care has to be taken not to "pigeonhole" the rating based on only one word or phrase. The following is one suggested method for determining proper component 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 generally works with all of the component condition rating guidelines.

4.2.3 Channel and Channel Protection Condition Ratings

General

For structures located over waterways, a Structure Inventory and Appraisal (SI&A) condition rating is provided for the channel and channel protection:

- Item No. 61 – Channel and Channel Protection

Overall Condition

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 component condition rating of the superstructure and substructure.

Evaluate and code the condition in accordance with the previously described general component condition ratings, procedures to account for critical findings, 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 deficiency. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor deficiency. 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 deficiency. Trees and brush restrict the channel.
4	Bank and embankment protection is severely undermined. River control devices have severe deficiency. 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.

4.2.4 Culvert Condition Ratings

General

When assigning a culvert condition rating, all areas of the culvert and the possible effects on the overall structure are investigated. The inspector considers whether the component is functioning properly, whether it could pose a threat to safety or cause property damage, and whether it could cause more extensive damage if not repaired.

Evaluating Elements

Chapter 14 addresses the individual elements of various culverts. The overall component condition rating considers all of the elements which make up a culvert and are useful in establishing maintenance, rehabilitation, and replacement programs and priorities.

Although some of the individual elements of culverts are not directly considered in the *FHWA Coding Guide*, these supplemental items are useful in determining the overall culvert condition ratings. They may also be included as part of an agency's bridge management system.

Evaluating Components

In addition to the major components of bridges (deck, superstructure, and substructure), culverts also receive a Structure Inventory and Appraisal (SI&A) overall component condition rating:

- Item No. 62 – Culverts

Component Condition Rating Guidelines

This item evaluates the alignment, settlement, joints, structural condition, scour, and other items associated with culverts. The component condition rating code is intended to be an overall condition evaluation of the culvert. Integral wingwalls to the first construction or expansion joint are included in the evaluation.

Item 58 – Deck, Item 59 – Superstructure, and Item 60 – Substructure should be coded N for all culverts.

Evaluate and code the culvert condition in accordance with the previously described general component condition ratings, procedures to account for critical findings 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.

4.2.5 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
- Item No. 36 – Safety Features
- Item No. 113 – Scour Critical Bridges

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 is 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 roadway approaches to the bridge compared to the general highway.

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 are evaluated using identical standards. Therefore, tables and charts have been developed which are 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 *FHWA Coding Guide*) are used to evaluate structural evaluation (Item 67), deck geometry (Item 68), underclearances (Item 69), waterway adequacy (Item 71) and approach roadway alignment (Item 72).

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 Item 67 - Structural Evaluation, Item 68 - Deck Geometry, Item 69 - Underclearances, Vertical and Horizontal, Item 71 - Waterway Adequacy and Item 72 - Approach Roadway Alignment appear in the *FHWA 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 *FHWA Coding Guide*. This item is coded by the FHWA Edit/Update program, not the inspector. 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.
- Appraisal codes of 3 or less can be achieved without the superstructure and substructure controlling with the 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. This item is coded by the FHWA Edit/Update program, not the inspector. The *FHWA 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 *FHWA 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. This item is coded by the FHWA Edit/Update program, not the inspector.

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 *FHWA 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. This item is not coded by the FHWA Edit/Update program.

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. This item is not coded by the FHWA Edit/Update program. 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 are 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 *FHWA 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 from 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 *FHWA Coding Guide.*

4.2.6 Functionally Obsolete and Structurally Deficient

Definitions

A bridge is considered to be functionally obsolete if it has deck geometry, load carrying capacity, clearance or approach roadway alignment that no longer meets the criteria for the system of which the bridge is a part. Examples include bridges with inadequate lane widths or shoulder widths, insufficient vertical clearances to serve the traffic demand, or bridges that may be occasionally flooded.

Bridges are considered structurally deficient where significant load carrying elements are found to be in poor or worse condition due to deterioration and/or damage, or the adequacy of the waterway opening provided by the bridge is determined to be extremely insufficient to the point of causing intolerable traffic interruptions.

Any bridge classified as structurally deficient is excluded from the functionally obsolete category. Bridges that are structurally deficient and functionally obsolete are reported together as deficient bridges.

General Qualifications

In order to be considered for either the structurally deficient or functionally obsolete classification, a highway bridge must meet the following:

Structurally Deficient (SD) -

1. A condition rating of 4 or less for
   - Item 58 - Deck; or
   - Item 59 - Superstructures; or
   - Item 60 - Substructures; or
   - Item 62 - Culvert and Retaining Walls.(1) or
2. An appraisal rating of 2 or less for
   - Item 67 - Structural Evaluation; or
   - Item 71 - Waterway Adequacy.(2)

Functionally Obsolete (FO) -

1. An appraisal rating of 3 or less for
   - Item 68 - Deck Geometry; or
   - Item 69 - Underclearances;(3) or
   - Item 72 - Approach Roadway Alignment. or
2. An appraisal rating of 3 for
   - Item 67 - Structural Evaluation; or
   - Item 71 - Waterway Adequacy.(2)

Footnotes for structurally deficient and functionally obsolete:

(1) Item 62 applies only if the last digit of Item 43 (Structure Type) is coded 19.
(2) Item 71 applies only if the last digit of Item 42 (Type of Service) is coded 0, 5, 6, 7, 8 or 9.
(3) Item 69 applies only if the last digit of Item 42 is coded 0, 1, 2, 4, 6, 7 or 8.

4.2.7 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 *FHWA Coding Guide*.

Sufficiency Rating Formula

$$S.R. = S_1 + S_2 + S_3 - S_4$$

$$0 \leq S.R. \leq 100$$

0 (entirely deficient) ≤ S.R. ≤ 100 (entirely sufficient)

where: S_1 = 55% max.; based on structural adequacy and safety (i.e., superstructure, substructure or culvert condition and load capacity).

S_2 = 30% max.; deals with serviceability and functional obsolescence (items such as deck condition, structural evaluation, deck geometry, underclearances, waterway adequacy, approach road alignment).

S_3 = 15% max.; concerns essentiality for public use (items such as detour length, average daily traffic, and STRAHNET (Strategic Highway Corridor Network).

S_4 = 13% max.; deals with special reductions based on detour length, traffic safety features, and structure type.

Twenty NBI 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. The sufficiency rating is calculated by the FHWA Edit/Update program.

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 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 have developed 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.

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Topic 4.3 Introduction to Element Level Evaluation

4.3.1 Introduction

Managers of large inventories of infrastructure assets need a tool to effectively manage these assets. For bridge data, element level inspection has been successfully used as a basis for data collection, performance measurement, resource allocation, and management decision support. Although component condition rating and reporting, as described in the FHWA *Coding Guide*, provides a consistent method for evaluation and reporting, the data is not comprehensive enough to support bridge preservation performance-based decision support.

The Pontis CoRe (Commonly Recognized) Element Report (June 1993), which is the basis of the AASHTO CoRe Element Guide, was prepared by technical working group representatives from California, Colorado, Minnesota, Oregon, Virginia, Washington, and the Federal Highway Administration. The Pontis CoRE Report explains the reasoning behind the selection of bridge items that require inspection for a successful Bridge Management System. Pontis is 'bridge' in Latin.

In 2010, the AASHTO Bridge Element Inspection Manual was developed to address improvements to the existing CoRe Element Guide. This reference manual was prepared by representatives from California, Idaho, Michigan, Montana, New York and FHWA to further enhance bridge management.

Significant changes from the existing CoRe Element Guide:

- All elements have four defined condition states having general descriptions (good, fair poor, and severe).
- Wearing surfaces have been separated from decks/slabs and protective coatings.
- Elements have been categorized as National Bridge Elements (NBEs) or Bridge Management Elements (BMEs), with provisions for custom agency developed elements.
- Multiple distress paths provide the ability to incorporate all defects within the overall element assessment.
- Smart Flags (Defect Flags) have been revised to identify the predominant distress.

4.3.2 Element Level Inspection Development

In developing a system for standardized data collection, the FHWA needed to look at the shortcomings of NBI (National Bridge Inventory) data. The problems with NBI data included:

- Each bridge is divided into only three major parts for condition assessment: deck, superstructure, substructure and culvert.
- The rating scale for these parts is 0-9 by severity of the deficiency, which does not indicate the extent of the deficiency.

- The component condition ratings are based on subjective interpretation by the inspectors.

A system was developed which included a standardized description of bridge elements at a greater level of detail. The FHWA created a task force to revise the standards and created a manual called "*Commonly Recognized (CoRe) Structural Elements*". The AASHTO Guide for CoRe Element Manual defined each element, the unit of measurement, definitions of a set of 3-5 standardized condition states, and feasible actions for each condition state. The CoRe Element Manual was accepted as an official AASHTO manual in May 1995. Some states developed their own CoRe Element Manual based on the AASHTO *Core Element Manual*. Approximately 40 states perform element level inspection.

In 2010, the limitations of the CoRe Element Manual were again addressed. These problems included:

- Inconsistent number of condition states and descriptions between element types
- Inconsistent condition state definitions between agencies
- Limited distress path language defined within the condition states

The National Bridge Element and Bridge Management Element system provides multiple distress paths for each defined condition state. This allows for deficiencies to be identified within each overall element assessment. The AASHTO *Guide Manual for Bridge Element Inspection* defines each element, description, unit of measurement or quantity calculation, set of four standardized condition states, feasibility actions, element commentary, and element definitions. The AASHTO *Guide Manual for Bridge Element Inspection, First Edition, 2011*, was first published as an official manual in February 2011.

4.3.3 Element Level Rating Terminology

The AASHTO *Guide Manual for Bridge Element Inspection, First Edition, 2011*. (see Figure 4.3.1) provides a description of structural elements that are commonly used in highway bridge construction and encountered on bridge safety inspections.

The following terms are used to describe bridge element-level inspection:

- National Bridge Elements (NBEs) represent the primary structural components of bridges necessary to determine the overall condition and safety of primary load carrying members. They provide a uniform basis for data collection.
- Bridge Management Elements (BMEs) represent a recommended set of condition assessment language that may be modified to suit the agency's needs. Examples of these elements include expansion joints and seals, approach slabs, wearing surfaces, protective coatings and smart flags.
- Agency developed elements are customized elements that can be sub-sets of defined NBEs, sub-sets of BMEs, or elements that are independent of the defined AASHTO elements. Agency developed elements are used in addition to the NBEs and BMEs.
- Condition states describe the severity of the deficiencies in AASHTO Bridge Elements. All elements have four defined condition states having general descriptions of good, fair, poor, and severe. Condition State 1 (good) and Condition State 4 (severe).
- Environments are used to classify the operating conditions and the deterioration of the structure, which does not change due to maintenance work or deficiencies. Depending on the agency, inspectors may or may not be responsible for determining the environment.
- Sub-elements or sub-sets are divisions of NBEs or BMEs that are created to provide flexibility to track variations in cost or performance characteristics.
- Smart Flags or Defect Flags are BMEs and used when a specific condition exists, which may be described in the National Bridge Element condition state definitions. They inherit the same units of measure as the NBE or BME to which they are assigned.
- Feasible actions, as provided in the AASHTO *Guide Manual for Bridge Element Inspection*, are general actions to address deficiencies. Feasible actions are often further defined by agencies for each condition state. Agency procedures vary and some inspectors create work recommendations for feasible actions. The inspector may not be required to record feasible actions.

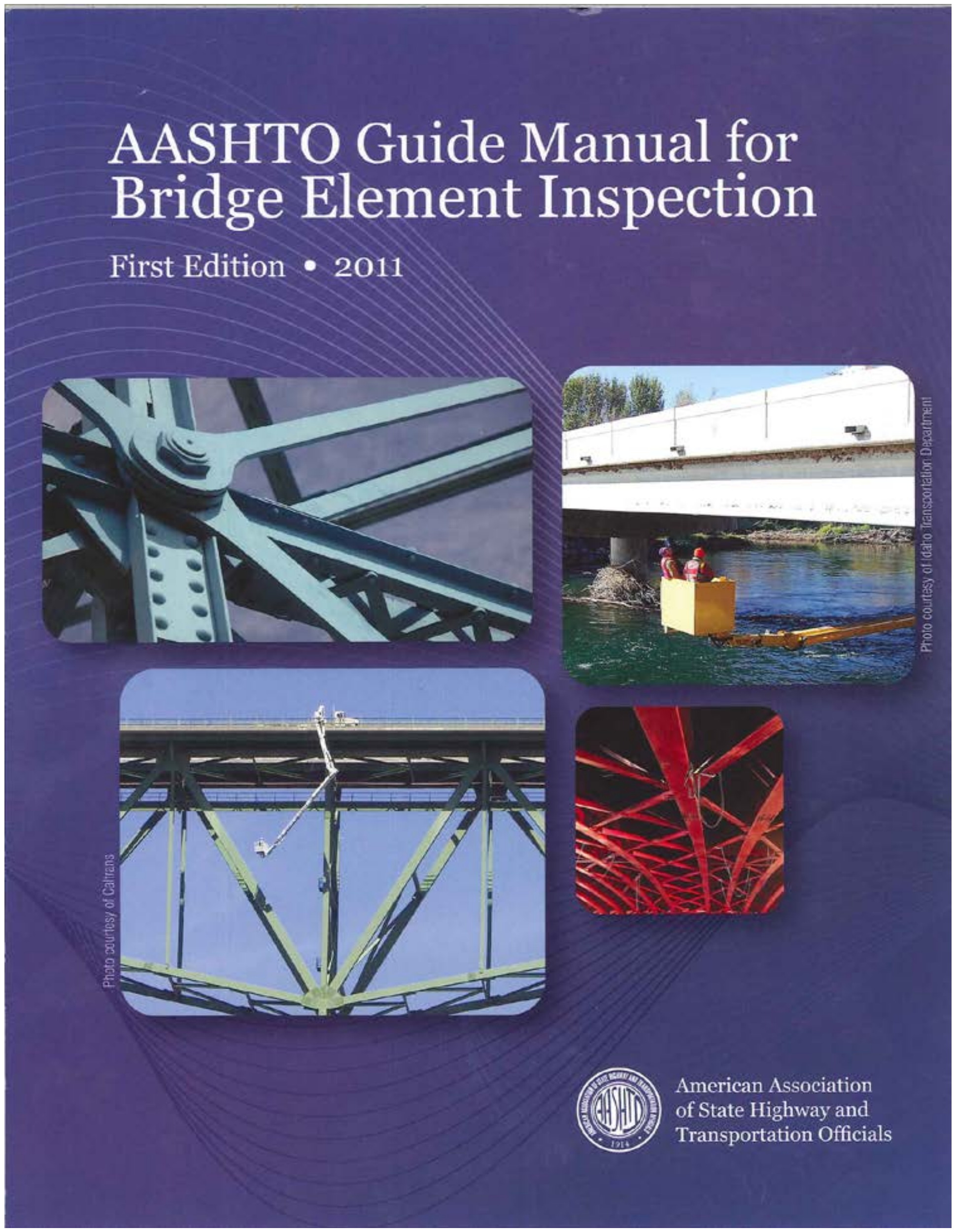

Figure 4.3.1 AASHTO *Guide Manual for Bridge Element Inspection*

4.3.4

Basic Requirements of National Bridge Elements

In the development of National Bridge Elements, it was important that the specification must be generic. Different agencies have varying maintenance practices, funding mechanisms, policy concerns and terminology. However, the physical components of bridges and deterioration processes are not unique. Agencies must be able to customize the generic standard to satisfy their own purposes without sacrificing the benefits of a common standard. Any changes to elements could introduce incompatibility between agencies. For this reason, agencies cannot change the number of condition states and the intent of the condition state language.

To avoid this from happening, the bridge element guide manual provides the ability of an agency to add custom agency developed elements or modify recommended

Bridge Management Elements. It is possible for future National Bridge Elements or Bridge Management Elements to be added. These elements must be permanent, have clear distinction and be defined as concisely as possible. The guidelines for developing National Bridge Elements include:

- Each element must be a primary load carrying element
- Each element must have a unique functional role.
- Distinguish elements that have significantly different maintenance requirements.
- Distinguish elements that are measured in different ways for costing or inspection.
- Distinguish elements whose conditions are described in different ways.
- Each element must be significant from the standpoint of maintenance cost or functionality. This is why, for example, secondary members are omitted from the list of National Bridge Elements. The level of detail in data collection would be too large relative to the effect of these elements on decision making.
- Deterioration behavior and maintenance alternatives for the element must be sufficiently understood. This is why, for example, composite materials such as fiber reinforced polymer are excluded from the list of National Bridge Elements.
- If an element is more significant than other elements, its behavior or condition description is complex, the element may be subdivided into smaller elements. An example of this type of element would be a pin and hanger assembly.
- A formal definition of each element must be developed to clarify thinking.

One primary use of definitions is to establish a useful inventory. In the field, each element must be clearly identified, measured and counted economically. It is also important to describe element attributes, such as size, material, condition and serviceability, quantitatively. The commonality aspect of National Bridge Elements depends on having definitions that are widely understood and are stable over time. One major factor contributing to definitions being widely understood is NHI's bridge inspection related training courses.

4.3.5 Bridge Element Identification

National Bridge Elements

AASHTO National Bridge Elements describe primary load carrying members, including:

- Girders
- Trusses
- Arches
- Cables
- Floorbeams
- Stringers

- Abutments
- Piers
- Pins and Hangers
- Culverts
- Bearings
- Railings
- Decks
- Slabs
- Gusset Plates
- Column/Piles
- Caps

See Figures 4.3.2 - 4.3.4 for a list of decks/slabs, superstructure, and substructure AASHTO National Bridge Elements.

Element	Units	Element Number (Decks)	Element Number (Slab)	Other
Reinforced Concrete Deck/Slab	AREA	12	38	
Prestressed/Reinforced Concrete Top Flange	AREA	15		
Steel Deck - Open Grid	AREA	28		
Steel Deck - Concrete Filled Grid	AREA	29		
Steel Deck - Corrugated/Orthotropic/Etc.	AREA	30		
Timber Deck/Slab	AREA	31	54	
Bridge Rail		Other		
Metal Bridge Railing	LENGTH			330
Reinforced Concrete Bridge Railing	LENGTH			331
Timber Bridge Railing	LENGTH			332
Other Bridge Railing	LENGTH			333
Masonry Bridge Railing	LENGTH			334

AREA = square feet (square meter)
LENGTH= feet (meters)

Figure 4.3.2 Decks/Slabs National Bridge Elements in the AASHTO *Guide Manual for Bridge Element Inspection*

Element	Units	Steel	Prestressed Concrete	Reinforced Concrete	Timber	Masonry	Other
Girder/Beam	LENGTH	107	109	110	111		
Closed Web/Box Girder	LENGTH	102	104	105			
Stringer	LENGTH	113	115	116	117		
Truss	LENGTH	120			135		
Arch	LENGTH	141	143	144	146	145	
Floor Beam	LENGTH	152	154	155	156		
Cable	EA	147, 148					
Gusset Plate	EA	162					
Pin and/or Pin and Hanger Assembly	EA	161					

LENGTH= feet (meters)
EA = Each

Figure 4.3.3 Superstructure National Bridge Elements in the AASHTO *Guide Manual for Bridge Element Inspection*

Element	Units	Steel	Prestressed Concrete	Reinforced Concrete	Timber	Masonry	Other
Column/Pile Extension	EA	202	204	205	206		
Column Tower (Trestle)	EA	207			208		
Submerged Pile	EA	225	226	227	228		
Pier Wall	LENGTH			210	212	213	211
Abutment	LENGTH	219		215	216	217	218
Pier Cap	LENGTH	231	233	234	235		
Pile Cap/Footing	EA			220			
Culvert	LENGTH	240		241	242	244	243
Bearings							
Elastomeric Bearing	EA						310
Moveable Bearing (roller, sliding, etc.)	EA						311
Enclosed/Concealed Bearing	EA						312
Fixed Bearing	EA						313
Pot Bearing	EA						314
Disk Bearing	EA						315

LENGTH= feet (meters)
EA = Each

Figure 4.3.4 Substructure National Bridge Elements in the AASHTO *Guide Manual for Bridge Element Inspection*

Bridge Management Elements

AASHTO Bridge Management Elements represent a recommended condition assessment language that can be modified to suit the agency's needs. The following types of elements are defined as Bridge Management Elements:

- Joints
- Approach Slabs
- Wearing Surfaces
- Protective Systems
- Smart Flags (Defect Flags)

See Figures 4.3.5 - 4.3.6 for a list of decks/slabs and wearing surfaces and protection systems AASHTO Bridge Management Elements.

Element	**Units**	**Element Number**
Joints		
Strip Seal Expansion Joint	LENGTH	300
Pourable Joint Seal	LENGTH	301
Compression Joint Seal	LENGTH	302
Assembly Joint/Seal (modular)	LENGTH	303
Open Expansion Joint	LENGTH	304
Assembly Joint w/o Seal	LENGTH	305
Approach Slabs		
P/S Concrete Approach Slab	AREA	320
Reinforced Concrete Approach Slab	AREA	321

AREA = square feet (square meter)

LENGTH= feet (meters)

EA = Each

Figure 4.3.5 Decks/Slabs Bridge Management Elements in the AASHTO *Guide Manual for Bridge Element Inspection*

Element	Units	Element Number
Protective Systems		
Wearing Surfaces	AREA	510
Steel Protective Coating	AREA	515
Deck/Slab Protection Systems	AREA	520
Concrete Protective Coating	AREA	521

AREA = square feet (square meter)

Figure 4.3.6 Wearing Surfaces and Protective Systems in the AASHTO *Guide Manual for Bridge Element Inspection*

Defect Flags

Defect Flags are part of the Bridge Management Elements and are used to identify the predominant defect for that condition state. The severity of the deficiency is captured by coding the appropriate Defect Flag condition state. The NBI translator uses AASHTO element-level data that includes defect flag data to determine NBI component condition ratings.

Defect Flags inherit the units of the parent NBE or BME.

Steel Cracking/Fatigue:	This flag shall be used with steel elements to identify the predominant defect in a given condition state that is not corrosion.
Pack Rust:	This flag shall be used in conjunction with steel elements connection defects (including shapes in contact in built-up members) of steel bridges that are already showing signs of rust packing between plates.
Concrete Cracking:	This flag shall be used with concrete elements to identify the predominate defect in a given condition state that is not spalling or delaminations.
Concrete Efflorescence:	This flag shall be used with concrete elements to identify the predominate defect in a given condition state that is not spalling or delaminations.
Settlement:	This flag shall be used with all substructure and culvert elements to identify the predominate defect in a given condition state that is not material deterioration. The use of the flag is to identify the severity of the settlement.
Scour:	This flag shall be used with all substructure and culvert elements to identify the predominate defect in a given condition state that is not material deterioration. The use of the flag is to identify the severity of the scour.
Superstructure Traffic Impact:	This flag shall identify all traffic collisions with the superstructure. Application of the flag is in relation to the impact on the structures capacity to carry load.
Steel Section Loss:	This flag shall be used with steel elements to identify the predominate defect in a given condition state that is not corrosion. Setting this flag will identify the severity of section loss.
Steel Out-of-plane Compression Members:	This flag shall be used with steel truss or arch elements. The use of the flag shall denote any member that is not in plane with the panel (buckling). It shall be used to identify the predominate defect in a given condition state that is not material deterioration.
Deck Traffic Impact:	This flag shall identify all traffic collisions with the deck. Application of the flag is in relation to the impact on the structures capacity to carry load.

Substructure Traffic Impact:	This flag shall identify all traffic collisions with the substructure. Application of the flag is in relation to the impact on the structures capacity to carry load.
Barrel Distortion:	This flag is to identify the severity of the culvert barrel distortion. Its use shall be with culverts only. This flag shall describe predominate culvert deterioration that is not attributed to material deterioration.

Agency Developed Elements

Agencies may develop sub-elements that use the same condition state definitions as their associated NBE or BME elements. This allows for more detailed element descriptions. They are a subset of the NBE or BME and allow a more detailed classification. They are often created to distinguish a different size, location or exposure.

- Fascia girders and interior girders can be examples of Sub-Elements.
- The ends of girders can be examples of Sub-Elements.

Agency developed elements fall into three main categories: subsets of NBEs, BMEs, or elements that are independent of defined elements. Agency Developed Element guidelines are listed below:

Agency Defined Subsets of NBEs

For agency defined sub-sets of National Bridge Elements, the agency must be able to combine the sub-elements back together to form the original NBE element for NBI submission with the original condition state and element definition language.

Agency Defined Subsets of BMEs

For agency defined sub-sets of Bridge Management Elements, the agency is not required to combine the elements to form the original Bridge Management Elements since BMEs are not required for NBI submission. However, custom elements of this type must retain the original number of condition states using a good, fair, poor, severe description.

Independent Agency Developed Elements

For Agency Defined Elements that are not sub-sets of National Bridge Elements or Bridge Management Elements, the only requirement is the standardized number of condition states (four). These elements may include inventory items or specific aspects of the structure. Independent Agency Defined Elements may or may not include feasible actions, deficiency, or official condition state language.

Examples of potential independent agency developed elements include approach guardrail, approach guardrail ends, seismic retrofit components, tunnels, condition of drainage components or lighting fixtures, or ancillary items such as overhead signing structures.

4.3.6 Condition States

The scale of good-fair-poor-severe is not acceptable because these terms do not have precise definitions that can be observed in the field. It was decided to measure bridge condition on a single scale that reflects common processes for deterioration and the effect on serviceability. The general pattern for a Bridge Element having four condition status is as follows:

1. Good – No deterioration to minor deterioration
2. Fair – Minor to Moderate deterioration
3. Poor – Moderate to Severe deterioration
4. Severe – Beyond the limits established in condition state 3 and/or warrants a structural review to determine strength or serviceability of the element or bridge

Each of these levels of deterioration is called a condition state. The condition state methodology provides two types of information about a bridge element's deterioration:

- Severity – characterized by precise definition of each condition state
- Extent – the distribution of the total element quantity among condition states

The severity is important for selection of a feasible and cost effective preservation treatment, and extent is important for cost estimation.

Assignment of quantities to condition states is determined from element definitions and element commentary for National Bridge Elements. Condition state definitions are guidelines to the bridge inspector for categorization of the severity of the deficiency. Element commentary represents additional considerations for the inspector during the collection of data. From this information, the inspector can complete the element level evaluation. Additionally, element level Smart Flags (Defect Flags) are used to describe a condition which is not included in the National Bridge Element or Bridge Management Element condition state language.

4.3.7 Feasible Actions

Feasible actions are those that an agency may take to remove the defect. They represent a set of responses that may be taken for an element based upon quantities within a given condition state. They also represent general guidance on agency preservation strategies and can be customized by each agency for each element and condition state.

A summary of feasible actions and associated condition states is given below. Depending on the element, some feasible actions/conditions states may not be available. Other feasible actions, such as "Do Nothing", are available for all elements and condition states. "Do Nothing" can be used for all the elements in condition states since the possibility of nothing that needs to be done due to the condition of the element being good or to be used if the condition of the bridge is so severe, the bridge is closed and or there is a feasible action already taking place.

Feasible Action	Condition State 1	Condition State 2	Condition State 3	Condition State 4
Do Nothing	●	●	●	●
Protect	●	●	●	●
Preserve (for other culverts and other railings)	●	●		
Repair		●	●	●
Rehab			●	●
Reset (for bearings only)			●	●
Replace			●	●

4.3.8 Environments

Element can exist in one of four environments, which describe different weather or operating conditions. The environments are important for deterioration models and prediction of future conditions. The four environments are defined in general terms as follows:

1. Benign – No environmental or operational conditions affecting deterioration
2. Low – Environmental or operational conditions create no adverse impacts, or are mitigated by past non-maintenance actions or highly effective protective systems
3. Moderate – Typical level of environmental or operational conditions influence on deterioration
4. Severe – Environmental or operational conditions factors contribute to more rapid deterioration. Protective systems are not in place or are ineffective

Environment policies are used for element level inspection and set by individual state agencies.

4.3.9 The Role of Element Level Data in Bridge Management Systems

An immediate application of Bridge Elements is the collection and analysis of performance data. It is essential that original data collection be as objective and repeatable as possible. This raw, objective data must be stored so that the analysis may be updated or improved at a later time. Bridge Elements must be usable to support management decision making. The large volume of raw data collected must be transformed into useful information. For this reason, the development of bridge Bridge Elements was heavily influenced by the parallel development of Pontis software and previous CoRe elements.

Condition state data provides quantitative data about the physical condition and performance of bridge elements. This data is also, the effects of treatment actions can be tracked over time. Element level data is an essential part of the following BMS functions. Element level inspections can track the effectiveness of action over time by showing the various condition states and how they may change over time after the bridge element is either repaired, replaced, or nothing would be done. Potential applications for agencies includes:

- Identification of bridge needs (replacement and preservation)
- Development and testing of new maintenance techniques
- Treatment selection policies
- Project priority setting and programming
- Budgeting
- Funding allocation
- Long-range planning

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Table of Contents

Chapter 4

Bridge Inspection Reporting

Abbreviations for Field Inspection Notes

Abut. = Abutment
Adj. = Adjacent
B. = Bent
Btw. = 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
Exp. = 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
Lgth. = 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 4.4 Record Keeping and Documentation

4.4.1 Introduction

Bridge owners maintain a complete, accurate, and current record of each bridge under their jurisdiction. Such information relating directly to the inspection, design, performance and maintenance of the bridge is vital to the effective management of a population of bridges. Additionally, this information provides a record that may be important for repair, rehabilitation, or replacement of their assets.

The first section in this topic covers the critical components of the bridge record, while the remaining sections provide the inspector with guidance on how to thoroughly organize inspection data and produce an accurate and effective inspection report.

4.4.2 Bridge Records

Bridge records, or files, are used to maintain detailed, cumulative and up-to-date information on each 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.

The contents of any particular bridge 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. The bridge file is not only a resource to the bridge owner, but also a resource to the inspector. The inspector will gain valuable insight into the bridge by being familiarized with it prior to the inspection. It is recommended that the following types of information be assembled when possible.

According to the AASHTO *Manual for Bridge Evaluation*, the bridge record includes the following information:

- Plans, including construction plans, shop and working drawings, and “as-built” drawings
- Specifications
- Correspondence
- Photographs
- Materials and tests, including material certification, material test data, and load test data
- Maintenance and repair history
- Coating history
- Accident records
- Posting
- Permit loads
- Flood and scour data
- Traffic data
- Inspection history
- Inspection requirements

- Structure Inventory and Appraisal sheets
- Inventories and inspections
- Rating records

Plans

Construction, "as-built," or shop and working plans are included in a bridge record. If plans are not available, determine the following types of construction information: date built; type of structure, including size, shape, and material; design capacity; and design service life. Hydraulic data is also assembled where available, including structure profile gradeline, 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.

Specifications

The bridge record includes a complete copy of the technical specifications used to design and build the bridge. When a general specification was used, only the special provisions are included in the file. The edition and date of the general specifications are noted in the bridge record.

Correspondence

The bridge record includes any applicable letters, memorandums, and notices of project completion, construction diaries, telephone logs, and any other information directly concerning the bridge in chronological order.

Photographs

Photographs are used to supplement the inspection notes and sketches. A minimum of two photographs are included in the bridge record: a topside view of the bridge roadway and at least one elevation view of the bridge. Photographs showing major deficiencies or other features, such as utility attachments or channel alignment, also are included. Photographs that show load posting signs are also provided, if applicable.

Photo Log

Keep a photo log during the inspection. The photo log includes the date, photo number, and description of each photograph. It is best to be very specific when describing the photos (see Figure 4.4.1). Descriptions include both the location of the member and a brief description of any deficiencies.

[redacted]

PHOTO LOG FORM

INSPECTORS GRL (WB) **PHOTO LOG NO.**
BRIDGE NO. MP-056-0064-B00293 **DATE:** 7·9·11 (Photos 1-11) 7·10·11
BRIDGE NAME 7th Street to 13th Street Unit

PHOTO #	LOCATION	DESCRIPTION
1	Pier 46WB	General View
	from parking level	Sta Ahd
		at end of Ramp (left) beg. at
		Pier 47WB
2	Pier 49WB	Crack at end weld
		at Girder conn (S9 - Span 48WB)
3-7	Pier 49W	Gen View of end weld crack
	Span 49W	w/ close-ups.
8	North side	Gen View of Fascia Girder (S19)
	Pier 50WB	w/ full length long. stiffeners
9	Pier 55WB	torn strip seal over cross
		girder and pin & hanger assembly
10	Pier 55WB	Top View of Deck
	(north side)	Looking Bk Stations
11	Pier 55WB	Pin & Hanger Assembly
		w/ Bot support
		(Looking North)
12	Span 48WB	2nd Diaph (Cross Bracing) Top
	Girder S5	Conn & endweld cracking
		and toe weld cracking on Girder web
		to Flg (Previously reported - no change)
		opposite end of bay similar condition

Figure 4.4.1 Sample Photo Log

Materials and Tests

Certificates for the type, grade, and quality of materials used in construction of the bridge are included in the bridge record. Examples include steel mill certificates, concrete delivery slips, and any other manufacturers' certificates. The certificates are retained in accordance with bridge owner policy and statute of limitations.

Reports for any non-destructive or laboratory testing either during or after construction are included. If any field load testing is performed, provide the reports in the bridge record.

Maintenance and Repair History

Information about repairs and rehabilitation activities are included in the bridge record. This chronological record includes details such as the date, project description, contractor, cost, contract number and any other related data. The types and amount of repairs performed at a bridge or culvert site can be extremely useful. For example, 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.

Coating History

This information in the bridge record documents the surface protective coatings used, including surface preparation, application method, dry film paint thickness, types of paint, concrete and timber sealants, and other protective membranes.

Accident Records

Include details of accidents or damage to the bridge in the bridge record (see Figure 4.4.2). This information includes the date of the occurrence, description of the accident, member damage and repairs, and any investigative reports.

Figure 4.4.2 Accident Involving Construction Equipment and a Bridge

Posting

Each bridge record includes load capacity calculations and any required posting arising from the load ratings. The summary of posting actions includes the date of posting and a description of the signing used (see Figure 4.4.3).

Figure 4.4.3 Posted Bridge

Permit Loads

A record of the most significant single-trip permit loads using the bridge are included in the bridge record. This information is to include any applicable documentation and calculations.

Flood and Scour Data

A chronological history of major flooding events are included for bridges over water (see Figure 4.4.4). This history includes the high water marks at the bridge site, scour evaluation, scour history, and any plan of action.

Figure 4.4.4 Flood Event

Traffic Data

When available, the bridge record contains a history of the variations in Average Daily Traffic (ADT) and Average Daily Truck Traffic (ADTT) including the frequency and types of vehicles using the bridge. ADT and ADTT are important factors in determining fatigue life and are monitored for each bridge and each traffic lane on the bridge. If available, weights of the vehicles using the bridge are also included in the bridge record.

Inspection History

Reports 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 chronological record of inspections performed on the bridge includes the date and type of inspection. The initial inspection report is included in the bridge record. Earthquake data, fracture critical member information, deck evaluations, and corrosion studies are also included when available.

Inspection Requirements

Inspections are planned and prepared for by taking into account needed access, inspection equipment, structural details, inspection methods, and the required qualifications of inspection personnel. In addition, the National Bridge Inspection Standards require that written inspection procedures for specific types of more complex inspections (fracture critical, underwater, and complex bridges) be developed to address those items that need to be communicated to an inspection team leader to ensure a successful bridge inspection. Section 4 of the AASHTO *MBE* has general considerations regarding inspection plans. An owner may have general overall inspection procedures in their bridge inspection manual that address common aspects of these more complex inspections, however, each bridge will have written inspection procedures specific to each bridge which address items unique to each bridge. The following items are to be addressed for each of these types of bridge inspections, either in the bridge specific inspection procedures, or by referring to general inspection procedures (typically in an agency's bridge inspection manual):

- Identify each of the critical members to be inspected (fracture critical elements, past repairs, underwater elements, complex features, fatigue prone details, scour countermeasures, etc.) on plan sheets, drawings or sketches
- Identify special access needs or equipment necessary to gain the access required to inspect the features (under bridge inspection trucks, man lifts, traveler system, climbing, etc.)
- Describe the inspection method(s) and frequency to be used for the elements. For example, “Visually inspect all identified FCMs at arm’s length for cracks, deterioration, missing bolts, loose connections, broken welds… using PT to verify the existence of suspected cracks.”
- Address required proximity to details, such as “arm’s length”
- Identify special qualifications required of inspection personnel by the program manager, if any (successfully passed fracture critical course, certified electrician for movable bridge electrical components, qualified bridge inspection diver, etc., may be possible qualifications)

Other items that may be addressed depending on each unique situation might include:

- Special contacting procedures prior to inspection (Coast Guard, security, operations personnel, etc.)
- Safety concerns (snakes, bats, etc.)
- Best time of year to inspect the bridge (lake draw down, canal dry time, snow, ice, bird nesting seasons, etc.)
- Anything else the program manager wants the inspection team leader to be aware of in preparation for the inspection

Any special requirements to ensure inspector and public safety, including a traffic management plan, are also included.

Structure Inventory and Appraisal Sheets

A chronological record of SI&A forms used by the bridge owner is included in the bridge record. Refer to Topic 4.1 for a complete description of SI&A sample form.

Inventories and Inspections

Inspection reports are included as part of the bridge record. This information includes the results of all inventories and bridge inspections and can include construction or repair activities.

Bridge Inspection Forms

Many bridge owners 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 of standardized forms is that it organizes bridge reports into a consistent format (see Figures 4.4.5 and 4.4.24 that are located at the end of this topic).

ODOT Bridge Inspection Report Form

Bridge Number ________________ Date____________________

Bridge Name __________________ Inspector 1: _______________

Highway / Route _______________ Inspector 2: _______________

Milepost ____________________

List of Elements	Associated Smart (Defect) Flag	Associated Protection Systems	Quantity	Units	CS 1	CS 2	CS 3	CS 4

Figure 4.4.5 Element Level Example Inspection Form

Rating Records

A complete record of the determination of the bridge's load-carrying capacity is included in the bridge record (see Figure 4.4.6). This information will include the design load to indicate the live load the bridge was designed for, the analysis methods used to determine the inventory and operating ratings, and the inventory and operating ratings for the bridge. The capacity calculations will be signed and dated by the individual who determined them, together with any assumptions used.

Nebraska Department of Roads - Bridge Division

Load Rating Summary Sheet

State Bridge Number	C008500335	Analyst	
County Bridge Number	T4N R4W SEC 8 L	Analysis Date	3/8/2007
Structure Type	Concrete continuous -- Slab	Year Built	1998
Highway System	Not on National Highway System	Year Reconstructed	
		Design Load	HS20

NBI Rating Factor Summary (HS or HL93):
Inventory Capacity 1.11 Operating Capacity 1.85

Legal Truck Summary:
Type 3 (Tons) 61 Type 3S2 (Tons) 67 Type 3-3 (Tons) 101

Recommended Posting Summary:
Type 3 (Tons/NA) Type 3S2 (Tons/NA) Type 3-3 (Tons/NA)
Posting is required for capacities less than 25T, 37T, and 43T respectively. Gross Posting should be avoided.

Permit Load Summary:
Type 3 (Tons) 79 Type 3S2 (Tons) 87 Type 3-3 (Tons) 132
For permitting purposes only, capacity based on a single lane distribution factor with no impact.
No other vehicles are to be allowed on the bridge, crawl speeds less than 5 mph, and no gear shifting or braking, are to be strictly observed

Rating Method: ☐ ASR ☒ LFR ☐ LRFR ☐ Other

Rating Information Provided: ☒ Plans ☐ Field Measurements ☐ Testing ☐ No Information Exists

Depth & Type of Overlay: 1 in. ☒ Concrete ☐ Gravel ☐ Asphalt ☐ Other

Condition Rating:
Deck: 9 Superstructure: 9 Substructure: 9 Pile: 9 Scour: 8

Load Rating Evaluation Summary: I = Investigated C = Controls (HS or HL93)

I	C		I	C	
☒	☒	+ M of Interior Girder / Beam	☐	☐	Truss Members
☒	☐	+ M of Exterior Girder / Beam	☐	☐	Floor Beams
☒	☐	- M of Interior Girder / Beam	☐	☐	Stringers
☒	☐	- M of Exterior Girder / Beam	☐	☐	Pins
☐	☐	Shear At/Near Reactions	☐	☐	Hangers
☐	☐	Deck Overhang	☐	☐	Substructure Elements
☐	☐	Deck Between Girders	☐	☐	Sidewalks/Medians w/o Traffic Barriers
☐	☐	Fatigue Prone Details	☐	☐	Scour

Additional Comments (Include any section loss, location of section loss, assumptions, and hand calculation references used in this analysis)

(Seal & Date)

The recommended Rating and Posting for this structure are based on a theoretical analysis of the structural elements involved, and on a limited amount of information concerning their condition. These weight limits are intended only as a general guideline and may be varied accordingly by the officials responsible for this structure after an investigation of the structural condition, reaction to vehicular loads and any other items where judgement is required to establish a proper weight limit.

DR Form 464, Jan 07

Figure 4.4.6 Example Load Rating Summary Sheet

Post or restrict the bridge in accordance with the AASHTO *Manual for Bridge Evaluation* or in accordance with State law, when the maximum unrestricted legal loads or State routine permit loads exceed that allowed under the operating rating or equivalent rating factor.

4.4.3 Methods of Inspection Documentation

Traditional

Note all signs of distress and deterioration 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 4.4.7). This method is extremely flexible in that the inspector can draw whatever configurations are necessary to best describe and document deficiencies.

Figure 4.4.7 Inspector Taking Notes

Electronic Data Collection

Another method of record keeping is electronic data collection (see Figure 4.4.8). This 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 and save them as current inspection data. 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 any deterioration has gotten worse.

Figure 4.4.8 Electronic Data Collection

4.4.4 Inspection Report Documentation

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 file, in addition to any standard inspection forms. The inspection file contains:

- Standard nomenclature and abbreviations for the elements of members and the components made up of these 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)
- A standard notation system for indicating the condition of the elements or members
- A log or index for photographs
- Brief narrative descriptions of general and component conditions

When the above, detailed file format is selected for recording bridge inspection results, the information is to be recorded systematically. However, many bridge owners differ significantly in their required format. Most of the above information, if not provided on the inspection report, is available in the bridge record.

Element Identification

Identify the elements by the type of material, construction method, and the function that each element or member performs.

Some examples of elements or members and their abbreviations:

- Multi-beam (B1 – B6)
- Deck or 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 – U.S.)
- Spandrel wall (U.S., D.S. or N, S, E, W)
- Abutment (Abut. 1, Abut. 2)
- Pier (P1 – P4)

Verify that element descriptions or abbreviations are consistent with bridge owner nomenclature.

Structure Site Orientation

Structure site orientation is normally established according to highway direction of inventory, mile markers, segments, or stationing. It is important that the orientation of each bridge be clearly established. The following are some examples:

- I79, Milepost 155.28 NB
- SR0019 Segment 05010
- Union Township, Alpha Drive, Station 109+05

Bridge Member Orientation

When describing bridge members, it is important to clearly identify the specific element or member that has the deficiency. The following are some examples to orient bridge members:

- Substructure units (e.g., Abutment 1 and Pier 3) (see Figure 4.4.9).
- Floorbeam ends are identified by left/right looking in the direction of inventory or 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 to identify general areas on the bridge (see Figure 4.4.9).
- Individual beams or stringers left to right, looking in the direction of inventory (see Figure 4.4.10).
- Upstream or downstream designations can be assigned to structures over waterways (e.g., “upstream truss”, “downstream girder”, or “upstream arch”) (see Figure 4.4.11).
- For truss elements, identify the member with joint designations and specify if it is an upstream/downstream or north/south truss (see Figure 4.4.12). 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, clearly state these differences in the inspection notes.

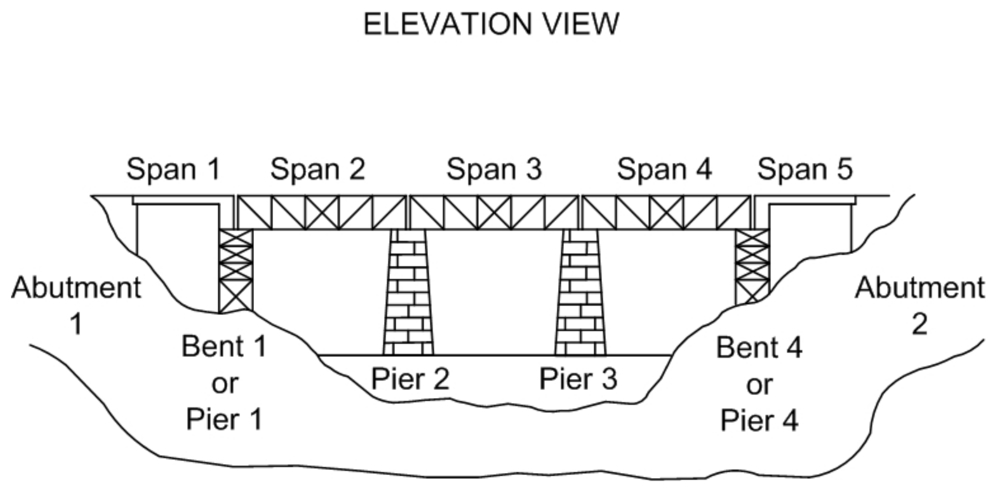

Figure 4.4.9 Sample Span Numbering Scheme

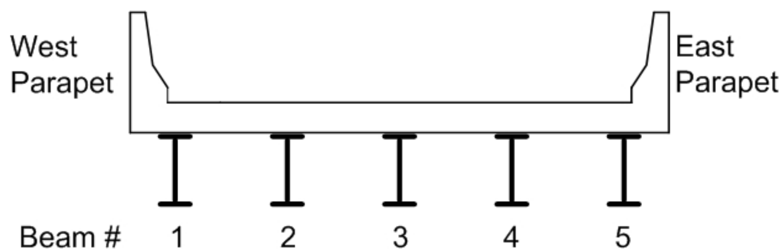

Figure 4.4.10 Sample Typical Section Numbering Scheme

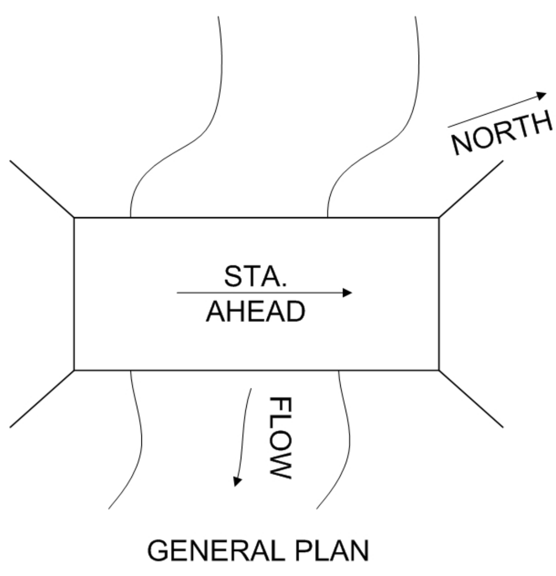

Figure 4.4.11 Sample Structure Orientation Sketch

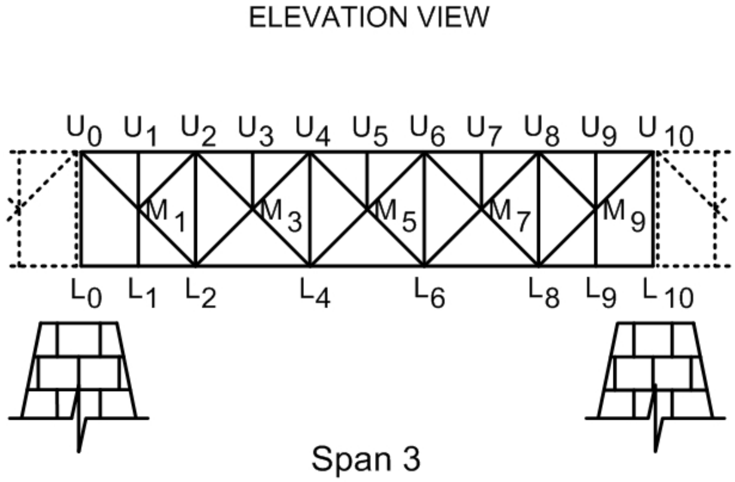

Figure 4.4.12 Sample Truss Numbering Scheme

Element Dimensions

Document sufficient dimensions to establish the size or cross section and other pertinent dimensions of elements. These include:

- Deck elements: length, width, and thickness
- Superstructure elements (beam, girder, floorbeam, stringer, and truss member): length, depth, width, flanges, and webs (see Figures 4.4.13 and 4.4.14)
- Substructure elements (abutment, columns and caps): width and depth (for rectangular shapes), diameter (for round columns), length, spacing, and pile batter and spacing (for pile bents)

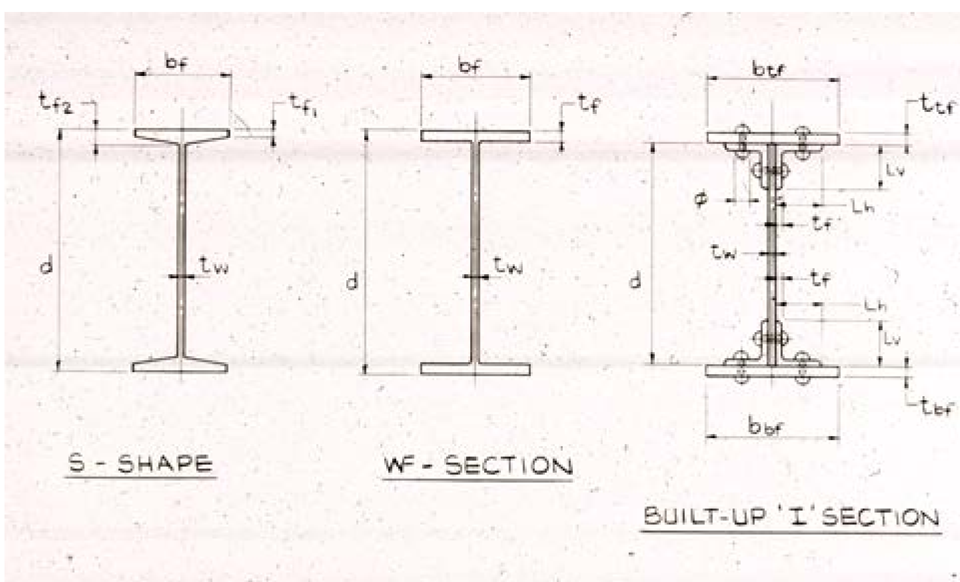

Figure 4.4.13 Steel Superstructure Dimensions

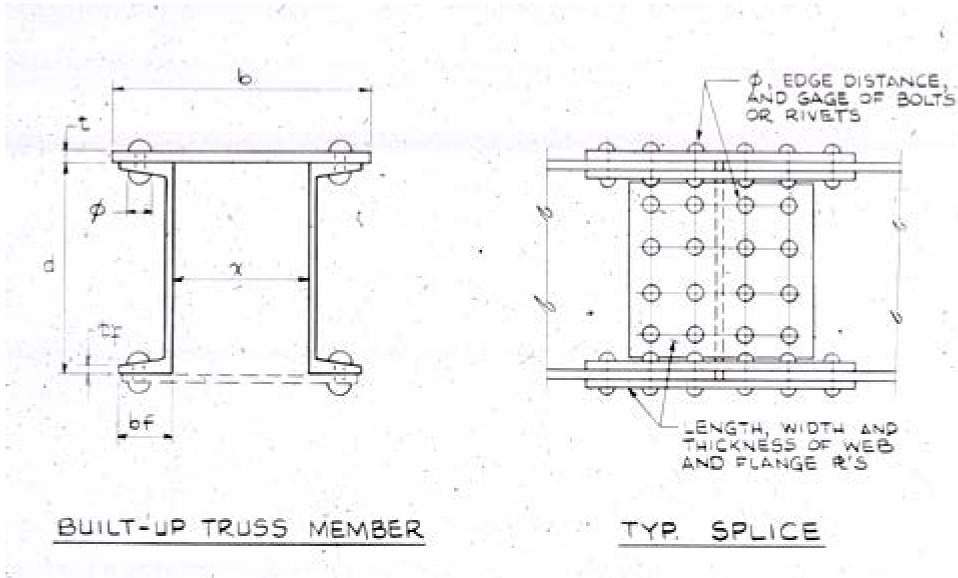

Figure 4.4.14 Truss Member and Field Splice Dimensions

Exact member dimensions are required to determine section properties used to calculate a load-rating analysis.

Inspection Notes and Sketches

In most cases, it will be possible to insert reproductions of portions of the plans in the inspection notes. 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 4.4.15 through 4.4.17).

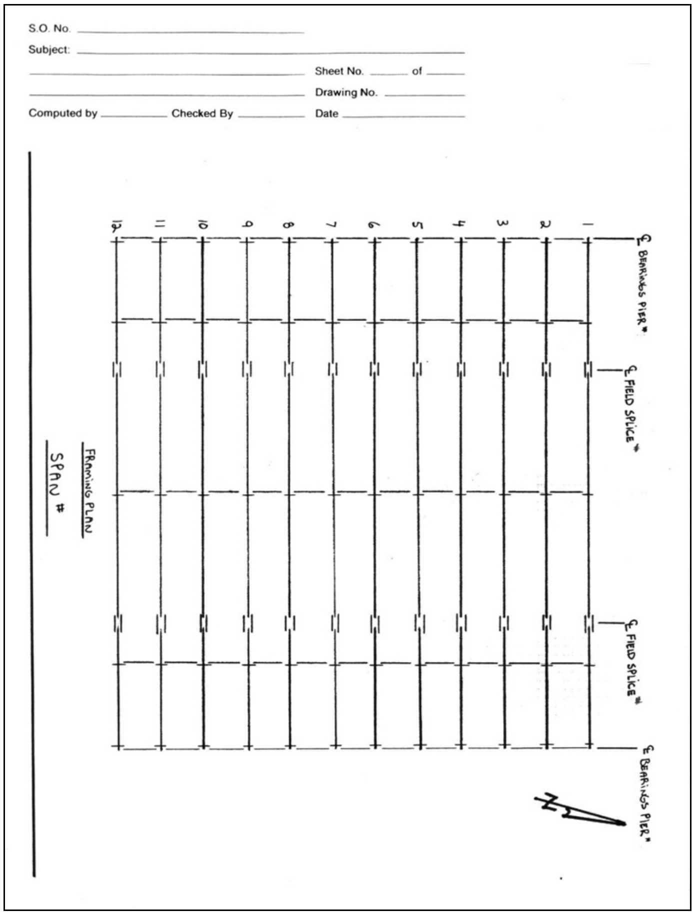

Figure 4.4.15 Framing Plan

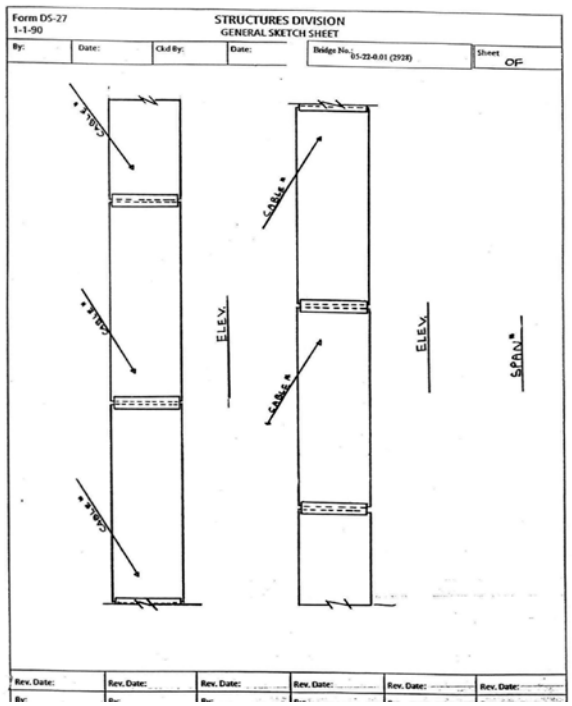

Figure 4.4.16 Girder Elevation

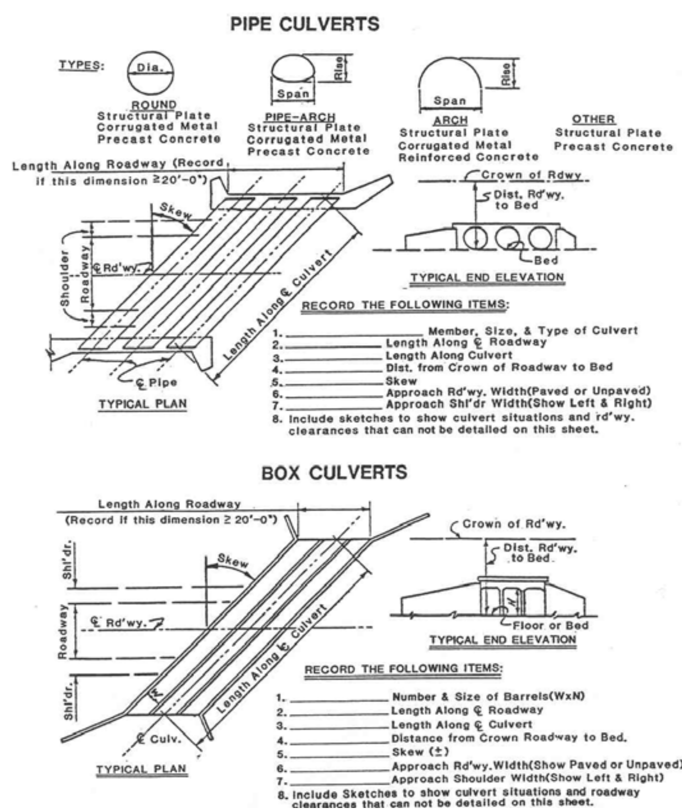

Figure 4.4.17 Typical Prepared Culvert Sketches

The first sketch in the field inspection notes normally portrays the general layout of the bridge and site information, illustrating the structure plan and elevation data (see Figures 4.4.18 and 4.4.19). The immediate area, the stream or terrain obstacle layout, major utilities, and any other pertinent details are also included.

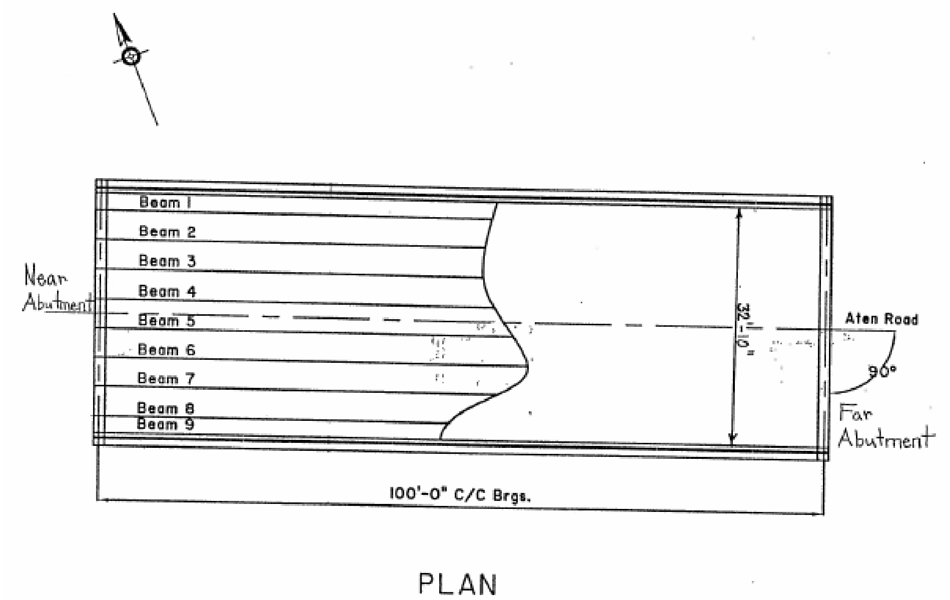

Figure 4.4.18 Sample General Plan Sketch

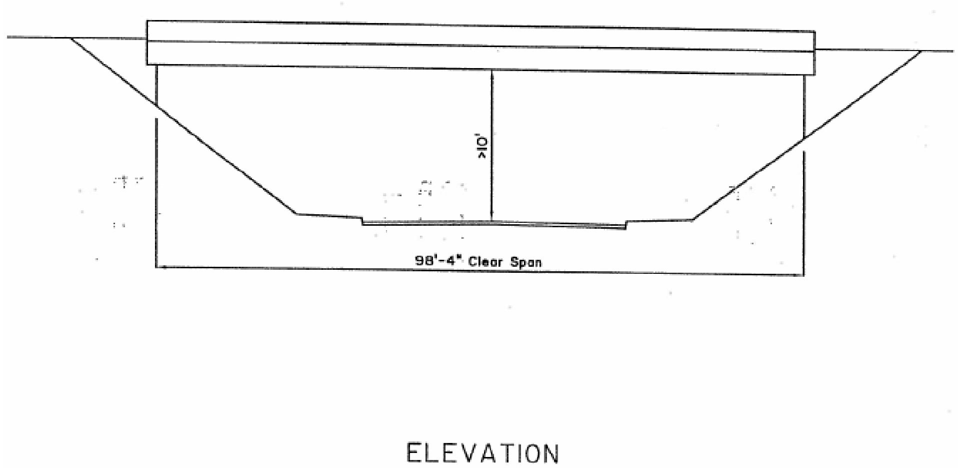

Figure 4.4.19 Sample General Elevation Sketch

Deficiency Identification

Identify material deficiencies. as presented in Topic 6.1 – Timber, Topic 6.2 – Concrete, Topic 6.3 – Steel, Topic 6.5 – Masonry.

The exact location, severity and extent of deficiencies are used to determine the capacity of the bridge in its current condition.

Deficiency Qualification Describe the seriousness of a deficiency. 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 the amount of misalignment

Deficiency Quantification Describe the quantity of a deficiency. For example:

- Spalling – 2 feet x 3 feet x 2 inches deep
- Scaling – 4 feet high by full abutment width
- Delamination – 1 foot x 6 inches
- Decay – 2 feet x 2 feet x 3 inches deep

Deficiency Location The exact position of the deficiency 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 deficiencies:

- Bending moment – Maximum positive moment occurs at or near midspan. Maximum negative moment occurs at the intermediate supports if the structure is continuous.
- Shear/bearing – Shear is maximum at or near the supports. Bearing is maximum at 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 deficiency 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 deficiency.
- 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, identify the exact position of the deficiencies in all members using reference points, regardless of the forces acting on the member.

Locating a deficiency may include tying it to an established permanent reference. Avoid using references that can change over time.

Some examples of proper referencing include:

- 7 feet-3 inches from fixed bearing on Beam 3 at Abutment 1
- 3 feet-1inch from west corner of Abutment 2
- 2 feet-6 inches below bridge seat on south face of Column 1, Pier 2

Reference points to avoid, since these locations vary between inspections:

- Expansion rocker faces
- Ground levels, especially those that may be exposed to water
- Water levels

When documenting the deficiency locations on the deck, include the condition of deck and haunch, expansion joints, construction joints, curbs, sidewalks, parapets, and railings with the deck sketches (see Figure 4.4.20).

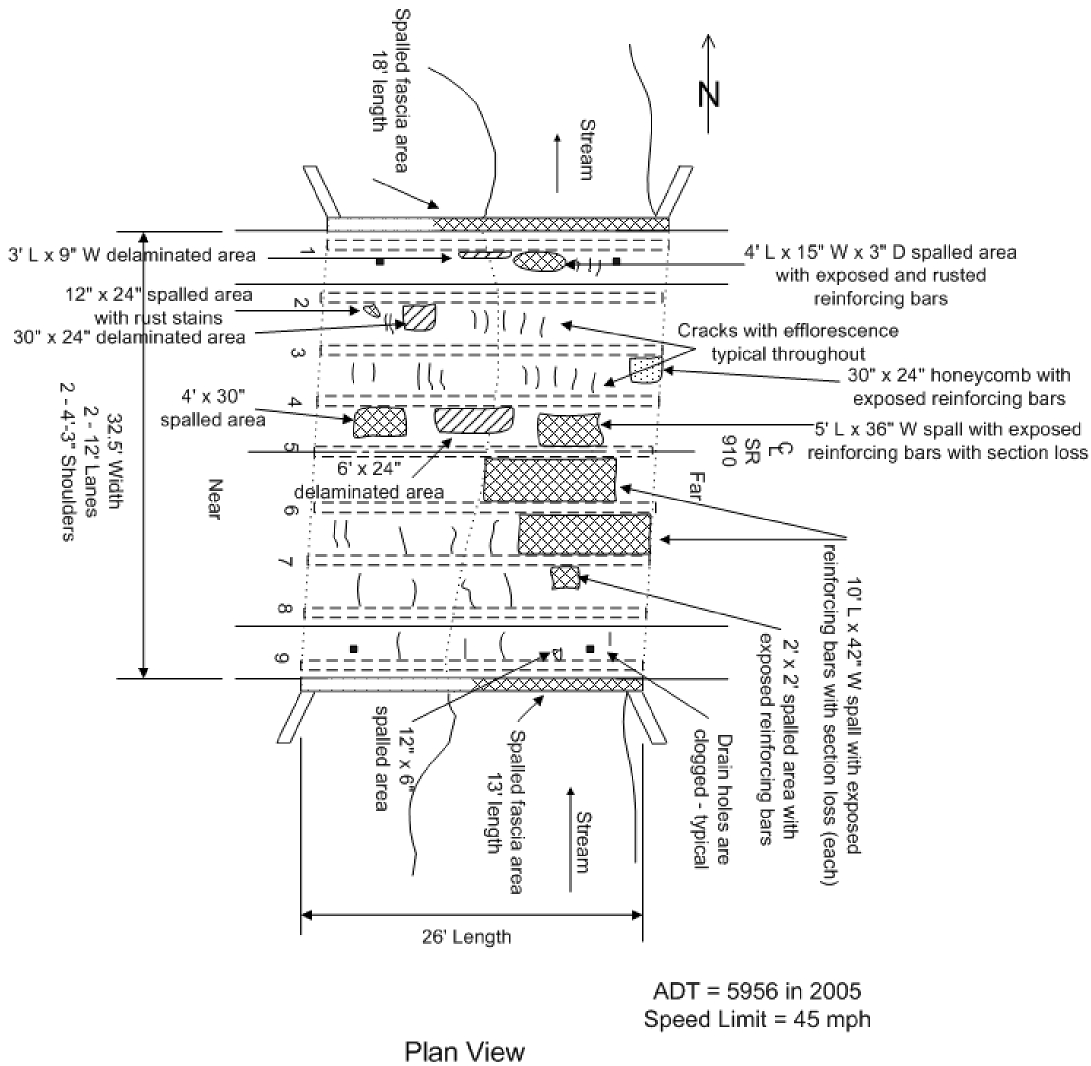

Figure 4.4.20 Sample Deck Inspection Notes

When documenting the deficiency location of the superstructure, sketch the superstructure units in plan view and elevation, or cross section if necessary. Items to be inspected include bearings, main-supporting longitudinal members, floorbeams, stringers, bracing, and diaphragms (see Figure 4.4.21).

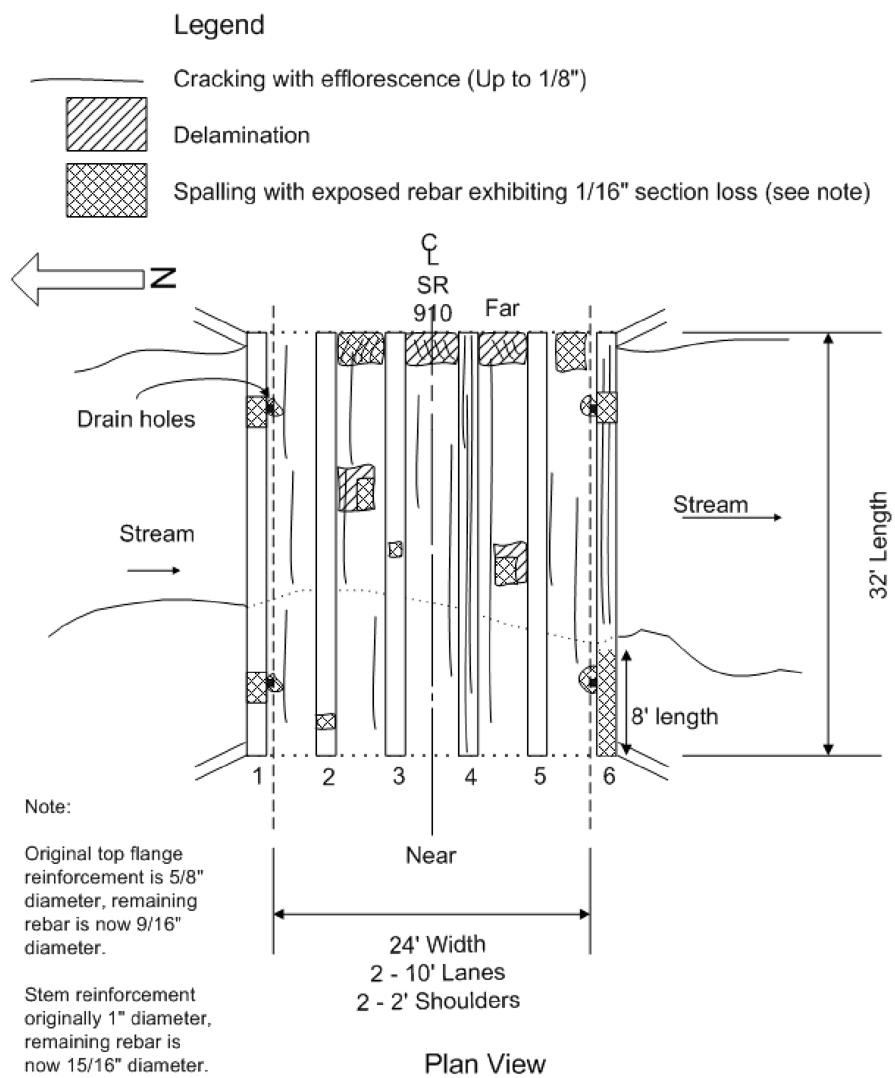

Figure 4.4.21 Sample Superstructure Inspection Notes

Include sketches or drawings to describe the condition of each substructure unit (see Figure 4.4.22). In many cases, it is sufficient to draw typical units that identify the principal elements and deficiencies of the substructure. Identify each element of the substructure unit so that they can be cross referenced to the notes or sketches. Items to be identified include piling, footings, vertical supports, lateral bracing of members, and caps.

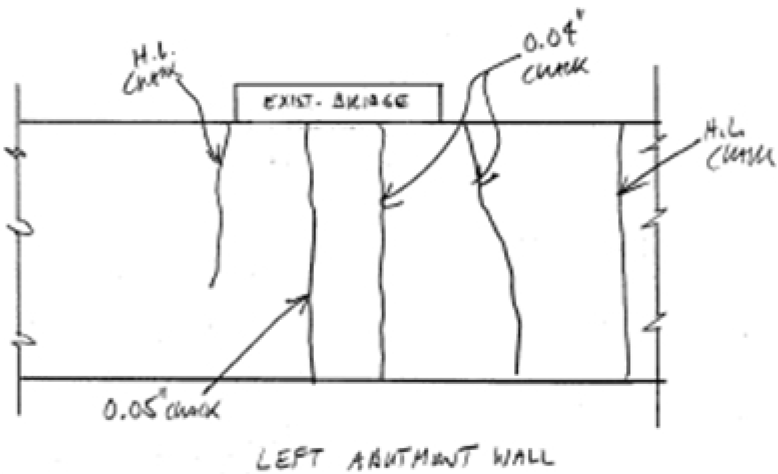

Figure 4.4.22 Sample Substructure Inspection Notes

Include sketches or drawings to describe the condition of the channel (see Figure 4.4.23). Streambed materials, alignment, condition of the banks, and the condition of the bottom of the waterway (including scour holes) are included in the sketch.

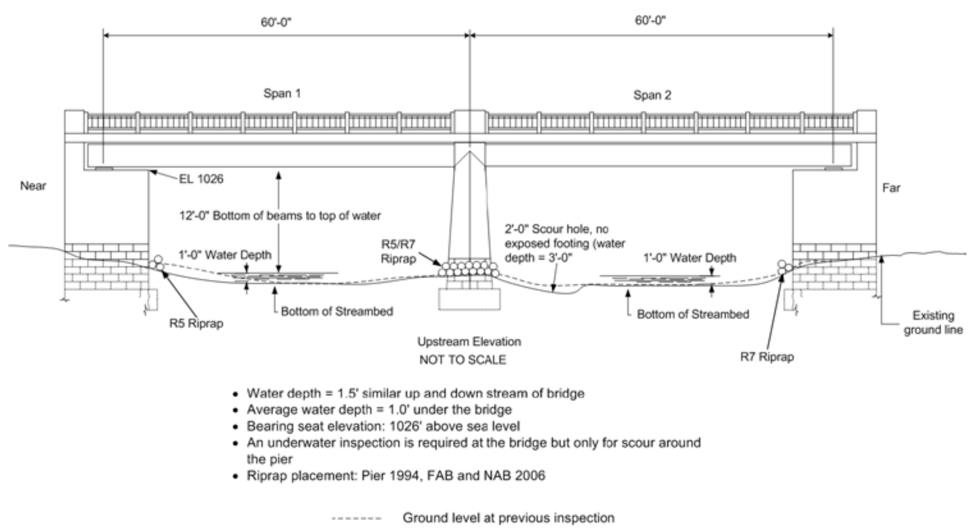

Figure 4.4.23 Sample Channel Inspection Notes

Summary of Findings

Report all deficiencies, no matter how minor they may seem. Be as descriptive as necessary to report not only the severity of the deficiency but the location as well. This will be described in further detail later in this topic. When reporting deficiencies, be objective and do not use terms such as “dangerous” or “hazardous”.

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SITE DATA
Form A

Form D-450A

5A01 SR ID: ____ **5A03** BMS Ref: ____ **7A01** Inspection Date: ____

1A09	Inspection Status:	____
7A02	Team Leader:	____
7A03	Inspection Type:	____
7A05	Inspected By:	____

Structure Description

5A08	FHWA Facility Carried:	____
5A07	Features Intersected:	____
5A09	Location:	____
5C01	Roadway Name:	____
5A06	City / Borough Name:	____

Structure Type

Main			Approach		
6A26	Material Makeup:	____	6A26	Material Makeup:	____
6A27	Physical Makeup:	____	6A27	Physical Makeup:	____
6A28	Span Interaction:	____	6A28	Span Interaction:	____
6A29	Structural Config:	____	6A29	Structural Config:	____

Sign Information

ID01 Type of Sign	ID02 Sign Needed	ID03 Sign Message	ID06 Near Adv	ID04 Bridge Site Near	ID07 Bridge Site Far	ID05 Far Adv	Comments
0 - Bridge							
1 - Bridge Weight Limit							
2 - Except Combinations							
3 - One Truck at a Time							
4 - Vertical Clearance On							
5 - Vertical Clearance Under							
6 - One Lane Bridge							
7 - Narrow Bridge							
8 - Hazardous Clearance							
9 - Other							

Report Version Date:

Page 1 of 3

This document includes structure safety inspection information that is confidential pursuant to 65 P.S. §66.1 et seq., 75 PA. C.S. §3754 and 23 U.S.C. §409 and may not be disclosed or used in litigation.

Figure 4.4.24 Example Inspection Form – PennDOT Form D-450

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SITE DATA
Form A

Form D-450A

5A01 **SR ID:** ____________ **5A03** **BMS Ref:** ____________ **7A01** **Inspection Date:** ____________

Features Intersected

6C02		5C03		5C06	5C29	4A20	4A19	6C18	6C19	6C20	6C21	6C22	6C23	6C24	6B17
SR ID		On/	Skew			Min Lat Cl		Tot Hor Cl		Min Vrt Cl Rdwys		Vert Cl Over 10ft		VT	
SR	Seg	Under	Angle	Dir	NHS	Left	Right	Left	Right	Left	Right	Left	Right	Sign	ADT

6B15 **Design Exceptions:** ____________
6A51 **Sub Latent Problem:** ____________
6A50 **Sup Latent Problem:** ____________

Deck Geometry

Table Used for Appraisal: ____________

Controlling Values

5C10 **ADT:** ____________
5C27 **Bridge Road Width:** ____________
4A10 **Appraisal:** ____________
Notes: ____________

4A11 **Underclr Appr:** ____________
6B13 **Controlling Vertical:** ____________
Controlling Lateral: ____________

Traffic Safety Features

Feature Type	IA01 Location	IA02 Adequacy Rating	IA03 Description	5C08 Posted Spd Lmt (mph)
1 - Railing				
Comment:				
2 - Transition				
Comment:				
3 - Approach Guiderail				
Comment:				
4 - Approach Railend				
Comment:				

Report Version Date:

Page 2 of 3

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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DECK AND SUPER STRUCTURE DATA
Form B

Form D-450B

5A01 SR ID: ________ 5A03 BMS Ref: ________ 7A01 Inspection Date: ________

Deck Wearing Surface

Main		Approach	
5B02	Type of Wearing Surface: ________	6A30	Type of Wearing Surface: ________
5B03	Type of Memb. Water-Proof: ________	6A31	Type of Memb. Water-Proof: ________
5B04	Deck Corrosion Protection: ________	6A32	Deck Corrosion Protection: ________
6A33	Thickness: ________	6A33	Thickness: ________
6A34	Date Recorded: ________	6A34	Date Recorded: ________

6B40 Condition Rating: ________

IC02 Dk WS Notes: ________

Expansion Joints 6A41 Number of Expansion Joints: ________

Joint Number	VD25 Joint Type	VD26 Movement Class	VD27 Manufacture Code

Deck

1A01 Condition Rating: ________

6B07 Est. Spall Delamination: ________ 6B08 Date: ________

6B10 Est. Chloride Content: ________ 6B11 Date: ________

1A07 Unrepaired Spalls: ________

Deck Top: ________

Deck Underside: ________

Deck Drainage: ________

Expansion Joints: ________

Deck Notes: ________

Report Version Date:

Page 1 of 2

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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DECK AND SUPER STRUCTURE DATA
Form B

Form D-450B

5A01 **SR ID:** ____________ **5A03** **BMS Ref:** ____________ **7A01** **Inspection Date:** ____________

Superstructure

1A04 **Condition Rating:** ____________

Narrative: ____________

Girders/Beams: ____________

Floorbeams: ____________

Stringers: ____________

Diaphragms: ____________

Truss Members: ____________

Bearings: ____________

Drainage System: ____________

Report Version Date:

Page 2 of 2

This document includes structure safety inspection information that is confidential pursuant to 65 P.S. §66.1 et seq., 75 PA. C.S. §3754 and 23 U.S.C. §409 and may not be disclosed or used in litigation.

Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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ABUTMENT DATA

Form C

Form D-450C

5A01 SR ID: ____________ 5A03 BMS Ref: ____________ 7A01 Inspection Date: ____________

1A02 Substructure Condition Rating: ____________

Notes: ____________

Near Abutment

Backwall: ____________

Bridge Seats: ____________

Cheekwalls: ____________

Stem: ____________

Wings: ____________

Footing: ____________

Piles: ____________

IN20 Scour / Undermine: ____________

Settlement: ____________

Embank Slope-Wall: ____________

Wall Drainage: ____________

Far Abutment

Backwall: ____________

Bridge Seats: ____________

Cheekwalls: ____________

Stem: ____________

Wings: ____________

Footing: ____________

Piles: ____________

IN20 Scour / Undermine: ____________

Settlement: ____________

Embank Slope-Wall: ____________

Wall Drainage: ____________

Report Version Date:

Page 1 of 1

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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PIER DATA
Form D

Form D-450D

5A01 **SR ID:** ____________ **5A03** **BMS Ref:** ____________ **7A01** **Inspection Date:** ____________

Navigational Control

4A21 **Controls Exist:** ________

4A22 **Vert Clearance:** ________
4A24 **Lift Vertical:** ________
4A23 **Horz Clearance:** ________
4A07 **Pier Protection:** ________

Pier Details

5D02 **Pier/Bent Number:** ________ **IN20** **Scour / Undermine:** ________
Condition Summary: ________
Bridge Seats: ________
Cheekwalls: ________
Columns/Stems: ________
Settlement: ________

5D02 **Pier/Bent Number:** ________ **IN20** **Scour / Undermine:** ________
Condition Summary: ________
Bridge Seats: ________
Cheekwalls: ________
Columns/Stems: ________
Settlement: ________

5D02 **Pier/Bent Number:** ________ **IN20** **Scour / Undermine:** ________
Condition Summary: ________
Bridge Seats: ________
Cheekwalls: ________
Columns/Stems: ________
Settlement: ________

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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ELEMENT DATA
Form E

Form D-450E

5A01 **SR ID:** ____________ 5A03 **BMS Ref:** ____________ 7A01 **Inspection Date:** ____________

6B03 **Inventory Item Review Recommended:** ____________

IC01 **Notes:** ____________

Element Details

5D02 **Span:** ____________ 5D04 **Span Type:** ____________

1B01 **Element ID:** ____________ **Inspect by Each:** ____________

Environment: ____________ 1B05 **Scale Factor Measurement:** ____________

Description: ____________

1A10 **Today QTY:** ____________ 1A11 **Cond State 1 QTY:** ____________ 1A11 **Cond State 2 QTY:** ____________

1A11 **Cond State 3 QTY:** ____________ 1A11 **Cond State 4 QTY:** ____________ 1A11 **Cond State 5 QTY:** ____________

Condition: ____________

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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FRACTURE CRITICAL
Form F

Form D-450F

5A01 SR ID: ____ 5A03 BMS Ref: ____ 7A01 Inspection Date: ____

Main

6A44 Group: ____
Critical Rating Factor: ____
Total Critical Rating Factor: ____

Structure Type

6A26 Material Makeup: ____
6A27 Physical Makeup: ____
6A28 Span Interaction: ____
6A29 Structural Config: ____

Approach

6A44 Group: ____
Critical Rating Factor: ____
Total Critical Rating Factor: ____

Structure Type

6A26 Material Makeup: ____
6A27 Physical Makeup: ____
6A28 Span Interaction: ____
6A29 Structural Config: ____

Fracture Critical Details

IF01 Location: ____ IF02 Type: ____ IF05 FC Stress Category: ____
IF03 Member: ____
IF04 Member Detail: ____
IF06 Notes: ____

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Page 1 of 1

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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UNDERWATER INSPECTION
Form G

Form D-450G

5A01 **SR ID:** ________ 5A03 **BMS Ref:** ________ 7A01 **Inspection Date:** ________

IU00a **UW Reviewer Action:** ________
IU00b **Reviewer Comments:** ________

IU02 **Number of Units:** ________ IU01 **Recalculate SCBI:** ________
IU03 **SCBI Source:** ________ 4A08 **SCBI:** ________

Overall SCBI: ________ **SAR:** ________
IU06 **Streambed Material #1:** ________
IU06 **Streambed Material #2:** ________
IU07 **Notes:** ________

Current Countermeasures

CM Num	IU21 Type	IU22 Location	IU23 Condition	IU24 Subunit

Possible Countermeasures

PCM Num	IU25 Location	IU26 Work Candidate

SAR Calculation Data

IU08 **Debris Potential:** ________
IU09 **Trapping Potential:** ________
IU10 **Pressure Flow:** ________
IU11 **NAB Location:** ________ IU12 **FAB Location:** ________

US Left Wingwall
IU13 **Presence:** ________ IU14 **Condition:** ________

US Right Wingwall
IU15 **Presence:** ________ IU16 **Condition:** ________

Horizontal Debris Blockage
IU17 **Start:** ________ IU18 **End:** ________

Vertical Debris Blockage
IU19 **Start:** ________ IU20 **End:** ________

Report Version Date: Page 1 of 2

This document includes structure safety inspection information that is confidential pursuant to 65 P.S. §66.1 et seq., 75 PA. C.S. §3754 and 23 U.S.C. §409 and may not be disclosed or used in litigation.

Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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DEPARTMENT OF TRANSPORTATION

UNDERWATER INSPECTION
Form G

Form D-450G

5A01 **SR ID:** ____________ **5A03** **BMS Ref:** ____________ **7A01** **Inspection Date:** ____________

Sub Unit OSA Data

Observed Scour Rating Components

IN01	IN12	IN13	IN14	IN15	IN19	IN04	IN05	IN06	IN07	IN08	IN09	IN10	IN11	IN03
Sub Unit	Pier/ Abut Type	Inv. Found Type	Found Type	Strmbd Mat	Move Ind	Chg Since Last Insp	Scour Hole	Debris Potential	Scour-ability	Opening Adeq. / Channel	Sediment	Alignment	Velocity/ Stream Slope	Observed Scour Rating

Other Subunit Details

IN01	IN16	IN18	IN17	IN20	IN21	IN02	IN22	IN23		
Sub Unit	UW Insp Type	Water Dept	Observed Scour Depth	Scour Undermine	Counter-measures	Info from Current Insp	100 yr Flood Calc Scour Depth	500 yr Flood Calc Scour Depth	SCBI Code	SAR

IN24 **Notes:**

IN24 **Notes:**

IN24 **Notes:**

IN24 **Notes:**

Underclearance

IL09 **Origin Description:** ____________
IL10 **Horizontal:** ____________
IL11 **Vertical:** ____________
IL12 **Notes:** ____________

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et seq., 75 PA. C.S. §3754 and 23 U.S.C. §409 and may not be disclosed or u

Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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CULVERT DATA
Form H

Form D-450H

5A01 **SR ID:** ____________ **5A03** **BMS Ref:** ____________ **7A01** **Inspection Date:** ____________

1A03 **Culvert Condition Rating:** ____________

Notes: ____________

5B18 **Length of Culvert Barrel:** ____________

#	Opening Type	Length	Min Fill Height	Max Fill Height	Eff Width
___	___	___	___	___	___

Top Slab: ____________
Barrel: ____________
Floor/Paving: ____________
Headwall: ____________
Wings: ____________
Settlement: ____________
Debris: ____________

Report Version Date:

Page 1 of 1

This document includes structure safety inspection information that is confidential pursuant to 65 P.S. §66.1 et seq., 75 PA. C.S. §3754 and 23 U.S.C. §409 and may not be disclosed or used in litigation.

Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

pennsylvania
DEPARTMENT OF TRANSPORTATION

CHANNEL AND WATERWAY DATA

Form J

Form D-450H

5A01 SR ID: ________ **5A03** BMS Ref: ________ **7A01** Inspection Date: ________

Channel

1A05 Channel/ Channel Protection Cond. Rating: ________

Channel: ________

Banks: ________

Streambed Movements: ________

Debris, Vegetation: ________

River Control Devices: ________

Embank/Strmbed Contr: ________

Drift, Other: ________

Waterway Adequacy

1A06 Appraisal Code: ________

Notes: ________

IL02 Overtop Risk: ________

IL03 Traffic Delay: ________

5C22 Functional Class: ________

High Water Mark

IL05 Elevation: ________ **IL06** Date: ________ **IL07** New High Water Mark: ________

Notes: ________

Report Version Date:

Page 1 of 1

This document includes structure safety inspection information that is confidential pursuant to 65 P.S. §66.1 et seq., 75 PA. C.S. §3754 and 23 U.S.C. §409 and may not be disclosed or used in litigation.

Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

pennsylvania
DEPARTMENT OF TRANSPORTATION

PAINT, STRUCTURE APPRAISAL AND LOAD RATINGS
Form K

Form D-450K

5A01 **SR ID:** ________ 5A03 **BMS Ref:** ________ 7A01 **Inspection Date:** ________

Paint Condition

6B36 **Paint Cond Rating:** ________ 6B37 **Ext of Paint Cond:** ________

6B35 **New Paint:** ________

Int Beam / Gird: ________

Fascias: ________

Splsh Zone: Truss Gird: ________

Truss: ________

Bearings: ________

Other: ________

4B03 **Brdge Cap. Appraisal:** ________

6B16 **Controling:** ________

4A09 **Struct Cond Appraisal:** ________

Structure Condition Appraisal Based on

____

Load Ratings

4B15 **Load Rating Review Recommended:** ________

Due To: ________

Calculation Date: ________

Rating Approval Date: ________

Load Rating Details

LOAD TYPE	IR10 IR LOAD	IR11 OR LOAD	IR05 NBI IND	IR06 RTNG ANAL METH	IR07 CONT MEM TYPE	IR16 ANALYSIS ENGINEER	IR14 AASHTO MANUAL YEAR	IR15 AASHTO SPEC YEAR	IR13 OPR GOV CRITERIA	IR12 INV GOV CRITERIA
Notes:										
Notes:										
Notes:										
Notes:										

Report Version Date:

Page 1 of 1

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

pennsylvania
DEPARTMENT OF TRANSPORTATION

MAINTENANCE NEEDS DATA

Form M

Form D-450M

5A01 **SR ID:** ____________ **5A03** **BMS Ref:** ____________ **7A01** **Inspection Date:** ____________

IM01 Type of Work	IM03 Action	IM04 Est Qty	UOM	IM05 Priority	IM06 Date Rec	IM09 Location	IM08 Target Year	IM11 Ass. WK

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

IM07 **Status:** ____________ **IM15** **Notes:** ____________

Report Version Date: Page 1 of 1

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Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

pennsylvania DEPARTMENT OF TRANSPORTATION

INSPECTION ADMINISTRATION

Form P

Form D-450P

5A01 **SR ID:** ______ 5A03 **BMS Ref:** ______ 7A01 **Inspection Date:** ______

Current Inspection

7A03 **Primary Type:** ______

7A06 **Types of Inspections Performed:**

NBI	Underwater	Element	Fracture Critical	Other Special
____	____	____	____	____

Inspection Man Hours

6B26 **NBI Crew:** ______ 6B30 **Underwater:** ______

6B28 **Fracture Critical:** ______ 6B29 **Other 1:** ______

6B27 **Crane:** ______ 6B31 **Other 2:** ______

Inspection Cost (in hundreds)

6B32 **Engineering:** ______ 6B33 **Rigging:** ______

6B34 **Office:** ______

Special Equip Used:

6B12 **Temperature:** ______ 6B09 **Weather:** ______

6B03 **Inventory Review Recommended:** ______

Change Notes: ______

Inspection Team

7A04 **Inspected By:** ______

7A02 **Team Leader:** ______

6B23 **Team Member:** ______

6B24 **Hired By:** ______

6B25 **Insp Contract Num:** ______

2A02 **Inspection Notes:** ______

Next Inspection

7A14 **Next Inspection By:** ______

6B20 **Next Insp Type:** ______

Schedule

Insp Types	7A09 Required	7A09 Frequency	7A10 Next Date
NBI	____	____	____
Fracture Critical	____	____	____
Underwater	____	____	____
Other Special	____	____	____
Element	____	____	____
Crane	____	____	6B18 ____

6B01 **Special Insp Type:** ______

Estimated Inspection Man Hours

7A12 **NBI Crew:** ______ 7A17 **Underwater:** ______

7A15 **Fracture Critical:** ______ 7A16 **Other 1:** ______

7A13 **Crane:** ______ 7A18 **Other 2:** ______

Report Version Date: Page 1 of 1

This document includes structure safety inspection information that is confidential pursuant to 65 P.S. §66.1 et seq., 75 PA. C.S. §3754 and 23 U.S.C. §409 and may not be disclosed or used in litigation.

Figure 4.4.24 Example Inspection Form – PennDOT Form D-450 (continued)

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Table of Contents

Chapter 4
Bridge Inspection Reporting

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Topic 4.5 Critical Findings

4.5.1 Definition

A critical finding are a structural or safety related deficiency that requires immediate follow-up inspection or action.

A structure related deficiencies can interrupt the load path, not allowing loads to be transferred as designed. This can cause surrounding elements to become overstressed or unstable, potentially leading to partial or total collapse of the structure. Critical findings may also be non-structural deficiencies which jeopardize the safety of motorists or pedestrians.

4.5.2 Procedures

As stated in the NBIS regulations, each state or federal agency is required to "establish a statewide procedure to assure that critical findings are addressed in a timely manner." Although specific procedures vary among agencies, general steps must be taken to assure that critical findings are identified and resolved as quickly and efficiently as possible. The viable options available are permanently repair, temporarily repair or restrict loads on the bridge.

Currently, states employ two approaches to coding condition items when localized areas of severe deterioration are encountered. Some will account for the severity of a localized area of deterioration by lowering the condition rating of an entire component. The component condition rating is adjusted after the deteriorated area is improved (i.e., rating may rise if physical improvements are made, or may stay the same if the bridge is posted for load restrictions and/or supported with temporary shoring). FHWA recognizes this approach <u>when the severity of the localized deterioration affects the load-carrying capacity of the component</u>.

Other states rate to the general condition regardless of the severity of a localized area of deterioration. This approach relies heavily on ensuring that critical findings are addressed in a timely manner regardless of the component condition rating value. If the localized area of severe deterioration is not improved following the critical finding follow-up process, the component rating may need to be lowered to account for the severity of the deterioration if structural capacity is affected.

Either approach to coding the condition items results in the same ultimate outcome, i.e. critical inspection findings are addressed to allow continued safe use of the bridge. Component ratings <u>eventually</u> reflect the overall condition of the component. If the approach is to consider both the severity and extent of a component's deterioration in rating each component at the time of inspection (or up to 90 days after the inspection as required by the NBIS), there cannot be any assumptions about future improvements made to a localized area. Only if an improvement is made, the rating should then be raised as appropriate. If the improvement is made within 90 days of the inspection, there is no need to consider the localized deterioration in the rating.

Critical findings / critical follow-up report categorical contents with the documented status:

1. Bridges that have critical findings in the process of being addressed.
2. Bridges with work scheduled but not started yet.
3. Bridges that have no plan in the works.
4. Critical Finding is scour related.

Procedures for Inspectors

Upon identifying a potential critical finding, immediately report the deficiency to the appropriate agency official, bridge owner, or governing authority. For most agencies, a verbal notification is required soon after identifying the potential critical deficiency.

In addition to a verbal notification, agencies require immediate written notification of the potential critical finding. This notification is often presented in a standardized hardcopy or electronic format (see Figures 4.5.1 and 4.5.2), and is submitted soon after the verbal notification for most agencies. The written notification serves to document the critical finding by describing the extent of the deficiency complete with notes, photographs, sketches and drawings, measurements, possible causes, and recommendations for repair. Temporary actions may also be taken at this time to safeguard the public until proper repairs can be completed. These actions may include:

- Load posting
- Traffic restrictions from the damaged area
- Speed restrictions
- Temporary lane closure
- Temporary shoring
- Complete bridge closure

After submittal of the written report, the finding will be assessed and the severity determined along with a proposed repair strategy or plan of action. In accordance with NBIS regulations, the agency is also required to notify the FHWA of the critical finding. Public works officials or law enforcement may also be contacted as needed.

Missouri Department of Transportation
Critical Inspection Finding
State System

Bridge ________ District ____ County ________________ Route _________ AADT _________
Location ________________ Inspector _______________________ Inspection Date _________

Reason for Critical Inspection Finding report: Be specific about deficiencies. Attach Photographs.

Inspector's Immediate Recommendations:

- ☐ Immediate Closure Required ☐ Immediate Blocking/Shoring Required
- ☐ Reduce traffic to one-lane. Carry traffic on ☐ NB ☐ SB ☐ EB ☐ WB lane
- ☐ Other:

Immediate Notification: ☐ State BM Engr ☐ Supv Bridge Insp Engr ☐ __________

MoDOT Action Plan by Bridge Maintenance and the District: **Date**: _________

Follow-up Actions: **Completion Date**: _________

Figure 4.5.1 Missouri DOT Critical Inspection Finding Form

Washington State
Department of Transportation

Critical Damage - Bridge Repair Report

Agency Name	Charge Code	Bridge Name
Structure Identifier	Bridge Number	Bridge Location (Longitude/Latitude)
Inspector (Print Name)	Inspector's ID Number	Inspection Date
Describe Deficiency		
Describe Recommended Repair		
Anticipated Date of Completion	Submitted By (Print Name)	Date Submitted
Describe Work Done		
Date of Completion	Submitted By (Print Name)	Date

DOT Form 140-151 EF
6/98

Figure 4.5.2 Washington State DOT "Critical Damage - Bridge Repair Report"

Office Priority Maintenance Procedures

Agencies establish priority maintenance procedures and prioritization criteria to help facilitate maintenance work plan strategies. Most agency systems utilize between three and five different prioritization levels ranging from general housekeeping and routine repairs to critical findings requiring immediate action. Examples of agency priority maintenance procedures are listed below in the order of most critical to least critical, with a description of each level.

Oregon Department of Transportation (ODOT)

- "Significant" – Severe deficiency to a primary bridge element that requires complete or partial closure of the bridge, or an immediate load restriction of the bridge.
- "Critical" – Serious deficiency to a primary bridge element that needs repair to prevent the bridge from being load posted.
- "Urgent" – Traffic safety related concern that does not jeopardize the reliability of the transportation system, protection of public investments, or maintenance of legal federal mandates.
- "Routine/Schedule" – Minor to moderate deficiency to a primary bridge element or moderate to major deficiency to a secondary element.
- "Monitor" – Non-structural housekeeping repairs such as cleaning the deck and drainage systems.

North Carolina Department of Transportation (NCDOT)

- "Critical Finding" – Severe deficiency to a primary bridge element that could cause partial or complete collapse or a safety feature deficiency that may jeopardize the safety of the public.
- "Priority Maintenance need" – Serious deficiency that may lead to load posting and/or bridge closures if left untreated.
- "Routine Maintenance need" – Minor to moderate deficiencies to primary or secondary bridge elements or non-structural housekeeping repairs such as cleaning the deck and drainage systems.

Pennsylvania Department of Transportation (PennDOT)

- "0 – Critical" – Severe deficiency to a primary bridge element that could directly or indirectly cause partial or complete structure collapse or a safety feature deficiency that may result in loss of vehicle operator control or failure to contain errant vehicles on the bridge deck.
- "1 – High Priority" – Serious deficiency to a primary bridge element that may lead to load posting and/or bridge closures. If left untreated, the deficiency may also jeopardize public safety.
- "2 – Priority" – Advanced deficiency on a primary bridge element or appurtenance that if left untreated, may lead to continuing deterioration, load posting, or partial or complete bridge closures.
- "3 – Schedule" – Minor deficiency to a primary bridge element or appurtenance that may continue to deteriorate if lead untreated.
- "4 – Program" – Note-worthy problem on a primary bridge element, secondary element, or appurtenance that may lead to a documentation-worthy deficiency if left untreated.
- "5 – Routine" – Non-structural housekeeping maintenance that may lead to deterioration of primary and secondary structural members if left untreated.

Bridge Closing Procedure

In some situations, the bridge may need to be closed until the critical finding can be repaired. The decision to close the bridge may result from the nature of the critical finding upon initial discovery, an unacceptable timeframe in which the repairs are scheduled to be completed, or agency policy on critical findings.

For situations recommending closure of the bridge by the bridge inspector and/or bridge maintenance supervisor, follow established State or Federal Agency procedures. Examples of acceptable procedures include:

- Contact the Bridge Maintenance Supervisor about the recommended closing.
- Contact the Bridge Engineer about the recommended closing.
- If both the Bridge Maintenance Supervisor and Bridge Engineer are unavailable, contact the District or Division office about the recommended closing.

4.5.3 Examples of Critical Findings

FHWA guidance for a follow-up may include a procedure where the State promptly submits to the Division office a copy of inspection reports or recommendations for all on-system and off-system bridges that meet the following criteria:

1. Bridges with recommendations for immediate work on fracture critical members;
2. Bridges with recommendations for immediate correction of scour or hydraulic problems;
3. Bridges with condition ratings of 3 or less for the superstructure or substructure or appraisal ratings of 3 or less for waterway adequacy; and
4. Bridges with recommendations for immediate work to prevent substantial reduction in the safe load capacity.

Source: http://www.fhwa.dot.gov/bridge/0650csup.cfm

Many state agencies publish examples of critical findings for bridge inspectors. It should be noted that these lists are not all-inclusive or comprehensive and should only be used as <u>guidance</u> in determining whether or not a deficiency is a critical finding.

The critical findings listed below are organized by material type and application. These deficiencies represent excerpts obtained from several agencies' critical finding documentation

Timber

The following deficiencies represent examples of critical findings for timber:

- Through-loss in deck planks and broken planks in danger of breaking through.
- Primary structural members with collision damage that compromises the structural capacity (including severe section loss, full length horizontal cracking, and section loss to truss compression members producing

member buckling or severe flexural cracking).

- Primary structural members with multiple open cracks in high stress regions or crushing/decay that may lead to superstructure settlement.
- Crushed or broken nailer boards or broken joists.
- Piles and pier caps that have loss of bearing capacity or soil retention through crushing, decay, or insect damage.
- Substructure units with severe scour and undermining of the substructure foundation causing instability.

Concrete

The following deficiencies represent examples of critical findings for concrete:

- Section loss (thru-hole) subject to enlargement by traffic or deep spalls with exposed rebar in danger of holing through, creating a safety hazard to passing traffic.
- Prestressed girder with spalling and broken strands or 100% deterioration at critical high stress areas.
- Non-composite prestressed adjacent box beams with serious deterioration and existing strand loss, loss of camber or torsional cracking.
- Reinforced concrete girder or pier cap with spalling and broken main rebar or 100% deterioration, with more than one bar affected at the same location in the girder.
- Reinforced or prestressed concrete girder bearing area resulting in loss of bearing area and making girder subject to settlement.
- Reinforced concrete columns with spalling and rebar section loss causing the column to be subject to failure.
- Primary structural members with collision damage that compromises the structural capacity (including severed prestressing tendons, reinforcing steel that results in flexural cracking and negative beam camber, pier shafts, and columns).
- Concrete pier column or cap with significant structural cracking that is supporting a fracture critical bridge or fracture critical component.
- Falling concrete or concrete that is delaminated or partially detached and anticipated to fall, presenting a safety hazard to under-passing motorists and/or pedestrians.
- Bearing seats that are severely deteriorated or undermined.
- Sidewalk structural supports or walking surface with damage or deterioration presenting a hazardous condition to pedestrians.
- Substructure units with severe scour and undermining of the substructure foundation causing instability.

Steel

The following deficiencies represent examples of critical findings for steel:

- Steel members with deteriorated areas that have failed in buckling, crippling, more than 10% of the connectors in a connection are missing, etc., or which makes failure likely in the near future.
- Secondary structural members (diaphragms, bracing, etc.) with extensive section loss.
- Fracture critical members subjected to impact damage including gouging or tearing, perpendicular stress cracks in either the base metal or weld metal, parallel stress cracks resulting from out-of-plane distortions or poor weld details, and severe corrosion in girder flanges, webs, in truss members, or in gusset plates.
- Primary structural members with collision damage that compromises the structural capacity (including fractures, large gouges, significant twisting/kinking of beams, and section loss to truss compression members producing member buckling or severe flexural cracking).
- Primary structural member (non-FCM member) with a completely fractured tension member due to fatigue or vehicular collision.
- Pin and hanger systems in fracture critical members with severe deterioration or severe accumulation of debris or rust packing.
- Bottom flange cover plates with cracked welds at the end of a partial length welded cover plate for a steel multi-girder or steel floorbeam.
- Substructure units with severe scour and undermining of the substructure foundation causing instability.

Roadside Hardware or Safety Features

The following deficiencies represent examples of critical findings for traffic safety features:

- Bridge railing (bridge parapets, median barriers, or structure-mounted guardrail) with damage or deterioration that may prevent containment and/or redirection of errant vehicles traveling at the posted speed limit.
- Pedestrian railing that is missing or detached, allowing a pedestrian to fall off the structure.
- Guardrail connections to bridge railing, concrete barrier rebar, or guardrail that is detached and in close proximity or projecting into traffic with potential for impact.

Signs and Lighting

The following deficiencies represent examples of critical findings for signs and lighting:

- Load posting or vertical clearance signs that are missing, damaged, improperly located, or visually obstructed including relevant advance warning signs.
- Signs, traffic signals, or strain poles presenting a safety hazard to passing motorists and/or pedestrians due to extensively damaged, split or buckled sections, or with cracked welds at either pole/base connections or

member/member connections.

- Sign, traffic signal, or strain pole 4-bolt base plate connections with one or more loose nuts presenting a safety hazard to passing motorists and/or pedestrians.
- Signs with deteriorated or missing panel connectors, allowing sign to "flop" under wind loading that present a safety hazard to passing motorists and/or pedestrians.
- Lighting fixtures with split sections, buckled sections, significant section loss, and/or cracked welds at the pole/base connection that present a safety hazard to passing motorists and/or pedestrians.

Other

The following deficiencies represent other examples of critical findings:

- Expansion joints that are deteriorated, damaged, or loose which may present a safety hazard to passing traffic.
- Rocker bearings that are critically tilted either exceeding the acceptable amount of tilt or bearing on the outer one-quarter width of the rocker.
- Excessive debris and/or sediment buildup at the hydraulic opening for scour critical bridges or other bridges with unknown foundations.

4.5.4 Example Plans of Action

As previously mentioned, a statewide or Federal agency wide procedure must be established to assure that critical findings are addressed in a timely manner. The appropriate actions to be used for repair or mitigation of the critical finding must be quickly identified and efficiently carried out. The FHWA must be periodically notified of the actions that have been taken to resolve or monitor critical findings. It is the responsibility of Bridge Owners to implement procedures for addressing critical deficiencies including:

- Immediate critical deficiency reporting steps
- Emergency notification of police and the public
- Rapid evaluation of the deficiencies
- Rapid implementation of corrective or protective actions
- A tracking system to ensure adequate follow-up
- Provisions for identifying other bridges with similar structural details for follow-up inspections

Some agencies have very strict timeframes (3 to 7 *calendar* days) for developing and accepting plans of action. For circumstances involving immediate attention or a more detailed solution, it may be necessary to begin addressing the critical finding (through permanent or temporary work) prior to the 100% development and acceptance of the plan of action. Example plans of action are given below for Pennsylvania DOT (Figure 4.5.3) and Washington State DOT (Figure 4.5.4).

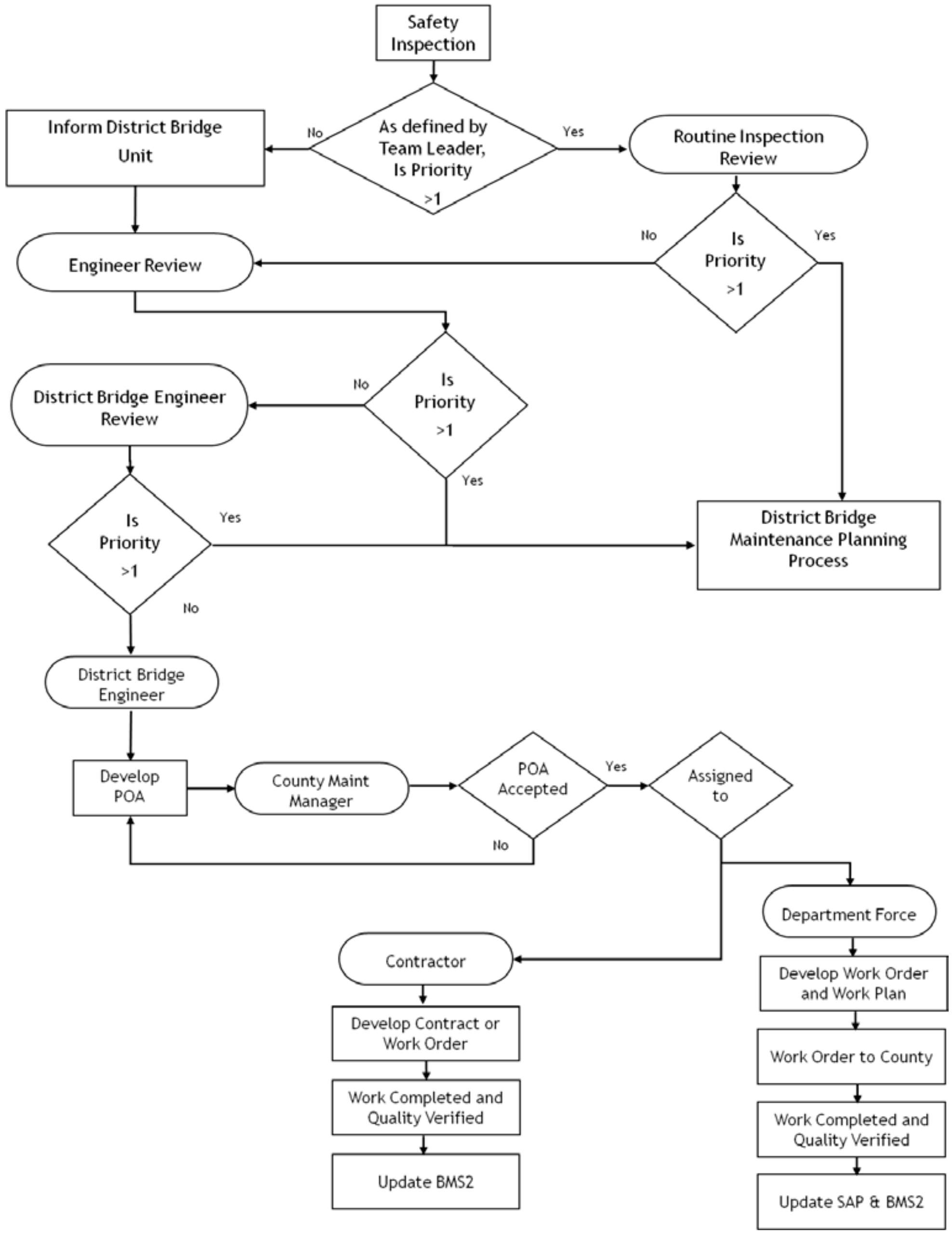

Figure 4.5.3 Pennsylvania DOT Critical and High Priority Maintenance Items – Flowchart for Plan of Action

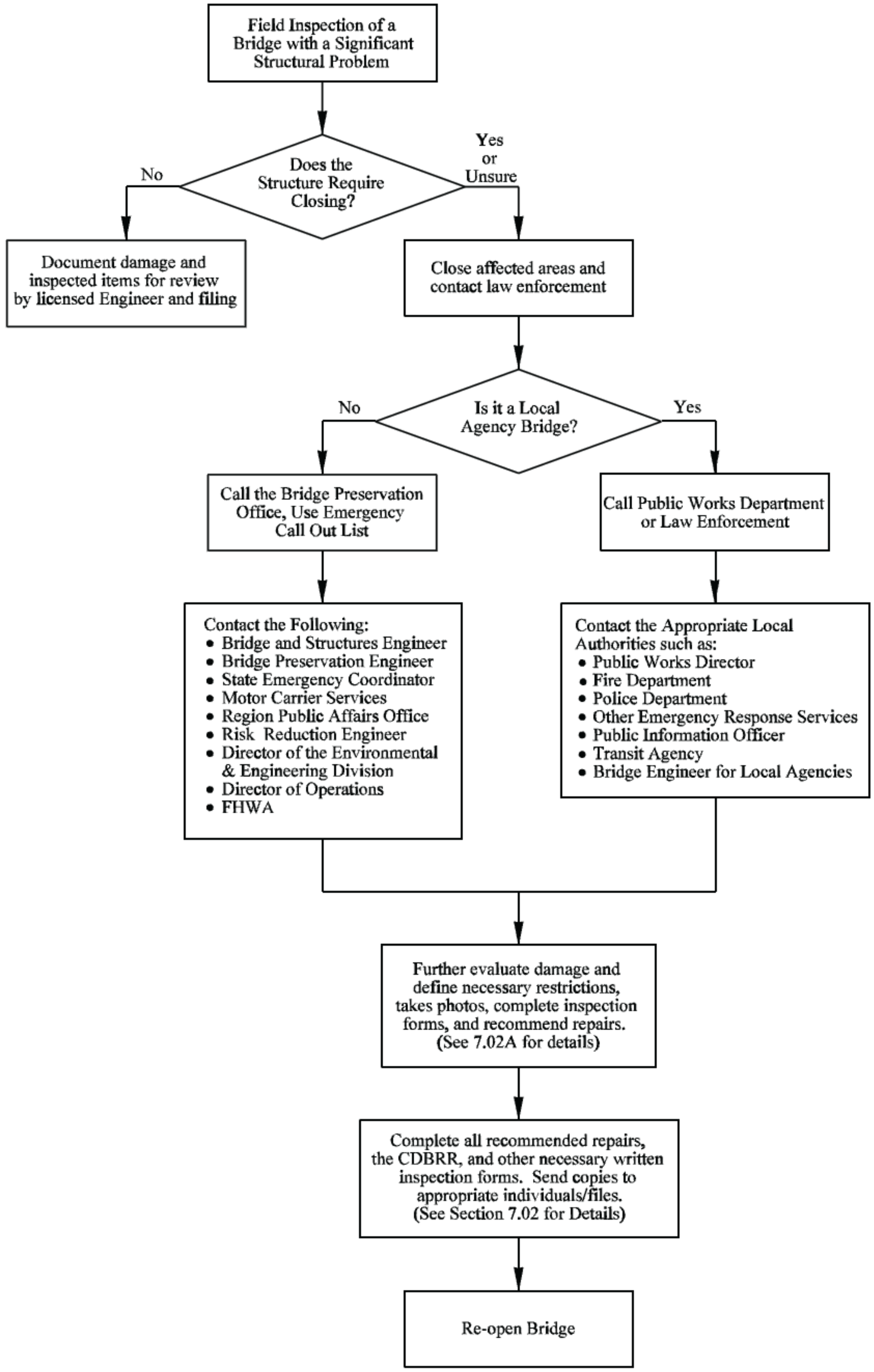

Figure 4.5.4 Washington State DOT Flowchart for Field Inspection Procedure

After the plan of action has been accepted, recommended repair work will then be performed and completed within a few days up to several weeks, depending on the individual agency's regulations. A post-repair report will be generated documenting all necessary work done to address the critical finding and the date of completion. A follow-up inspection will also be conducted to assess the condition of the repairs. The FHWA will be notified of the repair and post-repair progress.

Table of Contents

Chapter 4
Bridge Inspection Reporting

Topic 4.6 The Inspection Report

4.6.1 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 represents 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 documents all signs of distress and deterioration with sufficient precision so that future inspectors can readily make a comparison of condition. Bridge owners normally set the format to be used when preparing a bridge inspection report. A complete inspection report contains several parts, as outlined in this topic. A sample bridge inspection report is presented in Appendix A. Inspection reports are 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, reports are still generated for each type of bridge inspection. Some bridge owners also request a special bridge inspection and report when planning a major rehabilitation.

4.6.2 Basic Components of a Comprehensive In-Depth Bridge Inspection Report

Table of Contents

The table of contents presents 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 is normally included with a scale large enough to positively locate the structure. The bridge is clearly marked and labeled, and the map has a north arrow to aid with orientation. Some agencies may choose to use GPS coordinates or latitude/ longitude descriptions.

Bridge Description and History

The bridge description and history section of the report contains all pertinent data concerning the design, construction, and service of the bridge. The type of superstructure will generally be given first, followed by the type of abutments and piers or bents, along with their foundations. If data is available, indicate the type of foundation soil, maximum bearing pressures, and deep foundation capacities. The type of deck is also indicated.

The history of the bridge is from a structural standpoint and is developed from information obtained from design, construction and rehabilitation plans, previous inspection reports, maintenance records, discussions with maintenance crews and local residents, and any other available source that offers pertinent information. Typical items included in the history narrative are:

- Historical flood frequencies and high water marks
- Maintenance measures and repairs
- Chronological record of conditions (in order to help determine a rate of deterioration of all bridge components and the channel). The agency establishes criteria for the number of bridge inspections kept on file.
- Reference drawings
- Photos, which would consist of a typical approach photograph showing the approach roadway, bridge and any load restriction signs, as well as an elevation/profile photograph showing upstream/downstream of the bridge. Other photographs, such as those conveying the condition of the bridge and its components, would be found in the Appendix or in the Inspection Results section of the inspection report.

Design Data

The design information includes a description of the following:

- Skew angle
- Number and length of spans
- Span type and material
- Total length
- Bridge width
- Deck structure type
- Wearing surface
- Deck protection and membrane
- Sidewalks
- Railing and median
- 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 includes 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 “design”, “as-built”, or “rehabilitation” drawings.

Service Data

The average daily traffic (ADT) count and the average daily truck traffic (ADTT) count are included, along with the date of record. This information is updated approximately every five years. Other service data to consider includes the year of ADT and ADTT, facility carried, functional classification, and bypass detour length and map. In addition, environmental conditions that may have an effect on the bridge, such as salt spray, industrial gases, bird droppings, and ship and railroad traffic, are noted in the report.

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. A typical executive summary identifies the bridge (e.g., name, number, and location) and the date of inspection. The executive summary presents any high priority repair items.

Inspection Procedures

The procedures used to inspect the bridge are documented in the inspection report. 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 for that sequence.

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 is typically 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)
- Permits required (e.g., railroad and Coast Guard)
- Inspection methods (e.g., visual, physical or advanced)
- 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, cross sections and scour criticality

When structure plans are not in the bridge records and a load rating has not been calculated, it may be necessary to obtain field measurements to assist in the calculation of the load capacity of the structure.

Inspection Results

Provide narrative descriptions of the conditions both quantitative and qualitative, indicating the locations and the extent of the affected areas. Use agency-approved 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 preservation strategy. Take photographs in the field to show deficiencies and cross reference in the report or on forms where deficiencies are noted. Supplement written notes with sketches and photos to show location and physical characteristics of deficiencies, including a known object in the photograph for scale reference.

Note any load, speed, or traffic restrictions on the bridge. Indicate if the signs are missing or damaged. Take approach roadway photograph to confirm placement of load posting signs that includes the approach roadway, bridge and sign. Check for advanced warning signs. Include information about high water marks and unusual loadings. Note the weather conditions such as temperature, rain, or snow. Note all work or repairs to the bridge since last inspection. Verify or obtain new dimensions when improvement work has altered the structure. New streambed profiles and cross sections are taken to detect scour, channel migration, or channel aggradation and degradation. Note any channel restrictions (e.g. debris) that could impact stream flow and increase scour potential. State the seriousness and amount of all deficiencies at the bridge site.

Load Rating Summary

A summary of any load capacity rating analysis that has been performed is included in the report. The summary is presented in a table or chart. Governing load ratings are shown for both inventory and operating levels for all types of loadings used in the analysis. Identify the governing member for each rating. 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 three in Bay five 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 is reported, and it would be identified as the governing member.

Conclusions and Recommendations

A good inspection report explains in detail the type, severity and extent of any deficiency found on the bridge and points out any deviations or modifications that are contrary to the "as-built" construction plans. The depth of the report is consistent with the importance of the deficiencies. Not all deficiencies 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's experience and judgment are called upon when interpreting inspection results and arriving at reasonable and practical conclusions. 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 deficiencies occurred. Seek advice from more experienced personnel when you cannot confidently interpret the inspection findings.

The recommendations made by the inspector constitute the "focal point" of the operation of inspecting, recording, and reporting. The inspector reviews previous inspection recommendations and identifies any recommendations 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 deficiencies.

All recommendations for preservation work, load rating, postings, and further inspection are included in this portion of the inspection report. Carefully consider the benefits to be derived from completing recommended work and the consequences if the work is not completed. List, in order of greatest urgency, any work that is necessary to maintain structural integrity and public safety. Recommendations concerning work are typically classified between three to five distinct prioritization levels, which range from the most severe or significant (critical) to a maintenance item that is considered routine or may only require monitoring (non-critical). The specific prioritization levels are set forth by each bridge-owning agency. Examples of agency priority maintenance procedures are listed in Topic 4.5.2.

The inspector decides whether a deficiency is a critical finding and needs immediate action using agency procedures. 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 immediate action is in order. Communicate the critical finding immediately and document actions taken in the report. By contrast, a slightly deteriorated bridge bearing may not be critical. A condition such as this would appropriately call for a recommendation for a preservation action.

Typically, most work recommendations submitted by the bridge inspector will be in the category of non-critical work. The recommended work is carefully described in the report along with a cost estimate.

If not already described in the executive summary, the conclusions and recommendations section of the report summarizes 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, not the inspector in the field, to prepare recommendations and cost estimates.

Report Appendices

To achieve maximum effectiveness of the inspection report, the report appendices contain any back-up information used to substantiate the inspector's findings, conclusions and recommendations. Typically, the appendices include photographs, drawings and sketches, and inspection forms (see Topic 4.4 for record keeping and documentation). Appendices may also include copies of any field notes used and specialist reports (e.g., underwater, nondestructive evaluation (NDE), and survey), or these documents may be referenced in the report. A load capacity rating analysis of the structure may also be incorporated into the report appendices. It is important to have the inspection report and all supplemental information, including report appendices, accurate with clear and concise descriptions or explanations.

Photographs

Photographs are a great asset to anyone reviewing reports on bridge structures. It is recommended that pictures be taken of any problem areas. Take pictures even if you think you can explain it completely in writing. It is better to take several photographs that may be considered unessential than to omit a photograph that could cause misinterpretation or misunderstanding of the report. At least two general photographs of every structure are provided in the appendix. One of these depicts the structure from the roadway, while the other photo is a view of the side elevation (see Figures 4.6.1 and 4.6.2). Captions are provided for each photograph. Photographs are numbered so that they can be referred to in the body of the report. Sketches may also be a substitute for missing as-built plans.

Figure 4.6.1 Near Approach - Toward Bridge

Figure 4.6.2 Downstream Elevation

Drawings and Sketches

Sketches and drawings needed to illustrate and clarify conditions of structural elements or serve as as-built plans are included or referenced. Sketches may be able to convey information not readily identified in a photograph (ie. remaining web thickness). Original drawings are very helpful during future investigations with 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 and bridge site, may be included as an appendix.

Some reports combine photographs and sketches or text boxes together to accurately describe and document a particular deficiency.

Inspection Forms

The inspection forms contain the actual field notes, as well as the numerical condition and appraisal ratings by the inspector. The inspection forms are normally signed by the inspection team leader. A complete SI&A form or equivalent is included in the appendix. Compare previous inspection forms to current conditions for inventory data accuracy.

Load Capacity Analysis

A load rating analysis is performed on the structure to determine the load-carrying capacity of the bridge. It includes the investigation of 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 a new load rating analysis. A new load rating analysis is performed if the condition of the primary members has changed considerably since the last inspection. The report also includes recommendations for a new load rating analysis when maintenance or improvement work, change in strength of members, or dead load has altered the condition or capacity of the structure.

Field Inspection Notes

Include the original notes taken by the inspectors in the field or photocopies thereof in the appendix section of the report. The original field notes are source documents and as such are typically included in the bridge record.

Underwater Inspection Report

If an underwater inspection of the substructure has been performed, a separate report is usually prepared by the dive team. If applicable, include the underwater inspection report in the appendix or cross-reference the location of the report.

Material Testing Results

Material testing may be performed on a structure in order to determine the strength and properties of an unknown or suspect material. Include the testing lab's report in the appendix.

4.6.3 Basic Components of a Comprehensive Routine Inspection Report

Location Map

A map with a scale may be included to help positively locate the structure. Some agencies may choose to use GPS coordinates or latitude/longitude descriptions to supplement or replace the location map.

Inspection Procedures

The procedures used to inspect the bridge may be documented in the inspection report. For inspection reports that include the inspection procedures, 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).

As with in-depth inspections, some routine inspections cannot follow this sequence due to traffic and lane-closure restrictions. Therefore, 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 is typically 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)
- Permits required (e.g., railroad and Coast Guard)
- Inspection methods (e.g., visual, physical or advanced)
- 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, cross sections and scour criticality

When structure plans are not in the bridge records and a load rating has not been calculated, it may be necessary to obtain field measurements to assist in the calculation of the load capacity of the structure.

Inspection Results

The results of the inspection are documented within the inspection forms. Narrative descriptions of the conditions are typically not included for routine inspection reports. As with in-depth inspections, use agency-approved 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 preservation strategy. Take photographs in the field to show deficiencies and cross reference in the report or on forms where deficiencies are noted. Supplement written notes with sketches and photos to show location and physical characteristics of deficiencies, including a known object in the photograph for scale reference.

Note any load, speed, or traffic restrictions on the bridge. Indicate if the signs are missing or damaged. Take approach roadway photograph to confirm placement of load posting signs that includes the approach roadway, bridge and sign. Check for advanced warning signs. Include information about high water marks and unusual loadings. Note the weather conditions such as temperature, rain, or snow. Note all work or repairs to the bridge since last inspection. Verify or obtain new dimensions when improvement work has altered the structure. New streambed profiles and cross sections are taken to detect scour, channel migration, or channel aggradation and degradation. Note any channel restrictions (e.g. debris) that could impact stream flow and increase scour potential. State the seriousness and amount of all deficiencies at the bridge site.

Load Rating Summary

For routine inspections, a load rating may be conducted. If performed, a load rating summary is included in the report and may also be included on the inspection forms. The summary is presented in a table or chart. Governing load ratings are shown for both inventory and operating levels for all types of loadings used in the analysis. Identify the governing member for each rating. The governing member is the one that has the lowest capacity for a given type of loading.

Conclusions

A routine inspection report may or may not contain conclusions of the inspection. If conclusions are included, explain in detail the type, severity and extent of any deficiency found on the bridge and point out any deviations or modifications that are contrary to the “as-built” construction plans. The depth of the report is consistent with the importance of the deficiencies. Not all deficiencies are of equal importance.

The inspector’s experience and judgment are called upon when interpreting inspection results and arriving at reasonable and practical conclusions. 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 deficiencies occurred. Seek advice from more experienced personnel when you cannot confidently interpret the inspection findings.

Recommendations

Recommendations are made by the inspector that constitutes the “focal point” of the operation of inspecting, recording, and reporting. The inspector reviews previous inspection recommendations and identifies any recommendations 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 deficiencies.

All recommendations for preservation work, load rating, postings, and further inspection are included in this portion of the inspection report. Carefully consider the benefits to be derived from completing recommended work and the consequences if the work is not completed. List, in order of greatest urgency, any work that is necessary to maintain structural integrity and public safety. Recommendations concerning work are typically classified between three to five distinct prioritization levels, which range from the most severe or significant (critical) to a maintenance item that is considered routine or may only require monitoring (non-critical). The specific prioritization levels are set forth by each bridge-owning agency. Examples of agency priority maintenance procedures are listed in Topic 4.5.2.

The inspector decides whether a deficiency is a critical finding and needs immediate action using agency procedures. 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 immediate action is in order. Communicate the critical finding immediately and document actions taken in the report. By contrast, a slightly deteriorated bridge bearing may not be critical. A condition such as this would appropriately call for a recommendation for a preservation action.

Typically, most work recommendations submitted by the bridge inspector will be in the category of non-critical work. The recommended work is carefully described in the report along with a cost estimate.

The recommendations section of the report summarizes the following:

- Further inspection
- Maintenance
- Repairs
- Painting
- Posting
- Rehabilitation
- Replacement

Some state and local agencies designate separate personnel, not the inspector in the field, to prepare recommendations and cost estimates.

Report Appendices

To achieve maximum effectiveness of the inspection report, the report appendices contain any back-up information used to substantiate the inspector's findings, conclusions (if included) and recommendations. Typically, the appendices include photographs, drawings and sketches, and inspection forms. See Topic 4.4 for record keeping and documentation. Note that for routine inspections, inspection forms comprise the report, itself. Appendices may also include copies of any field notes used and specialist reports (e.g., underwater, nondestructive evaluation (NDE), and survey), or these documents may be referenced in the report. Although typically not conducted for routine inspections, a load capacity rating analysis of the structure may also be incorporated into the report appendices if performed. It is important to have the inspection report and all supplemental information, including report appendices, accurate with clear and concise descriptions or explanations.

Photographs

Photographs are a great asset to anyone reviewing reports on bridge structures. It is recommended that pictures be taken of any problem areas. Take pictures even if you think you can explain it completely in writing. It is better to take several photographs that may be considered unessential than to omit a photograph that could cause misinterpretation or misunderstanding of the report. At least two general photographs of every structure are provided in the appendix. One of these depicts the structure from the roadway, while the other photo is a view of the side elevation (see Figures 4.6.1 and 4.6.2). Captions are provided for each photograph. Photographs are numbered so that they can be referred to in the body of the report. Sketches may also be a substitute for missing as-built plans.

Drawings and Sketches

Sketches and drawings needed to illustrate and clarify conditions of structural elements or serve as as-built plans are included or referenced. Sketches may be able to convey information not readily identified in a photograph (i.e., remaining web thickness). Original drawings are very helpful during future investigations with 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 and bridge site, may be included as an appendix.

Some reports combine photographs and sketches or text boxes together to accurately describe and document a particular deficiency.

Inspection Forms

The inspection forms comprise the actual routine inspection report and contain the field notes, as well as the numerical condition and appraisal ratings by the inspector. The inspection forms are normally signed by the inspection team leader. A complete SI&A form or equivalent is included in the appendix. Compare previous inspection forms to current conditions for inventory data accuracy.

Load Capacity Analysis

A load rating analysis may or may not be performed on the structure to determine the load-carrying capacity of the bridge. For routine inspections without a load capacity analysis, the results of the previous load capacity analysis are typically included in the report. If a load capacity analysis is performed, it is normally performed by engineers in the office, not by the inspector, and represents an investigation of primary load-carrying members of the bridge. A new load rating analysis is performed if the condition of the primary members has changed considerably since the last inspection. The report also includes recommendations for a new load rating analysis when maintenance or improvement work, change in strength of members, or dead load has altered the condition or capacity of the structure.

Field Inspection Notes

Include the original notes taken by the inspectors in the field or photocopies thereof in the appendix section of the report. The original field notes are source documents and as such are typically included in the bridge record.

Underwater Inspection Report

If an underwater inspection of the substructure has been performed, the summary of findings of the underwater inspection report (typically prepared by the dive team) is usually included in the appendix or cross-referenced to another location of the report.

4.6.4 Importance of the Inspection Report

Source of Information

A well-prepared report will not only provide information on existing bridge and bridge site 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 are reported in a factual manner, avoiding speculation. Terms such as "hazardous" or "dangerous" are subjective and are not used in the inspection report or inspection documentation that may be included in the appendix. Further action on such reports will be determined after review and consultation by experienced personnel.

Legal Document

In preparing an inspection 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 future litigation. The language used in reports needs to be clear and concise and, in the interest of uniformity, care needs to 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 needs to be clearly stated.

Some state agencies require inspection reports to be signed, dated and sealed by a professional engineer before accepting them. Other state agencies require inspection reports to be signed and dated by the inspection team leader. The AASHTO *MBE* states (per Article 2.2) that "the components of data entered in a bridge record should be dated and include the signature of the individual responsible for the data presented." No undocumented alterations are allowed to the report once it is accepted. Some inspectors retain copies of their reports for their personal files in the interest of self-protection if there is any litigation.

Critical Findings

Critical findings are documented in the inspection report. However, the inspection report does not provide guidance for the follow-up to critical findings - the inspector does not wait for the inspection report to communicate and take action on critical findings. Instead, the follow-up to critical findings is a separate procedure that is immediately communicated with action taken on the critical findings, in accordance with the requirements of the NBIS. Agency procedures are established to assure that critical findings are addressed in a timely manner. In many instances when the critical finding exists, a plan of action is established and the deficiency is addressed prior to the formal submittal of the inspection report.

The FHWA is periodically notified of the actions taken to resolve or monitor critical findings. Advanced inspection methods for one or more elements may be recommended. The report provides information which may lead to decisions to limit the use of a bridge or close it to traffic; any bridge which the inspection has revealed to be a potential public safety concern.

Maintenance

Another purpose of the inspection report is to provide useful information about the needs and effectiveness of preservation activities. An active preservation program is vital to the long-term structural integrity of a bridge. The inspection report enables bridge preservation to be programmed more effectively through early detection of structural deficiencies, therefore minimizing more costly future work and inconvenience to the traveling public.

Load Rating Analysis

When an inspection reveals deficiencies that may affect the load-carrying capacity of the structure, the findings need to be reviewed by an engineer to determine if a revised load rating analysis is required. A new load rating analysis is performed 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 revised load-carrying capacity (load rating) analysis become part of the bridge record.

Bridge Management

Another purpose of the inspection report is analysis by the bridge owners and the FHWA of the SI&A data. The intent of the analysis is to aid in the decisions for allocating and prioritizing funding.

Another important purpose of the inspection report is the data the report provides for use by the owner in managing the bridge asset. The data provided in the inspection report is important for the identification, prioritization, budgeting and programming of bridge preservation, improvement and replacement work. On a national level the data is used for reporting to Congress on the condition and performance of the Nation's bridges and for determining current and future estimates of funding needs. Furthermore, the data is used to: classify bridges according to serviceability, safety, and essentiality for public use; assign each a priority for replacement or rehabilitation; and determine the cost of replacing each such bridge with a comparable facility or of rehabilitating such bridge.

4.6.5 Quality

The accuracy and uniformity of information collected and recorded is vital for the management of an owner's bridges for preservation, improvement and replacement, and, most importantly, public safety. Quality cannot be taken for granted. The responsibility of ensuring quality bridge inspections rests with each bridge owner and the inspection team. Two phrases are frequently used when discussing quality; they are quality control and quality assurance.

NBIS regulations require each state to assure that systematic quality control (QC) and quality assurance (QA) procedures are being used to maintain a high degree of accuracy and consistency in the inspection program. Include periodic field review of inspection teams, periodic bridge inspection refresher training for program managers and team leaders, and independent review of inspection reports and computations.

The AASHTO *MBE* provides guidance for the implementation of appropriate quality control and quality assurance procedures. Quality control procedures include the "use of checklists to ensure uniformity and completeness, the review of reports and computations by a person other than the originating individual, and the periodic field review of inspection teams and their work." Quality assurance procedures include the "overall review of the inspection and rating program to ascertain that the results meet or exceed the standards established" by the bridge-owning agency.

Follow state-wide or agency-wide QC/QA procedures for a higher degree of accuracy and consistency in the inspection program.

See Topic 1.3 for a detailed description of quality control and quality assurance.

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Chapter 5
Bridge Mechanics

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Basic Equations of Bridge Mechanics

$$f_a = \frac{P}{A}$$ (Page 5.1.11)

$$f_b = \frac{Mc}{I}$$ (Page 5.1.13)

$$f_v = \frac{V}{A_w}$$ (Page 5.1.14)

$$\sigma = \frac{F}{A}$$ (Page 5.1.16)

$$\varepsilon = \frac{\Delta L}{L}$$ (Page 5.1.17)

$$E = \frac{\sigma}{\varepsilon}$$ (Page 5.1.18)

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

Common units:
p = pounds
in = inches
ft = feet = 12 inches
k = kip = 1000 pounds
psi = pounds per square inch
ksi = kips per square inch

$$Bridge\,Rating\,Factor\,(RF) = \frac{C - A_1 D}{A_2 L(1+I)}$$ (Page 5.1.23)

$$Bridge\,Rating\,Factor\,(RF) = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$ (Page 5.1.23)

Chapter 5
Bridge Mechanics

Topic 5.1 Bridge Mechanics

5.1.1 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 is primarily 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 or deterioration may have on the load-carrying capacity of a bridge component or element

While this topic 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.

5.1.2 Bridge Design Loadings

A bridge is designed to carry or resist design loadings 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 truck loads are normally 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 5.1.7.

Bridge design loadings can be divided into two principal categories:

- Permanent loads
- Transient loads

Permanent Loads

Permanent loads are loads and forces that are constant for the life of the structure. They consist of the weight of the materials used to build the bridge (see Figure 5.1.1). Permanent load includes both the self-weight of structural members and other permanent external loads. They do not move and do not change unless the bridge is modified. Permanent loads can be broken down into two groups, dead loads and earth loads.

Dead loads are a static load due to the weight of the structure itself. They include both the self-weight of the structural members and other permanent loads. Any feature may or may not contribute to the strength of the structure. Those features that may contribute to the strength of the structure include girders, floorbeams, trusses, and decks. Features that may not contribute to the strength of the bridge include median barriers, parapets, railings and utilities. Earth loads are permanent loads and are considered in the design of structures such as retaining walls and abutments. Earth pressure is a horizontal load which can be very large and it tends to cause abutments to slide and/or tilt forward. Earth surcharge is a vertical load that can increase the amount of horizontal load and is caused by the weight of the earth.

Example of self-weight: A 20-foot long beam weighs 50 pounds per linear foot. The total weight of the beam is 1000 pounds. This weight is called the self-weight of the beam.

Example of an external permanent 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 permanent load. The weight of the water line plus the self weight of the beam comprises the total permanent load.

Total permanent 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.

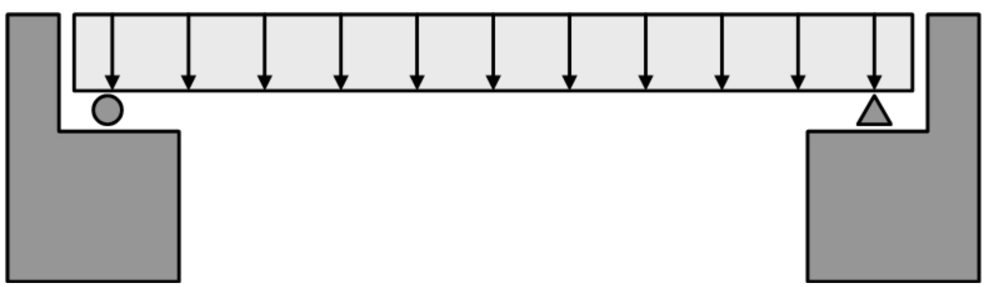


Figure 5.1.1 Permanent Load on a Bridge

Primary Transient Loads

A transient load is a temporary load and force that is applied to a structure which changes over time. In bridge applications, transient live loads are moving vehicular or pedestrian loads (see Figure 5.1.2). Standard AASHTO vehicle live loads do not represent actual vehicles, but it does provide a good approximation for bridge design and rating. AASHTO has designated standard pedestrian loads for design of sidewalks and other pedestrian structures.

To account for the affects of speed, vibration, and momentum, truck live loads are typically increased for vehicular dynamic load allowance. Vehicular dynamic load allowance is expressed as a percentage of the static truck live load effects.

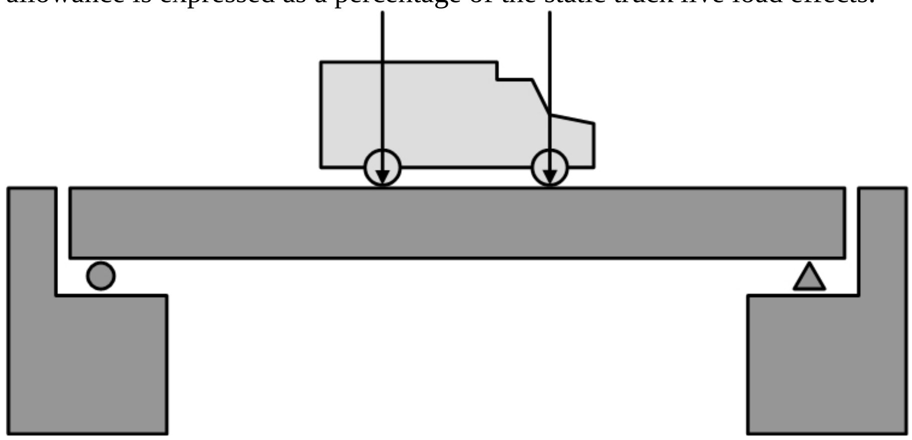


Figure 5.1.2 Vehicle Transient Load on a Bridge

AASHTO Truck Loadings

Standard vehicle live loads have been established by AASHTO for use in bridge design and rating. There are two basic types of standard truck loadings described in the current *AASHTO Specifications*. A third type of loading is used for AASHTO Load and Resistance Factor Design and Rating.

The first type is a single unit vehicle with two axles spaced at 14 feet and designated as a highway truck or "H" truck (see Figure 5.1.3). 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. 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*.

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 1935 truck loading used a train of trucks that imitated the railroad industry's standards.

As trucks grew heavier during World War II, AASHTO developed the new concept of hypothetical trucks. These fictitious trucks are used only for design and do not resemble any real truck on the road. The loading is now performed by placing one truck, per lane, per span. The truck is moved along the span to determine the point where it produces the maximum shear and moment. The current designation is H20-44 published in 1944.

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 5.1.4).

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 14 to 30 feet. The "HS" designation is followed by a number indicating the gross weight in tons of the tractor only.

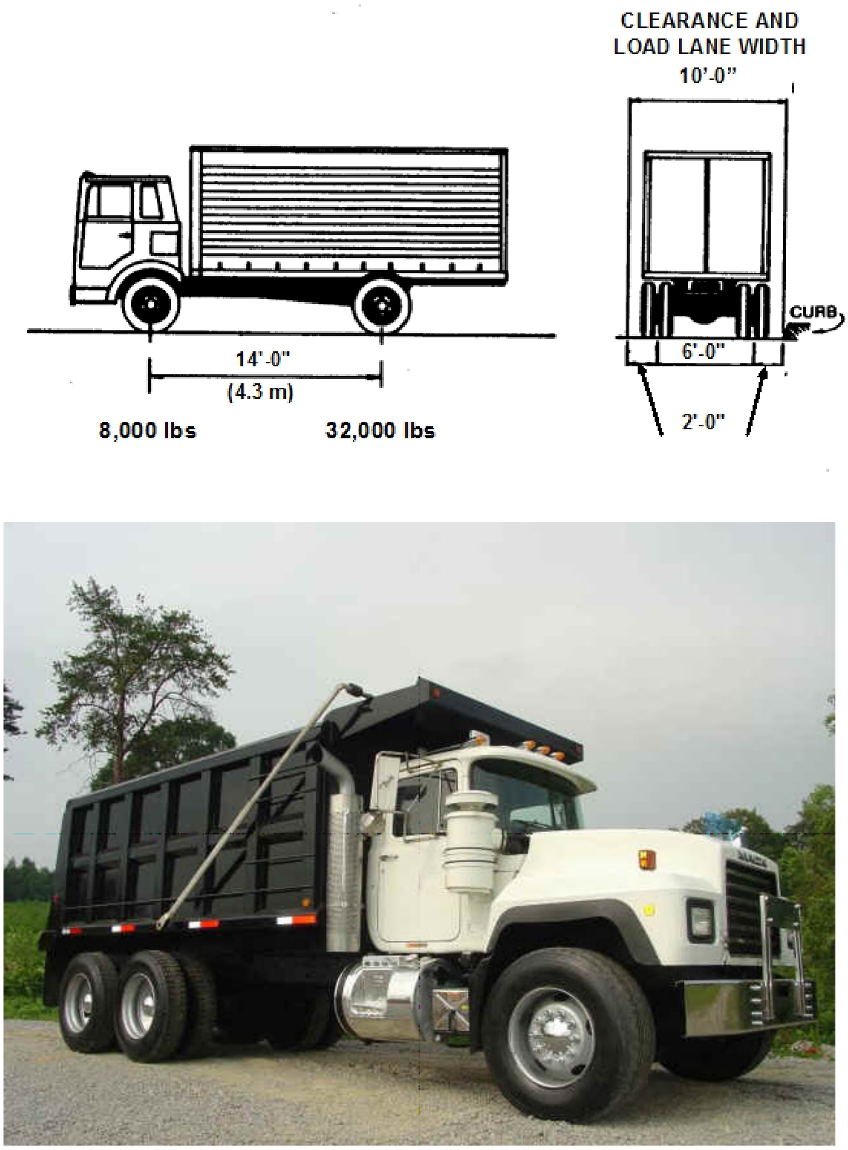


Figure 5.1.3 AASHTO H20 Truck

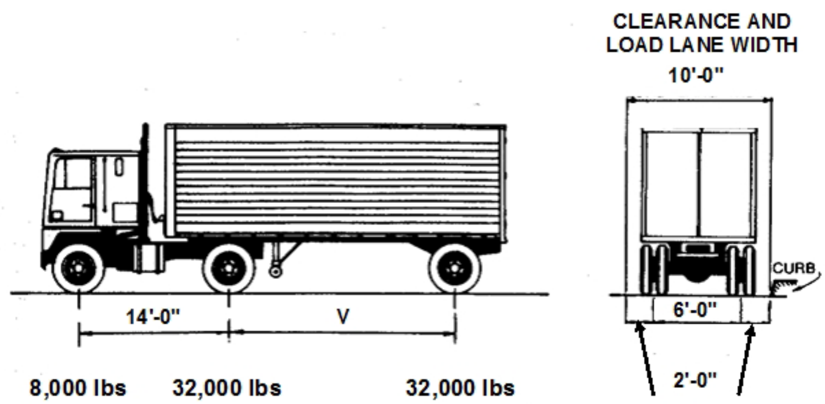


Figure 5.1.4 AASHTO HS20 Truck

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.

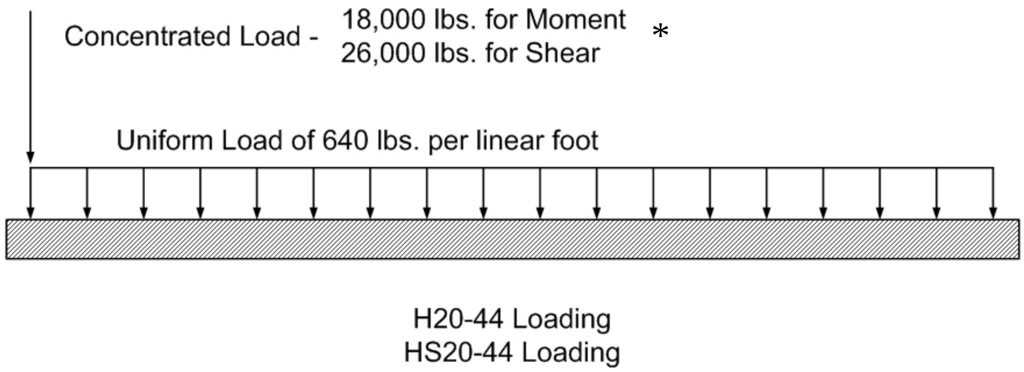
In specifications prior to 1944, a standard loading of H15 was used. In 1944, the policy of affixing the publication year of design loadings was adopted. In specifications prior to 1965, the HS20-44 loading was designated as H20-S16-44, with the S16 identifying the gross axle weight of the semi-trailer 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. These loads are used for the design of bridge members. Depending on such items as highway classification, truck usage and span classification, for example, an appropriate design load is chosen to determine the most economical member. Bridge posting is determined by performing a load rating analysis using the current member condition of an in-service bridge. Various rating methods will be discussed further in Topic 5.1.8.

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 truck load located on the span to produce the most critical situation in the structure (see Figure 5.1.5).

For design and load capacity rating analysis, make an investigation of both a truck loading and a lane loading 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.



* Use two concentrated loads for negative moment in continuous spans (Refer to *AASHTO LRFD Bridge Design Specifications 5th edition*, 2010 Interim; Article 3.6.1.2)

Figure 5.1.5 AASHTO Lane Loadings

LRFD Live Loads

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 is combined with the lane load and 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 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 0.64 kips per linear foot longitudinally and it is distributed uniformly over a 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. The HL-93 design load consists of a combination of the design truck or design tandem, and design lane load (see Figure 5.1.6).

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.

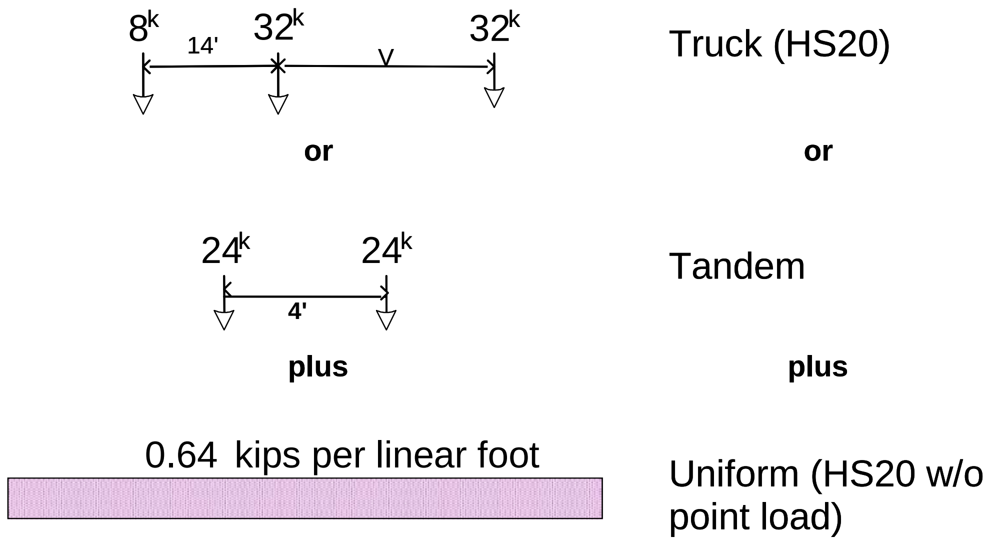


Figure 5.1.6 AASHTO LRFD Loading

Alternate Military Loading

The Alternate Military Loading is a single unit vehicle with two axles spaced at 4 feet and weighing 12 tons (or 24 kips) 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 whichever produces the greatest stress (see Figure 5.1.7).

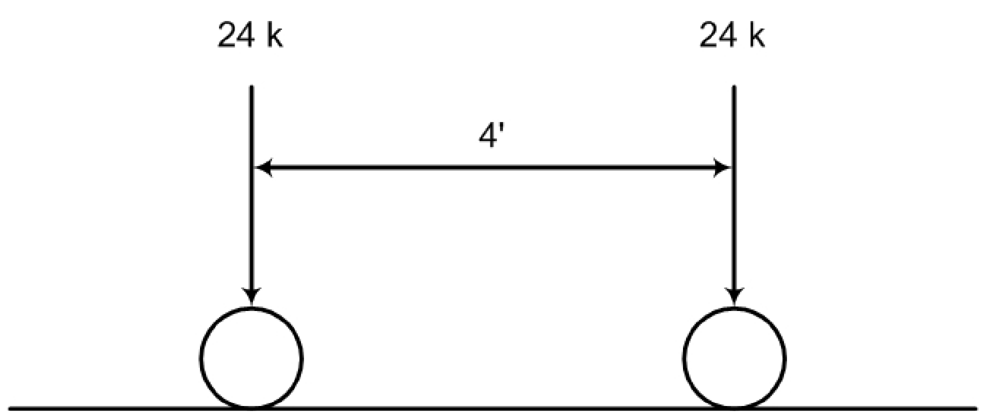


Figure 5.1.7 Alternate Military Loading

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 weights and 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 5.1.8).

Figure 5.1.8 Permit Vehicle

Secondary Transient Loads

In bridge applications, the transient loads are temporary dynamic loads and can consist of the following:

- **Vehicular braking force** - a force in the direction of the bridge caused by braking of live load vehicles
- **Vehicular centrifugal force -** an outward force that a live load vehicle exerts on a curved bridge
- **Vehicular collision force** – the force caused by the collision of a vehicle into either the superstructure or substructure of a bridge
- **Vessel collision force** – the force caused by the collision of a water vessel into either the superstructure or substructure of a bridge
- **Earthquake load -** bridge structures are built so that motion during an earthquake will not cause a collapse
- **Friction load** – a force that is due to friction based upon the friction coefficient between the sliding surfaces
- **Ice load** - a horizontal force created by static or floating ice jammed against bridge components
- **Vehicular dynamic load allowance** – loads that account for vibrations and resonance between bridge, live load, and vibrations due to surface discontinuities (i.e. deck joints, potholes, cracks)
- **Vehicular live load** – AASHTO standard live loads placed upon the bridge due to vehicles
- **Live load surcharge** – a load where vehicular live load is expected on the surface of backfill within a distance to one-half the wall height behind the back face of the wall
- **Pedestrian live load** – AASHTO standard live load placed upon a bridge due to pedestrians which include sidewalks and other structures
- **Forces effect due to settlement -** a horizontal force acting on earth-retaining substructure units, such as abutments and retaining walls
- **Temperature -** since materials expand as temperature increases and contract as temperature decreases, the force caused by these dimensional changes must be considered
- **Water load -** a horizontal force acting on bridge components constructed in flowing water
- **Wind load on live load -** wind effects transferred through the live load vehicles crossing the bridge
- **Wind load on structure -** wind pressure on the exposed area of a bridge

A bridge may be subjected to several of these loads simultaneously. AASHTO LRFD *Specifications* have established a table of Load Combination Limit States. For each Limit State, a set of load combinations are considered with a load factor to be applied to each particular load.

5.1.3 Bridge Response to Loadings

Each member of a bridge is intended to respond to loads in a particular way. It is important to 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, the inspector 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 parallel to the longitudinal axis of a member. An axial force causes compression if it is pushing and tension if it is pulling (see Figure 5.1.9). Axial forces are generally expressed in English units of pounds or kips.

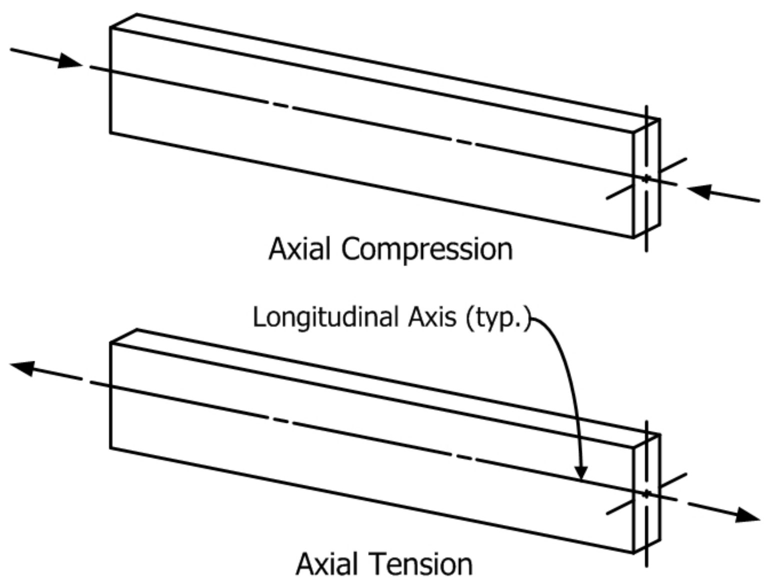


Figure 5.1.9 Axial Forces

Examples of axial forces: 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 either compression or tension forces.. Cables are designed for axial forces in tension.

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 (kips per square inch)
	P	=	axial force (kips)
	A	=	cross-sectional area (square inches)

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 has to 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 when a load is applied perpendicular to the longitudinal or neutral axis. A moment is commonly developed by the perpendicular 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 can be positive or negative and produce both compression and tension forces at different locations in the member (see Figure 5.1.10). Moments are generally expressed in English units of pound-feet or kip-feet.

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.

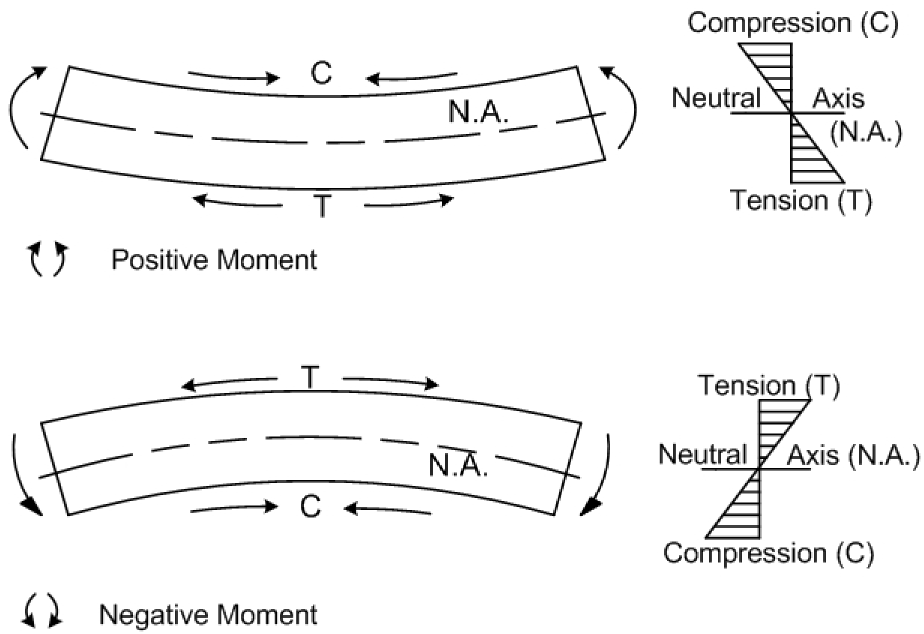


Figure 5.1.10 Positive and Negative Moment

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 5.1.11).

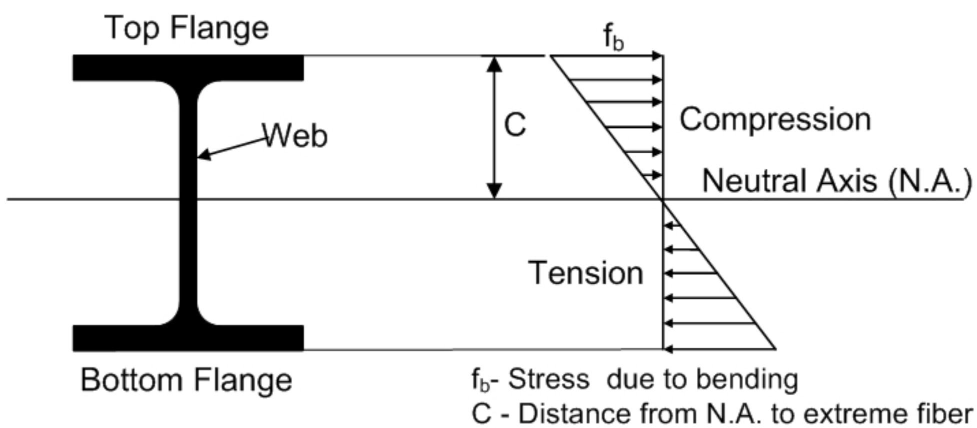


Figure 5.1.11 Girder Cross Section Resisting Positive Moment

Bending stress is normally considered zero at the neutral axis. On a cross section of a member, bending stresses vary linearly with respect to the distance from the neutral axis (see Figures 5.1.10 and 5.1.11).

The formula for maximum bending stress is (see Figure 5.1.11):

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

where:

f_b = bending stress on extreme fiber (or surface) of beam (kips per square inch)

M = applied moment (inch · lbf)

c = distance from neutral axis to extreme fiber (or surface) of beam (inches)

I = moment of inertia (a property of the beam cross-sectional area and shape) (lbf · square inch)

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 5.1.12). Shear forces are generally expressed in English units of pounds or kips.

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.

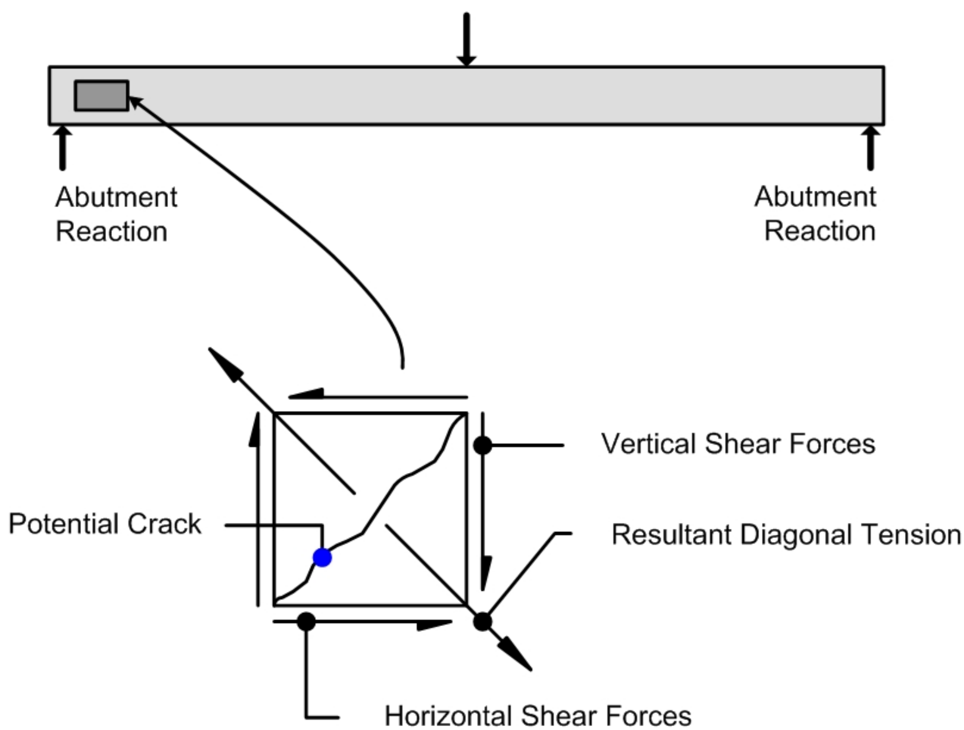


Figure 5.1.12 Shear Forces in a Member Element

Beams and girders are common shear resisting members. In an I- or T-beam, most of the shear is resisted by the web (see Figures 5.1.11 and 5.1.12). 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. The horizontal shear forces are required to keep the member in equilibrium (not moving). 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 (kips per square inch)
V = vertical shear due to external loads (kips)
A_w = area of web (square inches)

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.

Example of torsion: One end of a long rectangular 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 5.1.13).

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 5.1.14).

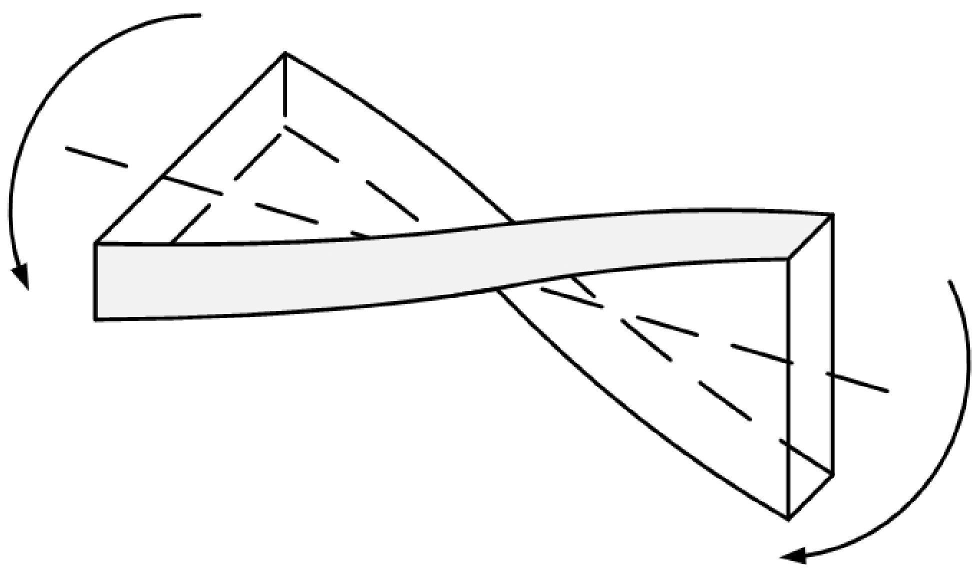


Figure 5.1.13 Torsion

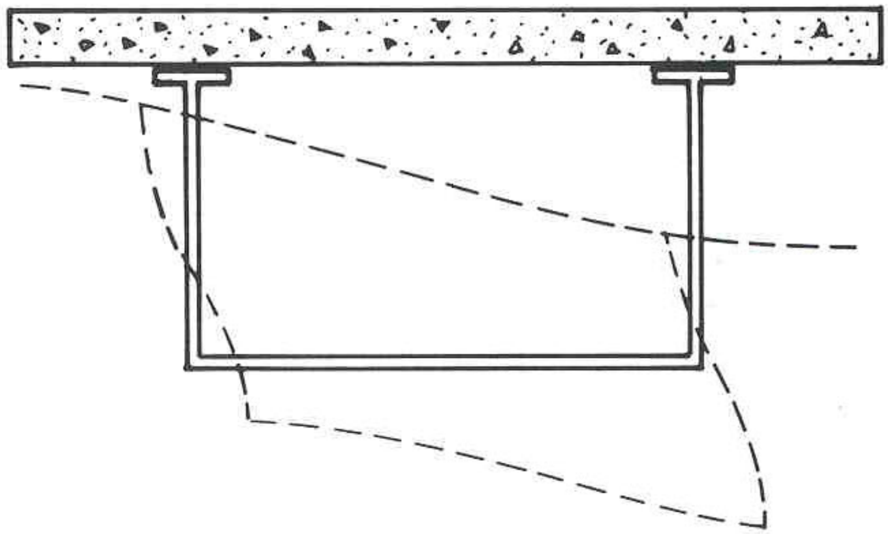


Figure 5.1.14 Torsional Distortion

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 5.1.15). 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 transmits 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.

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.

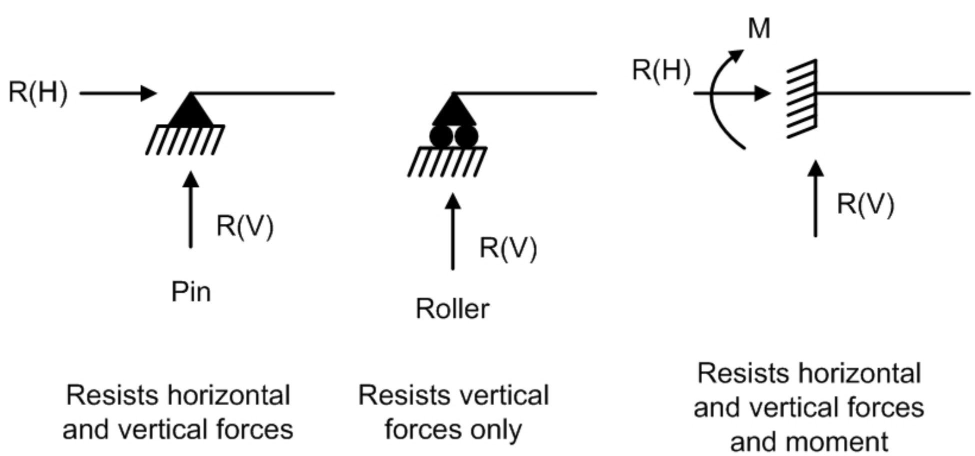


Figure 5.1.15 Types of Supports

The provided reactions at the bridge supports equal the applied permanent or transient loads. The equilibrium keeps the bridge in place.

5.1.4 Response to Loadings

Each bridge member 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 to be effective in their job.

Force

A force is the action that one body exerts on another body. Force has three aspects: magnitude, direction and point of application (see Figure 5.1.16). Every force can be divided into 3 distinct components or directions: vertical, transverse, and longitudinal. The combination of these three can produce a resultant force. Forces are generally expressed in English units of pounds or kips.

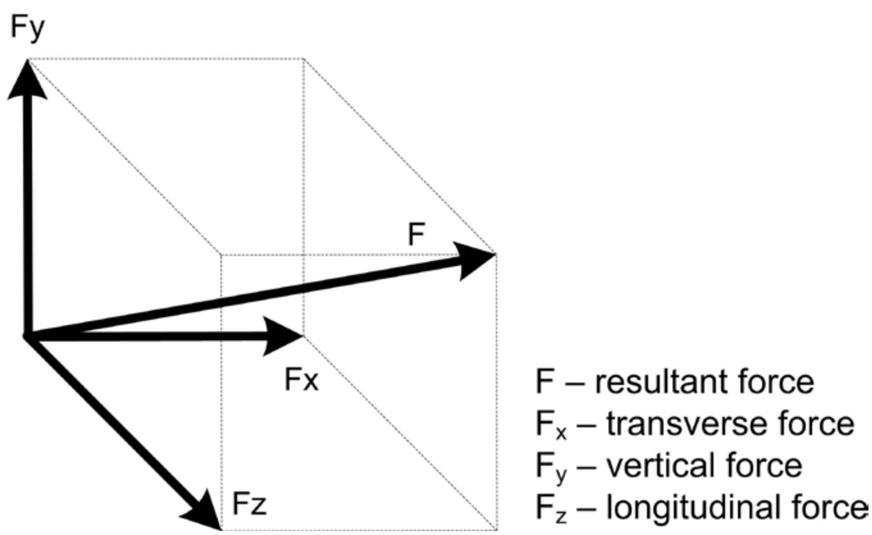


Figure 5.1.16 Basic Force Components

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.

$$Stress\,(\sigma) = \frac{Force\,(F)}{Area\,(A)}$$

The basic English unit of measure for 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. An allowable unit stress is generally established for a given material.

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).

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.

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

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 force 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%.

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 the 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 stress. 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 is 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 heavy paint cans remain left untouched on a thin wooden shelf for several months, the shelf will gradually deflect and change in shape. This deformation is due to the sustained application of a constant dead load 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 design for 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. Consider any thermal changes in these members since they can cause significant stresses.

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 5.1.17). 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.

$$Modulus\ of\ Elasticity\,(E) = \frac{Stress\,(\sigma)}{Strain\,(\varepsilon)}$$

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).

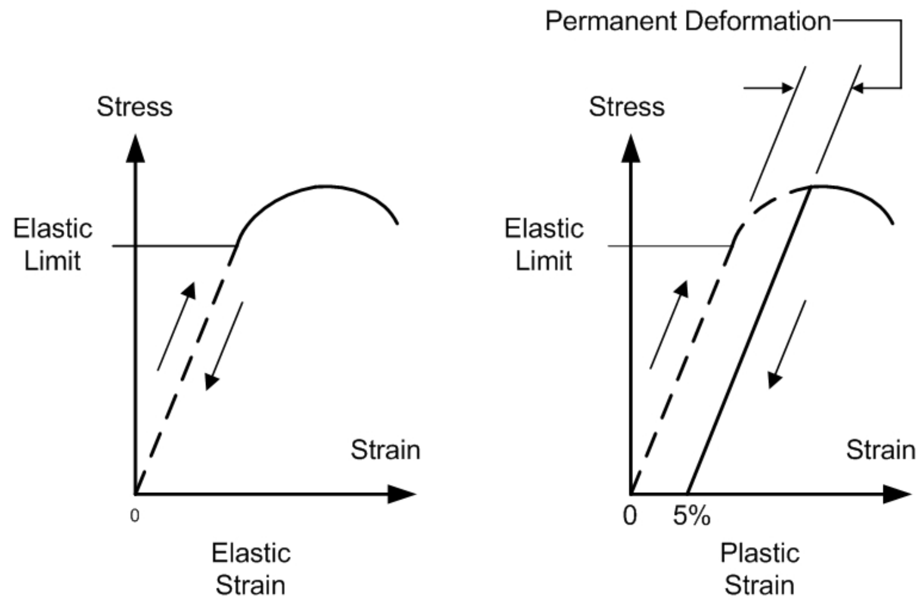


Figure 5.1.17 Stress-Strain Diagram

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

$$E = \frac{2{,}900\ psi}{0.0001 in/in} = 29{,}000{,}000\ psi = 29{,}000\, ksi$$

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 1600 ksi.

Overloads

Overload damage 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 crush 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, stretch or crack 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.

Critical Finding

An overload in a bridge member may be considered a critical finding. Critical findings are presented in Topic 4.5 and defined as "a structural or safety related deficiency that requires immediate follow-up inspection or action."

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
- Copper
- Aluminum
- 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
- Stone
- Cast iron
- 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.

Example of fatigue: If a rubber band is stretched and then allowed to return to its original position (elastic deformation), it is unlikely that the rubber band will break. However, if this action is repeated many times, the rubber band will eventually break. The rubber band failure is analogous to a fatigue failure.

For a description of fatigue categories for various steel details, refer to Topic 6.4.

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 6.2 and 6.3.

5.1.5 Mechanics of Materials

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

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 is ft-lbs @ degrees F.

5.1.6 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. Several factors control the amount of deflection: strength of material, depth and shape of structural member, and length of a member

In the absence of other criteria, the following limitations may be considered:

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.,1inch vertical movement per 67 feet of span length). For bridges that have sidewalks, AASHTO limits live load bridge deflection to 1/1000 (i.e., 1-inch vertical movement per 83 feet of span length).

Thermal Movements

The longitudinal expansion and contraction of a bridge is dependent on the range of temperature change, material, and most importantly, length of bridge used in construction. Thermal movements are frequently accommodated using expansion joints and movable bearings. To accommodate thermal movements, it is recommended the designer allow 1-1/4 inches of movement for each 100 feet of span length for steel bridges and 1-3/16 inches of movement for each 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.

5.1.7 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. See Topic 5.1.8 for inventory and operating rating levels.

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.

5.1.8 Bridge Load Ratings

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

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:

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

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

Bridge load rating for Load and Resistance Factor Rating (LRFR) is computed based on the following basic formula:

$$Bridge\,Rating\ Factor\,(RF) = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_L)(LL + IM)}$$

where:

RF=		rating factor
C	=	capacity
DC	=	dead load effect due to structural components and attachments
DW=		dead load effect due to wearing surface and utilities
P	=	permanent load other than dead loads
LL	=	live load effect
IM	=	dynamic load allowance
γ_{DC}	=	LRFD load factor for structural components and attachments
γ_{DW}	=	LRFD load factor for wearing surfaces and utilities
γ_P	=	LRFD load factor for permanent loads other than dead loads = 1.0
γ_L	=	evaluation live-load factor

Both of the formulas above determine a rating factor for the controlling member of the bridge. For either case, the safe load capacity in tons can be calculated as follows:

$$RT = RF \times W$$

where:

RT=		rating in tons for truck used in computing live-load effect
RF=		rating factor
W	=	weight in tons of truck used in computing live-load effect

Note that when LRFR lane loading controls the rating, the equivalent truck weight (W) to be used in calculating the safe load capacity in tons is 40 tons.

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 *AASHTO Manual for Bridge Evaluation* (Section 6B.6.2 for Allowable Stress Inventory Ratings and Section 6B.6.3 for Load Factor Inventory Ratings).

The LRFD design level is comparable to the traditional Inventory rating. Bridges that pass HL-93 screening at the Inventory level are capable of carrying AASHTO legal loads and state legal loads within the AASHTO exclusion limits described in the *LRFD Bridge Design Specifications*.

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 *AASHTO Manual for Bridge Evaluation* (Section 6B.6.2 for Allowable Stress Operating Ratings and Section 6B.6.3 for Load Factor Operating Ratings).

Permit Loading

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

The second level rating is a legal load rating providing a single safe load capacity for a specific truck configuration. The second level rating is comparable to the traditional Operating rating. Bridges that pass HL-93 screening at the Operating level are capable of carrying AASHTO legal loads, but may not rate for state legal loads especially those that are considerably heavier than AASHTO trucks.

The third level rating is used to check the serviceability and safety of bridges in the review of permit applications. Permits are required for vehicles above the legal load. Only apply this third level rating to bridges with sufficient capacity for AASHTO legal loads. Calibrated load factors by permit type and traffic conditions are specified for checking the effect of the overweight vehicle. Guidance on checking serviceability criteria are also given.

Rating Vehicles

Rating vehicles are truck loads applied to the bridge to establish the inventory and operating ratings. These rating vehicles (see Figure 5.1.18) include:

- H loading
- HS loading
- HL-93
- Alternate Interstate Loading (Military Loading)
- Type 3 unit
- Type 3-S2 unit

- Type 3-3 unit
- The maximum legal load vehicles of the state
- State routine permit loads

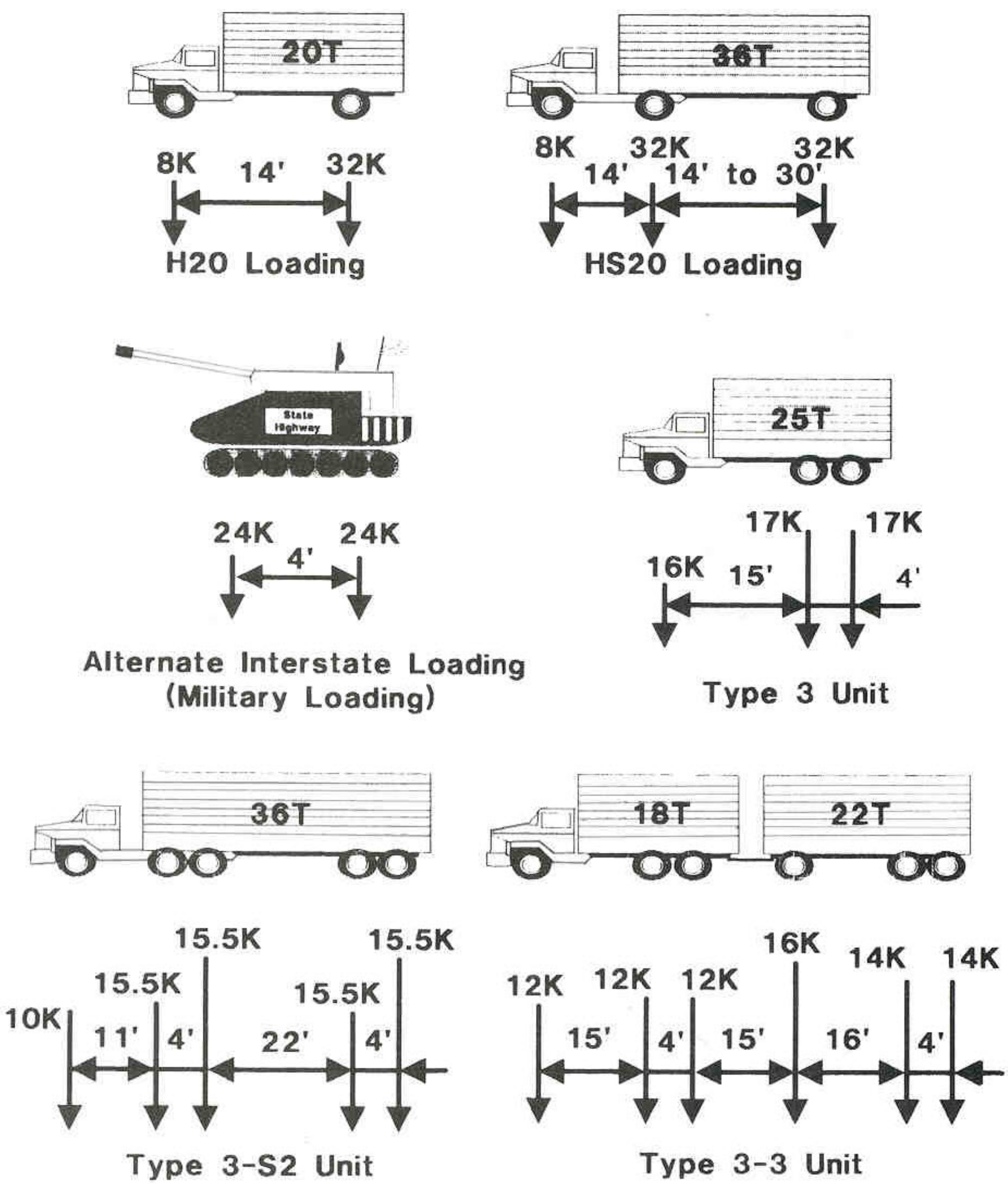


Figure 5.1.18 Rating Vehicles

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 described 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 regulation requires highway bridges on public roads to be inspected every twenty-four months for lengths greater than 20 feet. Post or restrict the bridge in accordance with the AASHTO Manual or in accordance with State law, when the maximum unrestricted legal loads or State routine permit loads exceed that allowed under the operating rating or equivalent rating factor.. It is the inspector's responsibility to gather and provide information that the structural engineer can use to analyze and rate the bridge.

The safe load-carrying capacity of a bridge considers the following criteria:

- Physical condition
- Potential for fatigue damage
- Type of structure/configuration
- Truck traffic data (include State legal loads and routine permit loads)

Bridge postings show the maximum allowable load by law for single vehicles and combinations while still maintaining an adequate safety margin (see Figure 5.1.19).

Figure 5.1.19 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 5.1.20).

Figure 5.1.20 Damaged Bridge due to Failure to Comply with Bridge Posting

5.1.9 Span Classifications

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 5.1.21).

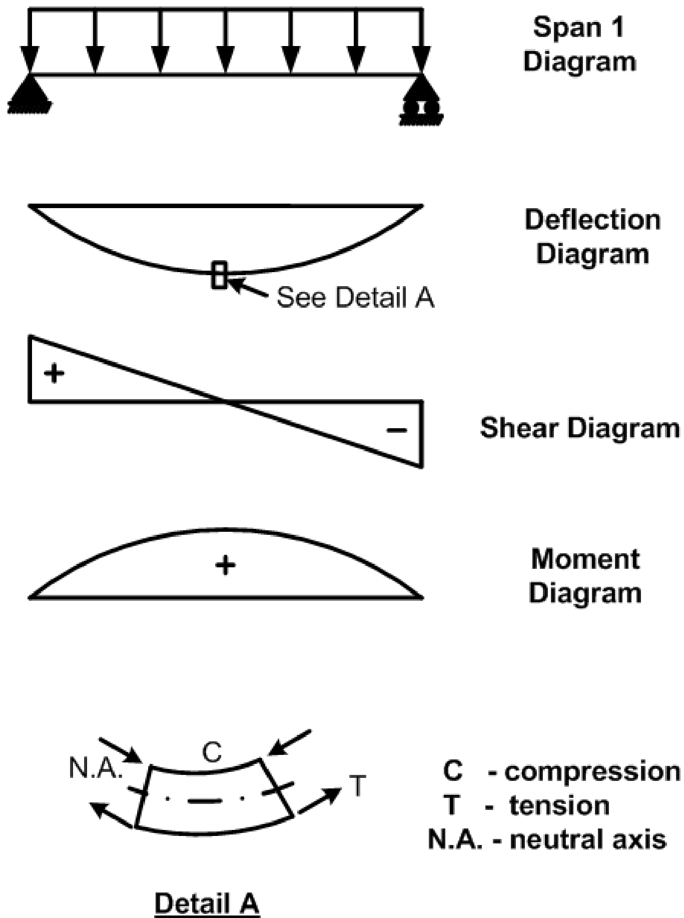


Figure 5.1.21 Simple Bridge

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
- 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 bridge has one or more intermediate supports and the behavior of each individual span is dependent on its adjacent spans (see Figure 5.1.22).

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

- When loaded, the spans deflect downward and rotate at the supports
- 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; the bending moment is zero at the end supports; 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 bridge member and tension occurs on the bottom portion of the bridge member
- For negative bending moments, tension occurs on the top portion of the bridge member and compression occurs on the bottom portion of the bridge member

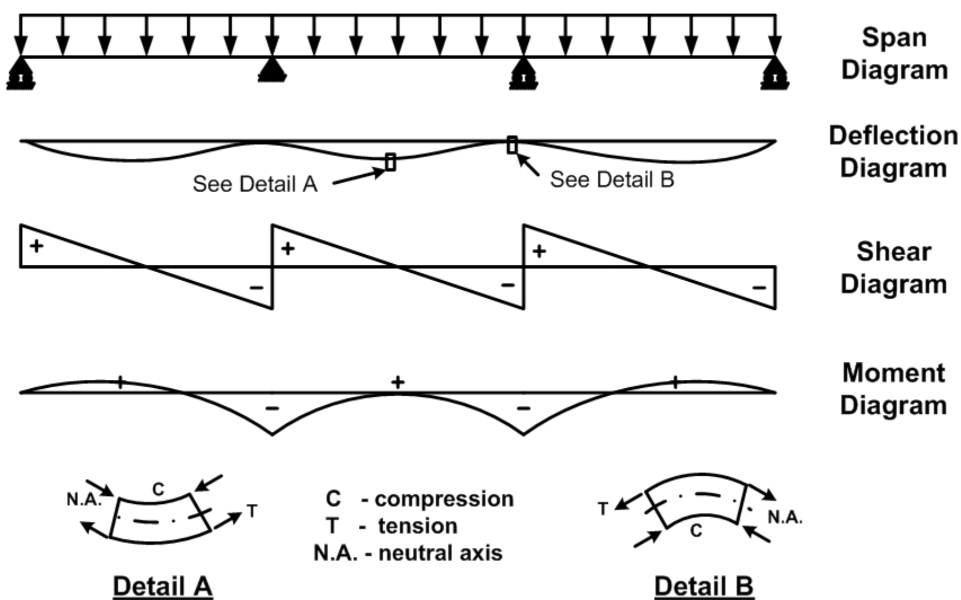


Figure 5.1.22 Continuous Bridge

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 shallower. However, a continuous bridge is more difficult to analyze than a simple span bridge and is more susceptible to overstress conditions if the supports experience differential settlement.

Cantilever

A cantilever span is a span with one end restrained against rotation and deflection and the other end completely free (see Figure 5.1.23). The restrained end is also known as a fixed support.

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

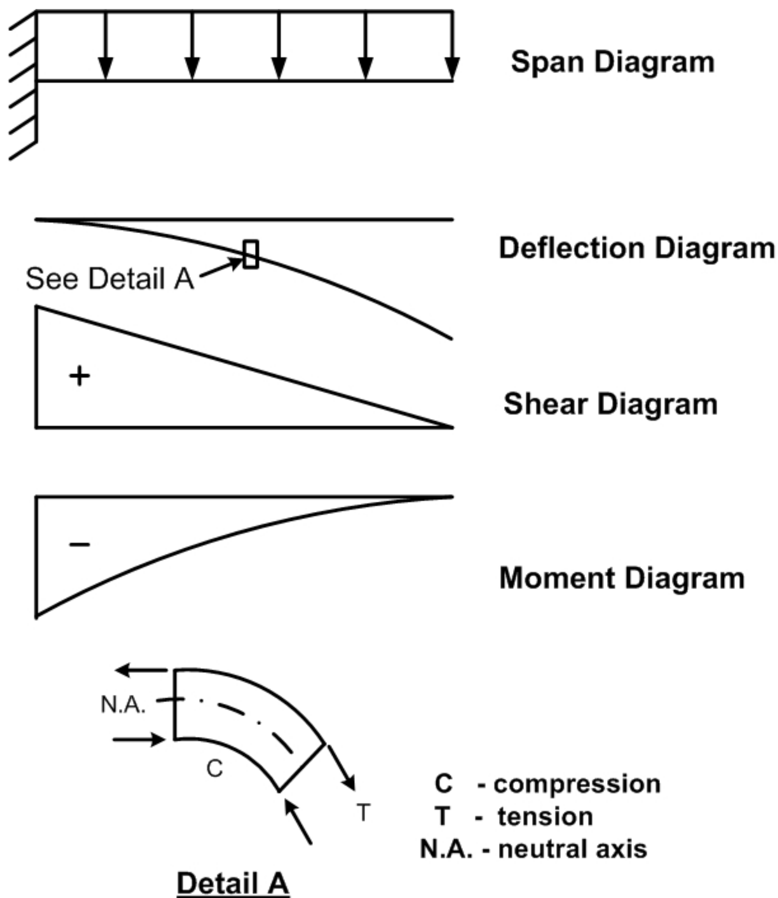


Figure 5.1.23 Cantilever Span

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 5.1.24).

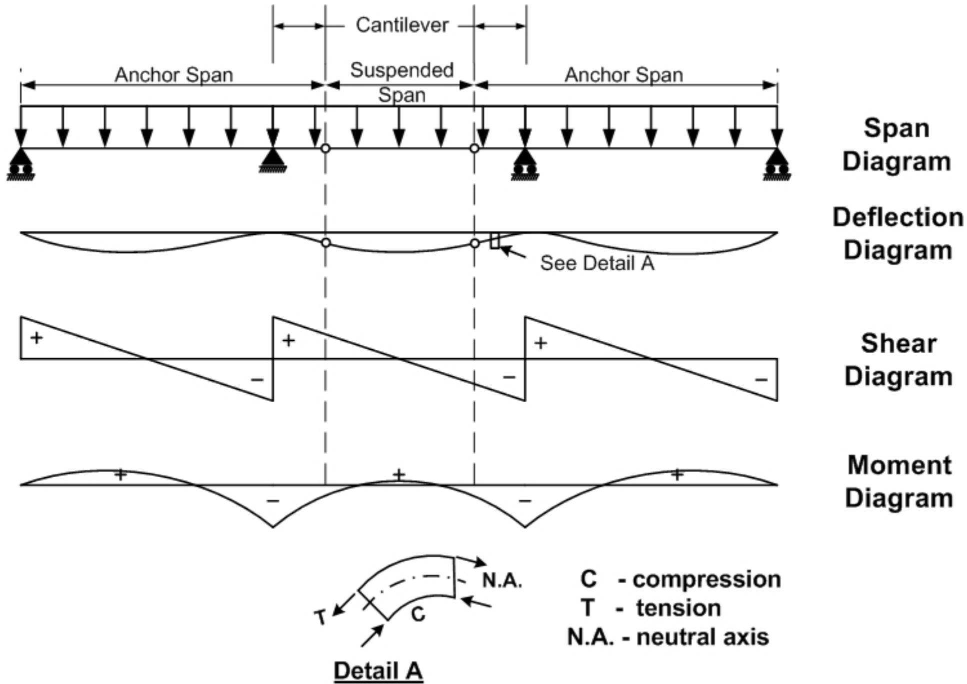


Figure 5.1.24 Cantilever Bridge

5.1.10 Bridge Deck Interaction

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

- Non-composite
- Composite
- Integral
- Orthotropic

Non-composite

A non-composite structure is one in which the superstructure acts independently of the deck. Therefore, the superstructure alone resists all of the loads applied to them, including the permanent loads and the transient loads.

Composite

A composite structure is one in which the deck acts together with the superstructure to resist the loads (see Figure 5.1.25). The deck material is 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 deck on steel superstructure and concrete deck on prestressed concrete superstructure. Shear connectors such as studs, spirals, channels, or stirrups that are attached to the superstructure and are embedded in a deck provide composite action. This ensures that the superstructure and the deck will act as a unit by preventing slippage between the two when a load is applied.

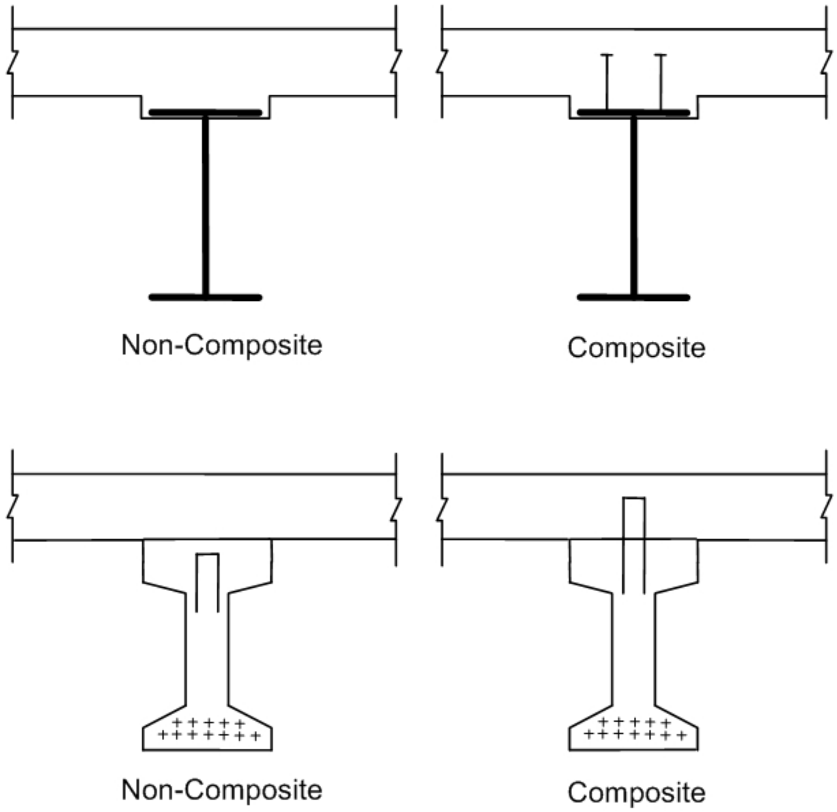


Figure 5.1.25 Non-Composite or Composite Concrete Deck on Steel Beams and Pretressed Concrete Beams

Composite action is achieved only after the concrete deck has hardened. Therefore, some of the permanent load is resisted by the non-composite action of the superstructure alone. These permanent loads include the weight of:

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

Other permanent loads, known as superimposed dead loads, are resisted by the superstructure 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 superstructure while at the bridge site. Therefore, review the bridge plans 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 Figures 5.1.26 and 5.1.27).

Integral configurations are similar to composite decks in that the deck contributes to the superstructure capacity. However, integral decks are not considered composite since the deck (or top flange) is constructed of the same material. Example of an integral bridge is a conventionally reinforced T-beam and is described in detail in Topic 9.2.

Figure 5.1.26 Integral Bridge

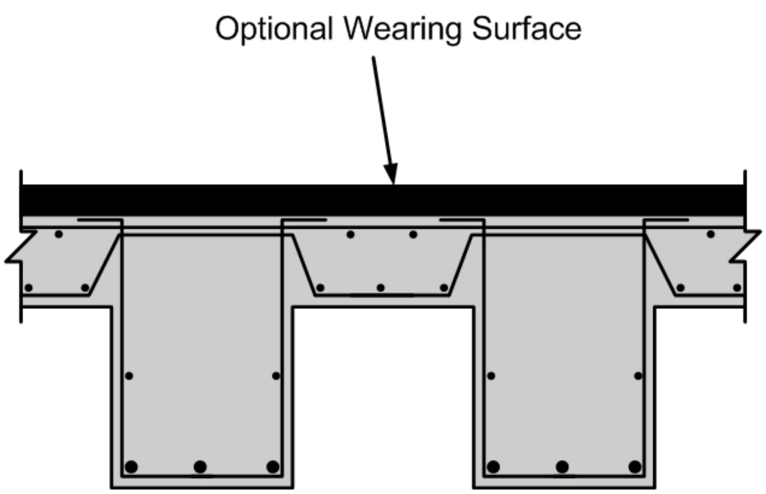


Figure 5.1.27 Cross Section of an Integral Bridge

Orthotropic

An orthotropic deck consists of a flat, thin steel plate stiffened by a series of closely spaced longitudinal ribs at right angles to their supports. 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 5.1.28).

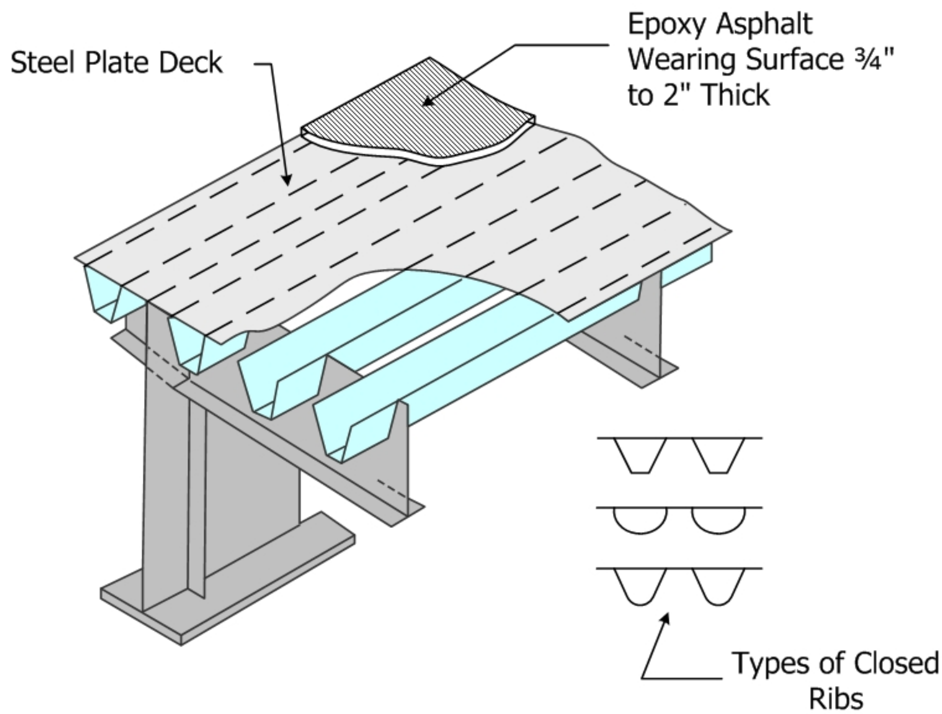


Figure 5.1.28 Orthotropic Bridge Deck

5.1.11 Redundancy

According to AASHTO Manual for Bridge Evaluation, bridge redundancy is the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members.

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 between supports. If one member were to fail, load would be redistributed to the other members and bridge failure would not be expected. 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 may not occur. A minimum of three continuous spans are needed to achieve structural redundancy in the interior spans.

Internal Redundancy

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

Nonredundant Configuration

Bridge inspectors are concerned primarily with load path redundancy and can neglect structural and internal redundancy when identifying fracture critical members. Nonredundant bridge configurations in tension contain fracture critical members. Many states currently perform 3-dimensional finite element analysis to help determine redundancy.

Redundancy is discussed in greater detail in Topic 6.4.

5.1.12 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:

- Shallow foundations commonly referred to as spread footings
- Deep 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 5.1.29).

Deep Foundations

A deep 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 (see Figure 5.1.30). “Caissons”, “drilled caissons”, and “drilled shafts” are frequently used to transmit loads to bedrock in a manner similar to piles.

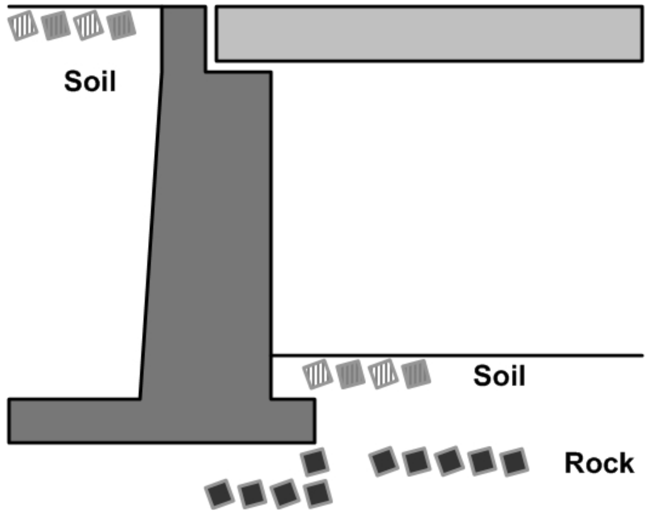


Figure 5.1.29 Spread Footing

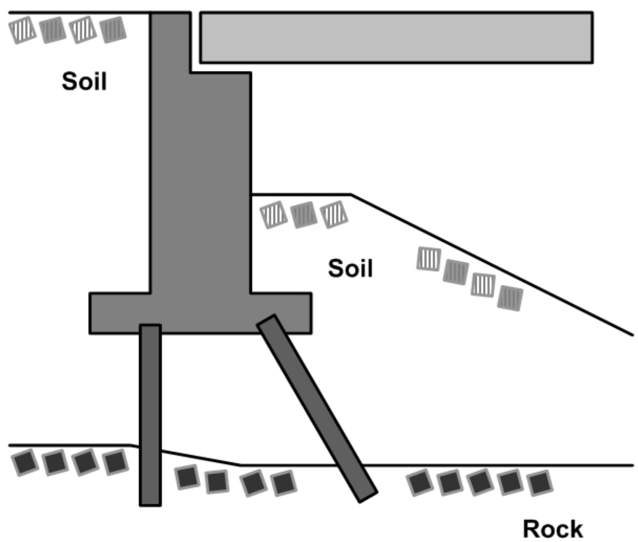


Figure 5.1.30 Deep Foundation

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Chapter 6
Bridge Materials

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		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	2"& less in thickness	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	2"& wider	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		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]	2"& less in thickness	1950	1300	2050	1,700,000	SPIB
M-18		2000	1200	1825	1,800,000	NLGA, SPIB
M-19	2"& wider	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

4 DESIGN VALUES

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)						Grading Rules Agency
		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	
BALSAM FIR								
Select Structural	Beams and Stringers	1350	900	65	305	950	1,400,000	
No.1		1100	750	65	305	800	1,400,000	
No.2		725	350	65	305	500	1,100,000	NELMA
Select Structural	Posts and Timbers	1250	825	65	305	1000	1,400,000	NSLB
No.1		1000	675	65	305	875	1,400,000	
No.2		575	375	65	305	400	1,100,000	
BEECH-BIRCH-HICKORY								
Select Structural	Beams and Stringers	1650	975	90	715	975	1,500,000	
No.1		1400	700	90	715	825	1,500,000	
No.2		900	450	90	715	525	1,200,000	NELMA
Select Structural	Posts and Timbers	1550	1050	90	715	1050	1,500,000	
No.1		1250	850	90	715	900	1,500,000	
No.2		725	475	90	715	425	1,200,000	
COAST SITKA SPRUCE								
Select Structural	Beams and Stringers	1150	675	60	455	775	1,500,000	
No.1		950	475	60	455	650	1,500,000	
No.2		625	325	60	455	425	1,200,000	NLGA
Select Structural	Posts and Timbers	1100	725	60	455	825	1,500,000	
No.1		875	575	60	455	725	1,500,000	
No.2		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	
Select Structural		1600	950	85	625	1100	1,600,000	
Dense No.1		1550	775	85	730	1100	1,700,000	
No.1		1350	675	85	625	925	1,600,000	
No.2		875	425	85	625	600	1,300,000	WCLIB
Dense Select Structural	Posts and Timbers	1750	1150	85	730	1350	1,700,000	
Select Structural		1500	1000	85	625	1150	1,600,000	
Dense No.1		1400	950	85	730	1200	1,700,000	
No.1		1200	825	85	625	1000	1,600,000	
No.2		750	475	85	625	700	1,300,000	
Dense Select Structural	Beams and Stringers	1850	1100	85	730	1300	1,700,000	
Select Structural		1600	950	85	625	1100	1,600,000	
Dense No.1		1550	775	85	730	1100	1,700,000	
No.1		1350	675	85	625	925	1,600,000	
Dense No.2		1000	500	85	730	700	1,400,000	
No.2		875	425	85	625	600	1,300,000	WWPA
Dense Select Structural	Posts and Timbers	1750	1150	85	730	1350	1,700,000	
Select Structural		1500	1000	85	625	1150	1,600,000	
Dense No.1		1400	950	85	730	1200	1,700,000	
No.1		1200	825	85	625	1000	1,600,000	
Dense No.2		800	550	85	730	550	1,400,000	
No.2		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	
No.1		1300	675	85	625	925	1,600,000	
No.2		875	425	85	625	600	1,300,000	NLGA
Select Structural	Posts and Timbers	1500	1000	85	625	1150	1,600,000	
No.1		1200	825	85	625	1000	1,600,000	
No.2		725	475	85	625	700	1,300,000	
DOUGLAS FIR-SOUTH								
Select Structural	Beams and Stringers	1550	900	85	520	1000	1,200,000	
No.1		1300	625	85	520	850	1,200,000	
No.2		825	425	85	520	525	1,000,000	WWPA
Select Structural	Posts and Timbers	1400	950	85	520	1050	1,200,000	
No.1		1150	775	85	520	925	1,200,000	
No.2		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)

Combination Symbol[4]	Species Outer Lams/ Core Lams[5]	BENDING ABOUT X-X AXIS (Loaded Perpendicular to Wide Faces of Laminations): Bending: Tension Zone Stressed in Tension F_{bxx}	Bending: Compression Zone Stressed in Tension[6] F_{bxx}	Compression Perpendicular to Grain: Tension Face[9,10] $F_{c\perp xx}$	Compression Perpendicular to Grain: Compression Face[9,10] $F_{c\perp xx}$	Shear Parallel to Grain[10] F_{vxx}	Modulus of Elasticity E_{xx}	BENDING ABOUT Y-Y AXIS (Loaded Parallel to Wide Faces of Laminations): 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}	AXIALLY LOADED: 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

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Chapter 6
Bridge Materials

Topic 6.1 Timber

6.1.1 Introduction

Approximately 4% of the bridges listed in the National Bridge Inventory (NBI) are classified as timber bridges. 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 6.1.1). 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 April 2005 manual published by the United States Department of Agriculture, Forest Service is an excellent reference to supplement timber information in this manual and is available for purchase. For other publications from Forest Products Laboratory in PDF format, access the following link: http://spfnic.fs.fed.us/werc/resources/PubSearch.cfm.

For detailed information concerning the design and analysis of timber structures, contact the American Wood Council for the National Design Specifications (NDS) for Wood construction at http://www.awc.org/standards/nds.html. Some useful information that can be obtained to assist bridge designers and inspectors includes:

- Nominal and minimum dressed sizes of sawn lumber
- Section properties of standard dressed (S4S) sawn lumber
- Section properties of structural glued laminated timber
- Reference design values for visually graded dimension lumber
- Reference design values for mechanically graded dimension lumber
- Reference design values for visually graded decking
- Reference design values for structural glued laminated timber

Figure 6.1.1 Glued-laminated Modern Timber Bridge

6.1.2 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 from solid round logs. Logs were generally used as beams, or stacked and used as abutments and foundations. Round timber members have been used as piles driven into the ground or channel (See Figure 6.1.2). 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, and curbs or railings (see Figure 6.1.2).

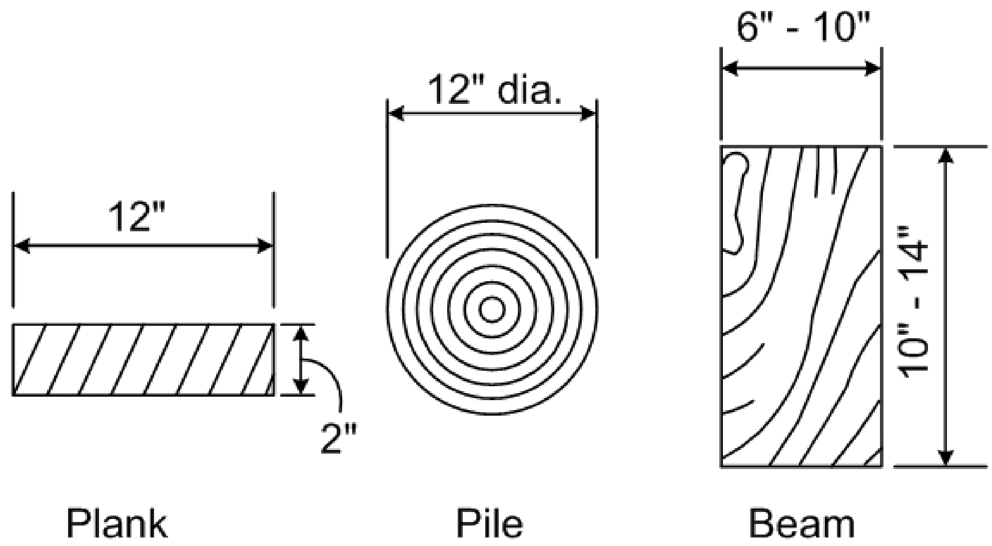

Figure 6.1.2 Timber Shapes

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 these are rectangular or deck/slab beams. Two other common examples are T-shaped and box-shaped beams (see Figure 6.1.3). Using glue-laminate technology and stress timber design, these shapes enable modern timber bridges to carry current legal loads.

Refer to Chapter 8 for further information on timber superstructures and Topic 7.1 for timber decks.

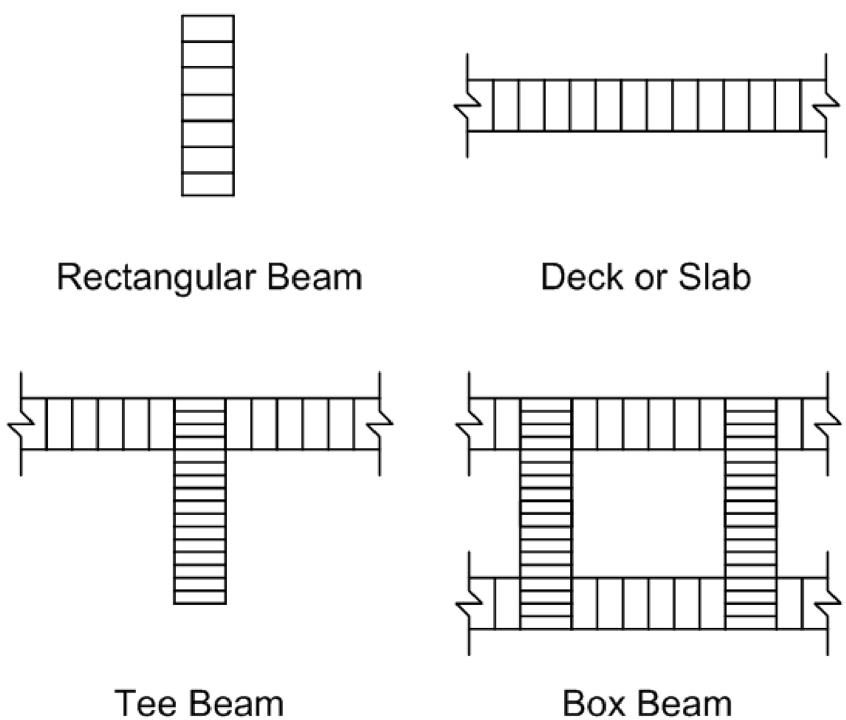

Figure 6.1.3 Built-up Timber Shapes

6.1.3 Properties of Timber

Because of its physical characteristics, timber is in many ways an excellent engineering material for use in bridges. Perhaps foremost is that it is a renewable resource. In addition, timber 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, timber 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, 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 timber 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 Figures 6.1.4 and 6.1.5):

- 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 are distinct rings of wood produced during a tree's growing season. One annual ring is composed of a ring of earlywood or springwood (light in color, cells have thin walls and large diameter) and a ring of latewood or summerwood (dark in color, cells have thick walls and small diameter). The rings can be easily seen in some trees (Douglas fir and southern pine) and exhibit little color difference in other species (spruces and true firs).
- Sapwood - the active, outer part of the tree that carries 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 that does not carry sap; 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
- Grain - the wood fibers oriented along the long axis of logs and timbers; the direction of greatest strength
- Wood rays - groups of cells, running from the center of the tree horizontally to the bark, which are responsible for cross grain strength
- Pith - center of the tree, representing the earliest growth of the tree. The pith is more resistant to rot.
- Resin canal - tubular passageways lined with living cells producing resin or "pitch". Hardwoods do not contain resin canals.
- Bark - outer layer of a tree. The outer bark is composed of mostly dead cells that form a protective barrier for the tree. The inner bark is made from living cells which transport sugars, and may also protect the tree from contaminants.

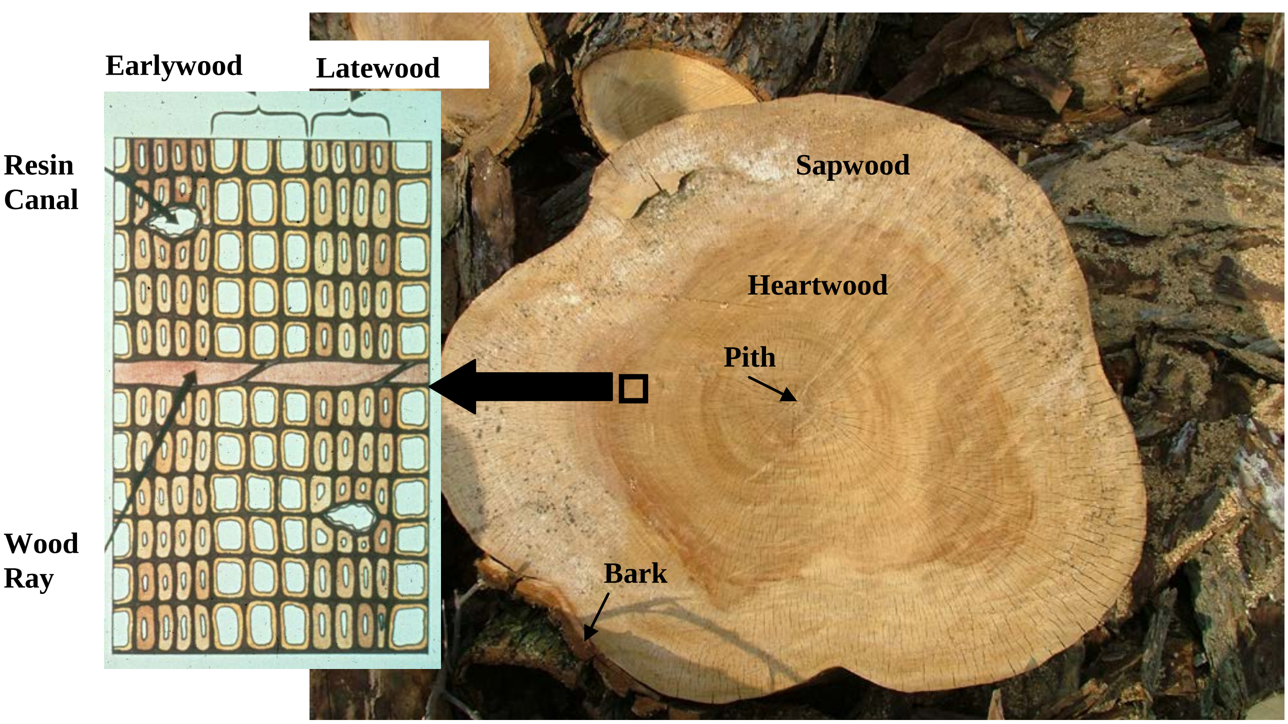

Figure 6.1.4 Anatomy of Timber

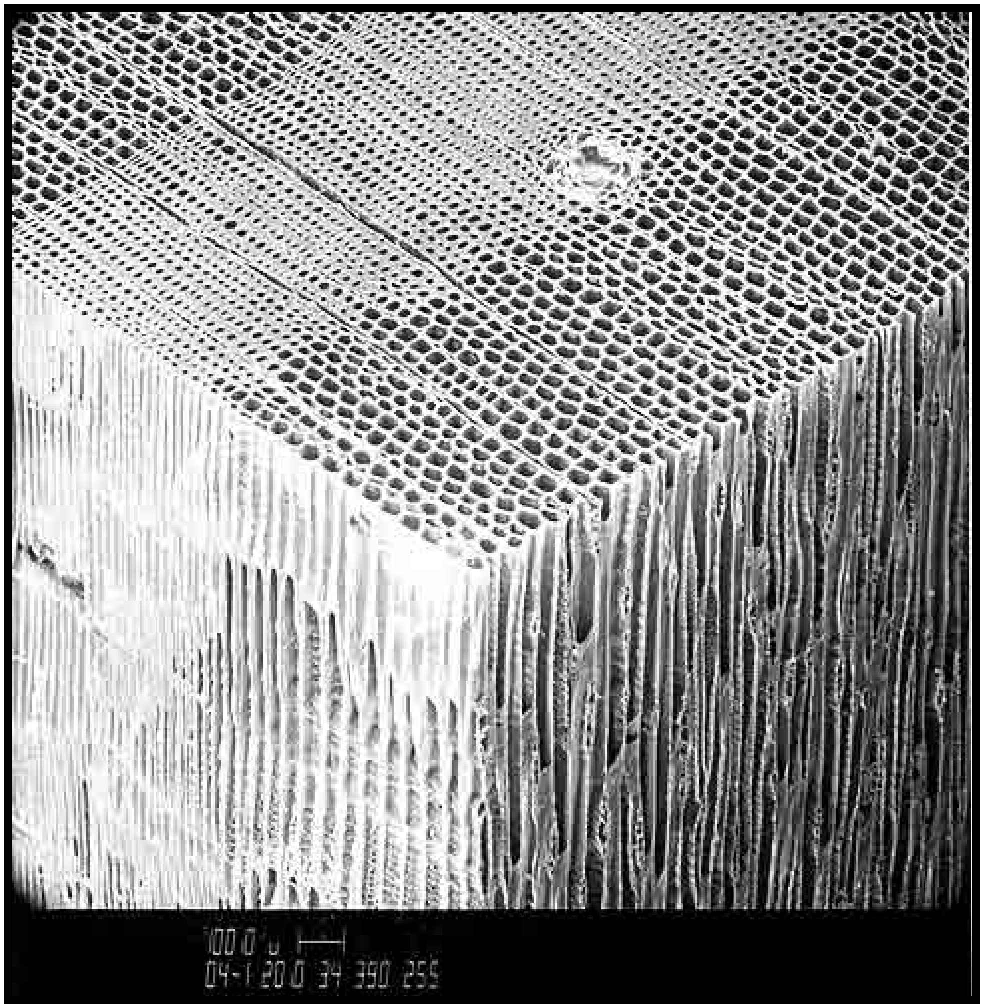

Figure 6.1.5 3-D Close-up of Softwood Timber Anatomy
(Source: Society of Wood Science and Technology)

Growth Features

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

- Knots and knot holes - due to growth around an embedded limb and associated grain deviation. Knots may be small or large, round or elongated
- 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 occurs between the growth rings
- 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 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

There are four basic mechanical properties that define timber behavior: orthotropic behavior, fatigue characteristics, impact resistance and creep characteristics.

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 or different 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 6.1.6).

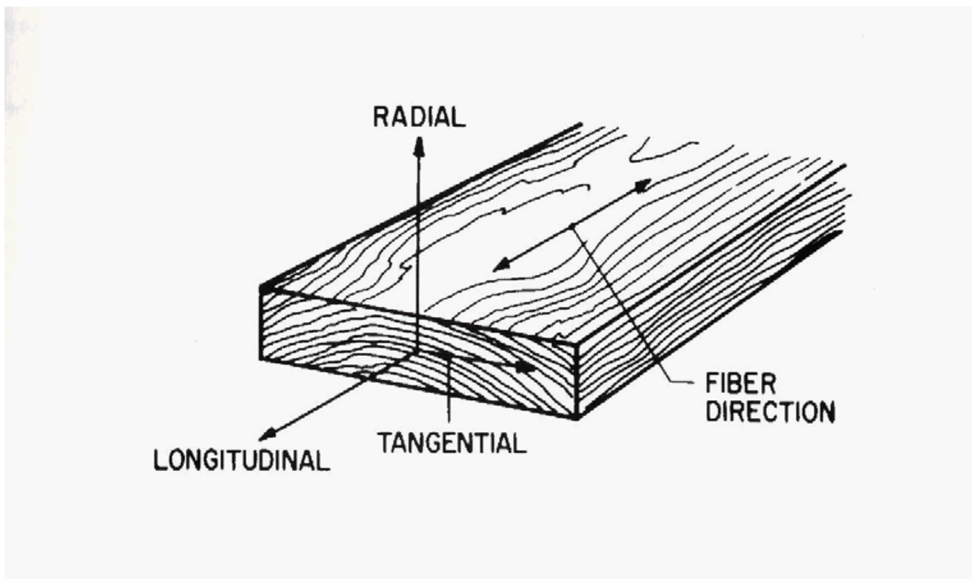

Figure 6.1.6 Three Principal Axes of Wood

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. American Society for Testing and Materials (ASTM) Standards and American Forest and Paper Association (AF&PA) 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 approximately 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 the timber member. That is, the initial deflection of the member increases with time. Unseasoned or "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 timber members will exhibit little permanent increase in deflection with time.

6.1.4 Timber Grading

Douglas fir and southern pines are the most widely used species of wood for bridge construction. 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 design 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

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 deficiencies. Reduction factors need to be applied to these values based on specific application.

Timber used for outdoor applications needs to be designed for wet service conditions. This is often done with the use of a wet service reduction factor. For certain species of timber, such as Southern Pine, this factor may already be incorporated into the design strength of the wood regardless of a wet or dry service condition..

Other application-based reduction factors include temperature, member size and length, member volume, member orientation, load duration, and specific use. For more information, refer to the National Design Specifications for Wood Construction, American Forest & Paper Association, American Wood Council.

Preservative treatment for decay resistance does not alter the design strength of wood, 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 have been established.

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. A 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 is not graded in the same way as sawn lumber (see the tables at the beginning of this topic). Members have a combination symbol that represents the combination of lamination grades used to manufacture the member. The symbols are divided into two general classifications which are bending combinations or axial (tension or compression) combinations. The classifications are based in the anticipated use of the member, either in bending as a beam or axial combination as a column or tension member.

Bending combinations are used for resisting bending stress caused by loads applied perpendicular to the wide faces of the laminations. In this case, a lower grade lamination is used for the center portion of the member (near the neutral axis) while a higher grade lamination is placed on the outside faces where bending stresses are higher.

Axial combinations are used for resisting axial forces and bending stress applied parallel to the wide faces of the laminations. In this case, the same grade lamination is used throughout the member.

6.1.5 Anticipated Modes of Timber Deficiency

Although timber is an excellent material for use in bridges, untreated timber is vulnerable to damage from fungi, parasites, and other sources. The untreated inner cores of surface treated timber 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. It is important for bridge inspectors to recognize the signs of the various types of damage and evaluate their effect on the structure.

Inherent Defects

Defects that form from growth features introduced in Topic 6.1.3 or from the lumber drying process include (see Figure 6.1.7):

- Checks - separations of the wood fibers, normally occurring across or through the annual growth rings, and generally parallel to the grain direction
- Splits - advanced checks that extend completely through the piece of wood. A split is also known as a through check
- Shakes - separations of the wood fibers parallel to the grain which occur between the annual growth rings
- Knots – separations of the wood fibers due to the trunk growing around an embedded limb. Knots may be small or large, round or elongated

Timber can crack, check or split due to differential shrinkage. Differential shrinkage occurs because the outer fibers in the shell dry first and begin to shrink. However, the core has not yet begun to dry and shrink, and consequently the shell is restrained from shrinking by the core. Thus the shell goes into tension and the core into compression. With the stresses from the shell and the core pulling in opposite directions the wood fibers break and a crack forms. The larger the timber member, the more stress is exerted to the timber member. This is the reason why to dry timber materials before using them where their final moisture content will be 15% or less.

These four inherent 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.

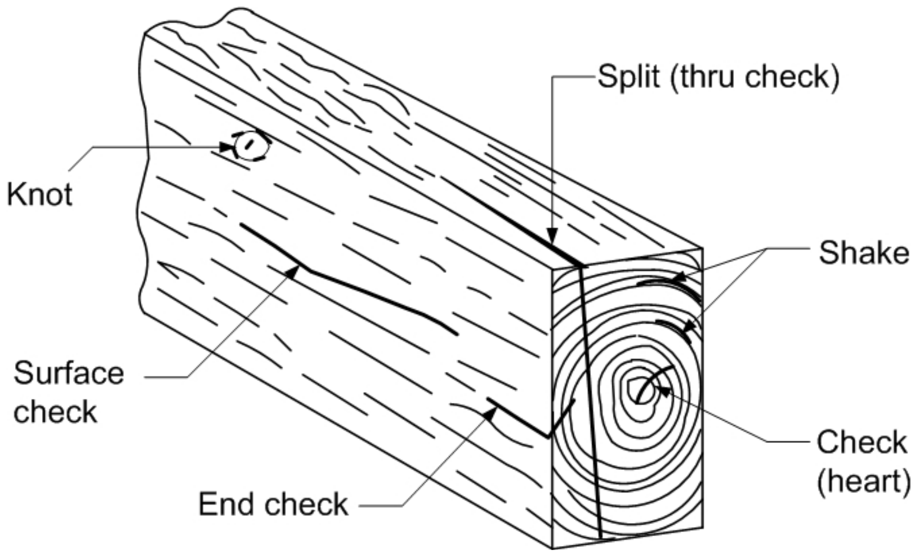

Figure 6.1.7 Inherent Timber Defects

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 6.1.8). 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.

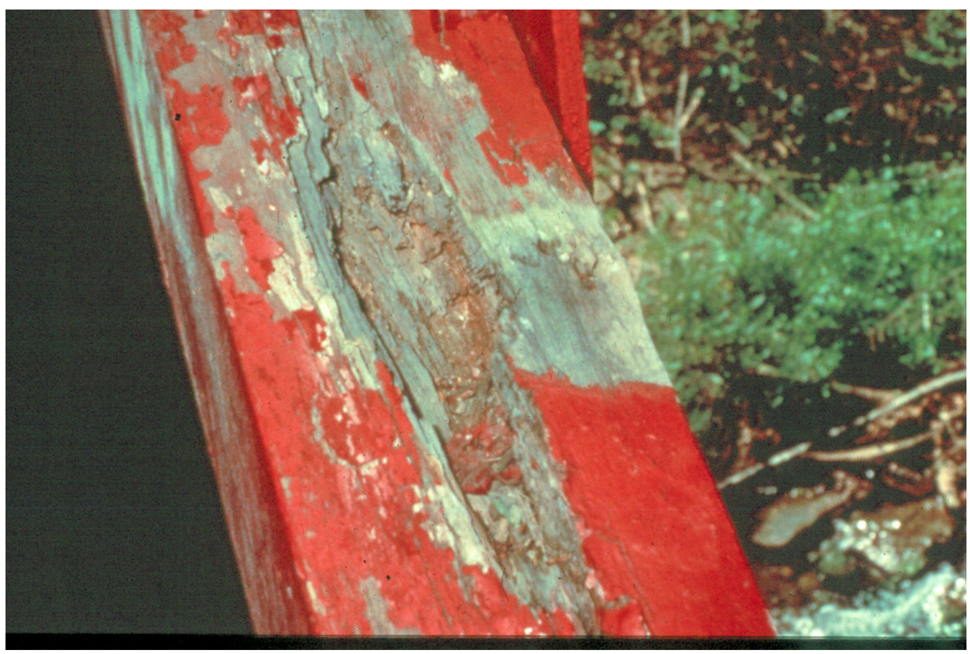

Figure 6.1.8 Decay of Wood by Fungi

Spores that survive and experience favorable growth conditions can penetrate timber 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. 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 freezing, 32ºF, the fungi become dormant but resumes its growth as the temperature rises above freezing to the 75ºF to 85ºF range, where growth is at its maximum. Above 90ºF, growth tapers off rapidly, and temperatures in excess of 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 protect 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 minimum moisture content of 20 percent to support fungi. Growth occurs when the moisture content is between 25 and 30 percent, with rapid growth of fungi above 30 percent. Rain or snow is the main source of moisture. Secondary sources are condensation, ground water, and stream water. Exposed surfaces allow moisture to evaporate harmlessly. However seasoning shakes, checks and splits, interfaces between timber members, and fastener holes are ideal for localized moisture accumulation which promotes the growth of fungi.

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 6.1.9)
- 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 (see Figure 6.1.9)
- 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

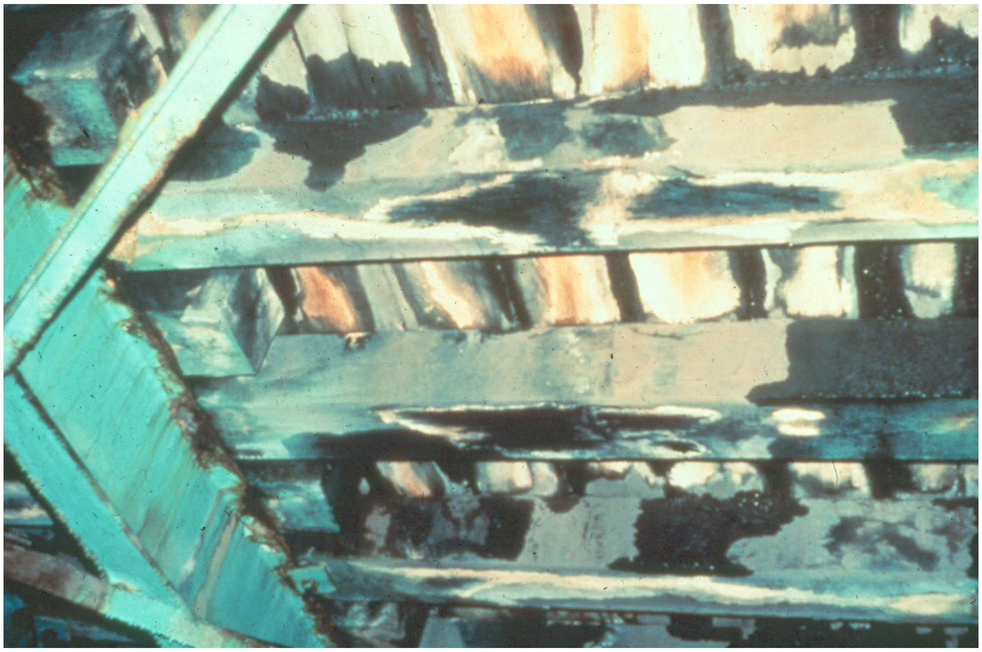

Figure 6.1.9 Mold and Stain on Underside of Timber Bridge

Some fungi types that weaken or cause damage to timber 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 6.1.10)
- White rot - feeds upon the cellulose, hemi-cellulose, and the lignin and makes the wood white and stringy (see Figure 6.1.10).

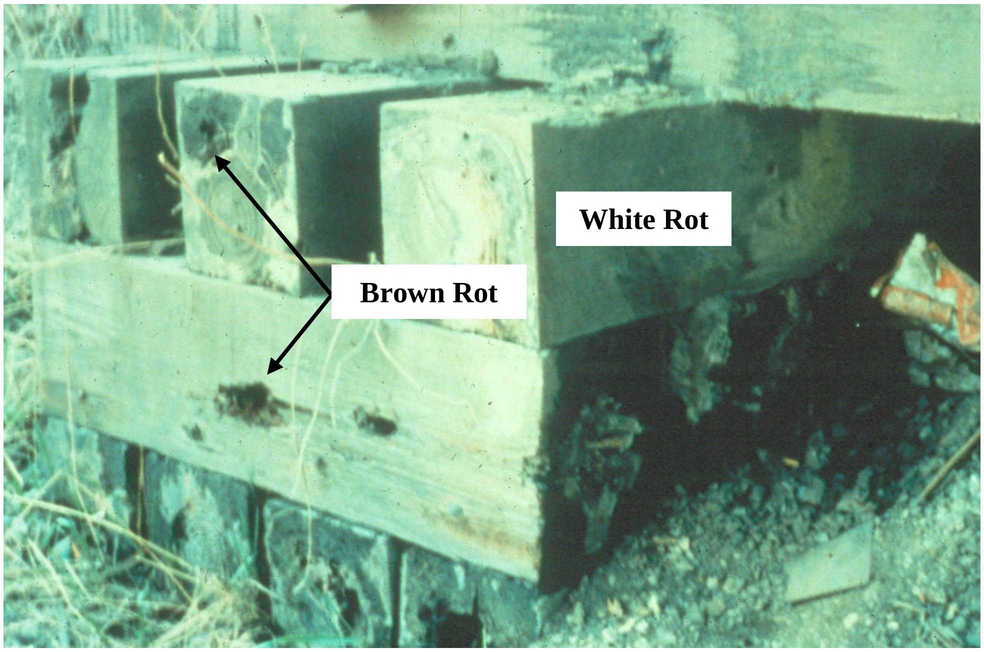

Figure 6.1.10 Brown and White Rot

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 timber 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 timber 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 40 years, 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 or shelter. Common types of insects that damage bridges include:

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

Termites

Termites are pale-colored, soft-bodied insects that feed on wood (see Figure 6.1.11). 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 in bridges throughout most of the country due to the constant vibration caused by traffic travelling over timber bridges.

Figure 6.1.11 Termites

Powder-post Beetles or Lyctus Beetles

Powder-post beetles (see Figure 6.1.12) 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.

Figure 6.1.12 Powder Post Beetle

Carpenter Ants

Carpenter ants are large, black ants up to 3/4 inches long that gnaw galleries in soft or decayed wood (see Figure 6.1.13). 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.

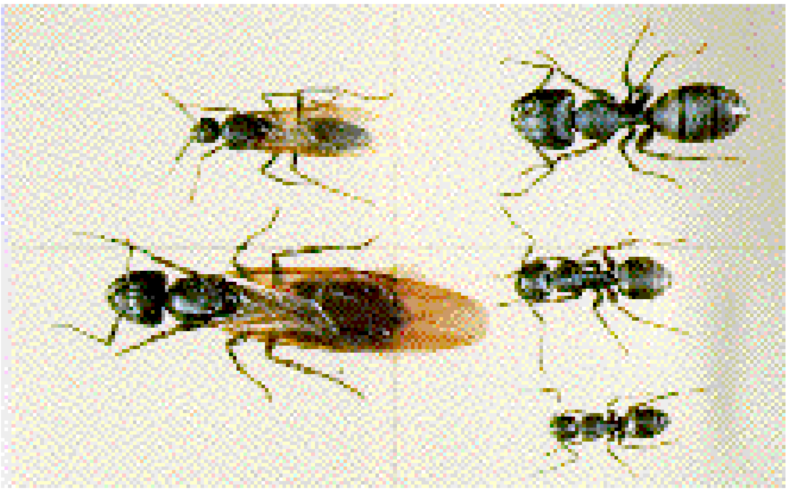

Figure 6.1.13 Carpenter Ants

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 6.1.14). 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 shelters. This explains why caddisfly larvae have been known to exist on creosote treated timber.

Figure 6.1.14 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 6.1.15). 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 6.1.16). The shipworm is one of the most serious enemies of marine timber installations. The teredo is the most common species of shipworm. 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 15 inches and a diameter of 3/8 inch. Some species of shipworm have been known to grow to a length of 6 feet and up to 1 inch in diameter. The teredo maintains a small opening in the surface of the wood to obtain nourishment from the sea water.

Figure 6.1.15 Marine Borer Damage to Wood Piling

Figure 6.1.16 Shipworm (Mollusk)

Another type of marine borer is the crustacean borer. The most commonly encountered crustacean borer is the limnoria or wood louse (see Figure 6.1.17). 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 1/8 to 1/4 inches long and 1/16 to 1/8 inches wide.

Figure 6.1.17 Limnoria (Wood Louse)

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 6.1.18).

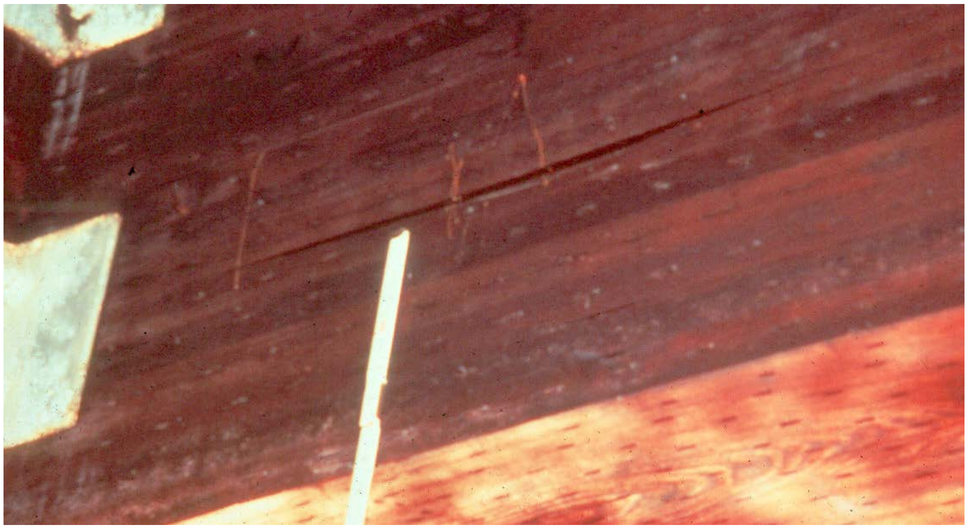

Figure 6.1.18 Delamination in a Glue Laminated Timber Member

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 6.1.19).

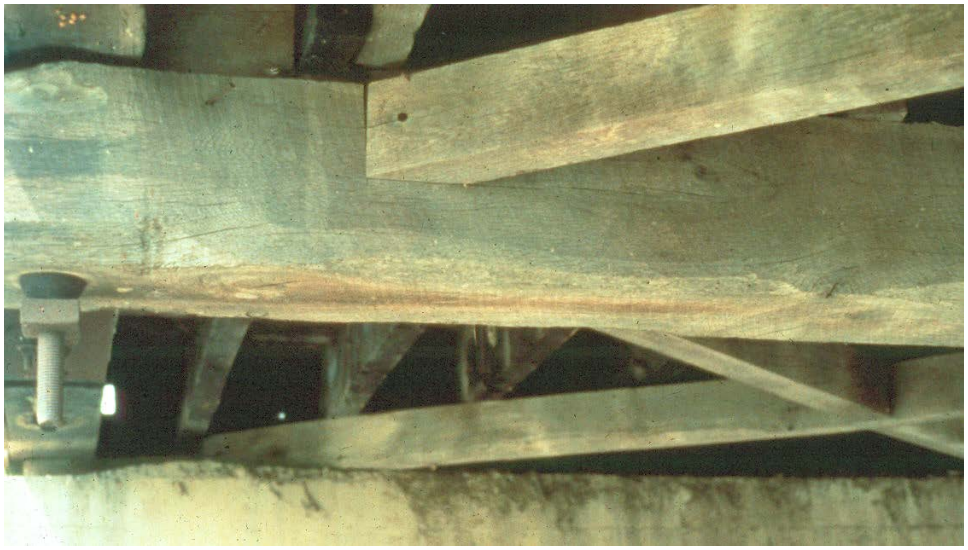

Figure 6.1.19 Loose Hanger Connection Between the Timber Truss and Floorbeam

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 per minute during the first 30 minutes of exposure, and 0.021 inches per minute thereafter (see Figure 6.1.20). 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 by fire. Preservative treatments are available to retard fire damage.

Figure 6.1.20 Fire Damaged Timber Members

Impact or Collisions

Severe damage can occur to truss members, railings, and columns when an errant vehicle strikes them (see Figure 6.1.21).

Figure 6.1.21 Impact/Collision Damage to a Timber Member

Wear, Abrasion and Mechanical Wear

Vehicular traffic is the main source of wear on timber decks (see Figure 6.1.22). Abrasion 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 6.1.22 Wear of a Timber Deck

Overstress

Each timber member has a certain ultimate load capacity. If this load capacity is exceeded, the member will fail. Failure modes include horizontal shear failure, bending moment or flexural failure, and crushing (see Figures 6.1.23, 6.1.24, and 6.1.25).

Figure 6.1.23 Horizontal Shear Failure in Timber Member

Figure 6.1.24 Failed Timber Floorbeam due to Excessive Bending Moment

Figure 6.1.25 Timber Substructure Member Subjected to Crushing and Overstress

Weathering or Warping

Weathering is the affect of sunlight, 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 6.1.26).

Figure 6.1.26 Weathering on Timber Deck

Protective Coating Failure

The following paint failures are common on timber structures:

- Cracking and peeling extend with the grain of the wood. They are caused by different shrinkage and swell rates of expansion and contraction between springwood and denser summerwood.
- 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.

6.1.6 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.

Types and Characteristics of Wood Protectants

Water Repellents

Water repellents slow or retard 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 layers due to swelling and shrinkage.

Preservatives

Wood preservatives prevent biological deterioration that can penetrate 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 8 feet in diameter and 140 feet long. The chamber is placed under a vacuum, drawing the air from the wood pores and cells. The treatment chemical is then fed into the chamber and pressure up to 200 psi is applied, forcing the chemical into the wood (see Figure 6.1.27). 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 50 pounds per cubic foot (pcf) compared to approximately 40-45 pcf for untreated timber.

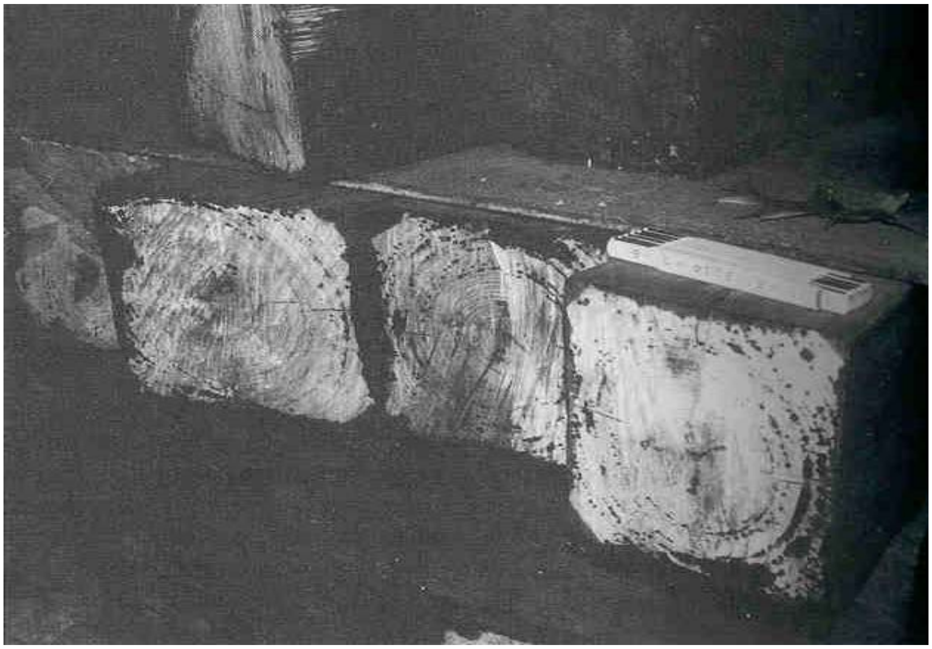

Figure 6.1.27 Bridge Timber Member Showing Penetration Depth of Preservative Treatment

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 6.1.28). 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. Due to environmental and health concerns, new bridge construction practices do not allow the use of creosote.

Figure 6.1.28 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 an above-ground 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, though this practice is not usually recommended.

Copper naphthenate is an organic solvent solution suitable for above-ground, ground contact, or freshwater applications. It is not standardized for salt water uses. When applied, copper naphthenate stains the wood to a light green color which weathers to a light brown. It leaves an oily surface and should not be used for frequent human contact. Timber members treated with this preservative may be painted several weeks after weathering. Cuts or holes may be treated in the field with copper naphthenate.

Oxine copper is an organometallic compound that is used for above-ground applications (when dissolved in a heavy oil). Examples include difficult-to-treat species such as Douglas-fir for bridges and railings. The effectiveness of oxine copper is significantly reduced when used in direct contact with ground or water, and has therefore not been standardized for those applications. Oxine copper is also used to pressure-treat wood and may used to control fungi and insects.

Chromated copper arsenate (CCA) was the most popular timber preservative from the late 1970's until 2004. CCA is an excellent waterborne salt decay inhibitor, but can also be used for above-ground, ground contact, and freshwater applications. CCA is applied by vacuum-pressure treatment and comes in three standard formulations: CCA Type A, CCA Type B, and CCA Type C (most common). Timber treated with CCA has a green appearance, but readily accepts painting. CCA also provides limited protection against the ultraviolet rays in sunlight. This compound has been voluntarily phased out for residential and other human contact applications and is restricted by the EPA.

Ammoniacal copper zinc arsenate (ACZA) is a refined variant of ammoniacal copper arsenate (ACA), which is no longer available in the United States. The color of ACZA treated wood ranges from olive to bluish green and has a slight ammonia odor until it has cured. This preservative is effective against fungi and insect attack over a wide range of exposures and applications, including ground and water contact. Despite accelerating fastener corrosion, many agencies require treatment with ACZA for highway structures and other critical structural components. This preservative is especially common in the Western United States.

Alkaline copper quaternary (ACQ) compounds have recently been developed and marketed in response to the rapidly declining use of CCA, despite the inability to be used in saltwater environments. Similar to CCA, ACQ has several different variations including ACQ Type B, ACQ Type C, and ACQ Type D. Treatment with ACQ-B is used for difficult-to-treat Western species, as this compound is more effective than other waterborne preservatives. ACQ-B gives off a dark greenish-brown color which later fades to a lighter brown. Treatment with ACQ-D is used for most other easy-to-treat applications, especially for pressure-treated lumber. ACQ-D gives off a lighter greenish-brown color. Applications for ACQ-C are still limited, as this variant is the most recently standardized. Overall, ACQ compounds have proven effective against fungi and insects for ground contact applications. Similar to treatment with ACZA, ACQ compounds accelerate corrosion of metal fasteners.

Copper azole is another recently developed compound marketed as an alternative to CCA. This chemical is designed to protect wood from decay and insect attack and comes in two different formulations, copper azole type A (CBA-A) and copper azole type B (CA-B). With CBA-A no longer used in the United States, CA-B is also frequently used for pressure-treated applications along with ACQ-D. For difficult-to-treat Western species, ammonia may be added to CA-B, though this addition darkens the otherwise greenish-brown color. As with ACZA and ACQ compounds, copper azole formulations increase corrosion rates of metal fasteners. Copper azole compounds cannot be used for saltwater applications.

Fire Retardants

Fire retardants will not indefinitely prevent wood from burning but will slow or 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 enough to paint. 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. In order for paint to adhere, mildew is 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.

6.1.7 Inspection Methods for Timber

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

- Visual
- Physical
- Advanced inspection methods

Visual Examination

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

The second type of visual inspection is called an in-depth inspection. An in-depth inspection is an inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine methods. Hands-in inspection may be necessary at some locations. 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 deficiencies. The hands-on inspection method may be supplemented by nondestructive testing.

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 deficiencies are identified visually, physical methods must be used to verify the extent of the deficiency. Most physical inspection methods for timber members involve destructive methods. An inspection hammer, on the other hand, does not damage timber 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.

Sounding the wood surface by striking it with a hammer or other object is one of the oldest and most commonly used inspection methods for detecting interior deterioration. Based on the tonal quality of the ensuing sounds, a trained inspector can interpret dull or hollow sounds that may indicate the presence of large interior voids or decay. Although sounding is widely used, it is often difficult to interpret because factors other than decay can contribute to variations in sound quality. In addition, because sounding will reveal only serious internal deficiencies, it is never to be the only method used. Sounding provides only a partial picture of the extent of decay present and will not detect wood in the incipient or intermediate stages of decay. Nevertheless, sounding still has its place in inspection and can quickly identify seriously decayed structures. When suspected decay is encountered, it must be verified by other methods such as boring or coring.

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 (see Figure 6.1.29). Sound wood splinters, while decayed wood breaks abruptly.

Figure 6.1.29 Inspector Performing a Pick Test

Timber Boring and Drilling Locations

The following are common timber boring and drilling locations (see Figure 6.1.30):

- 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.

These locations are suspect areas where moisture accumulates and could lead to decay.

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. Apply repairs to any drilled holes once the drilling and boring is complete.

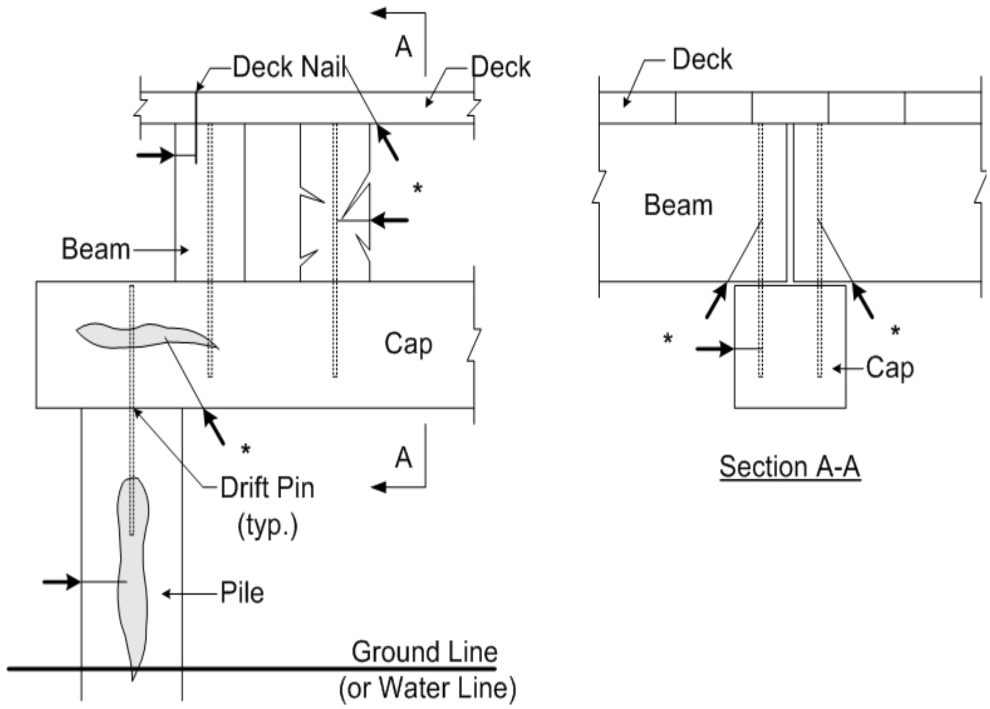

Figure 6.1.30 Timber Boring and Drilling Locations

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. Check the condition of the protective coatings.

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.

Another paint adhesion assessment method is performed in accordance with American Society for Testing and Materials (ASTM) D-3359 "Measuring Adhesion by Tape Test" which is used primarily for metal substrates. 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 can be directly measured with a special gage (see Figure 6.1.31). 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 6.1.31 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 Methods

In addition, several advanced methods are available for timber inspection. Nondestructive methods, described in Topic 15.1.1, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 15.1.3, include:

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

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Bridge Materials

Topic 6.2 Concrete

6.2.1 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, pozzolans)

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 212 to 2750 degrees Fahrenheit. 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 percent 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 approximately 140 to 150 pcf. Typical aggregate materials for normal weight concrete are sand, gravel, crushed stone, and air-cooled, blast-furnace slag. Ingredients such as sand is considered a ‘fine’ aggregate. Other aggregates are considered ‘course’ aggregates.

Lightweight concrete normally has a unit weight of 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 strength of concrete. 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 percent in areas where the concrete will be exposed to deicing salts and to 50 percent 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 of concrete. This type of admixture is needed when high temperatures are expected during placing and curing. Retarders slow down the setting time to reduce unwanted temperature and shrinkage cracks which result from a fast curing mix.

6.2.2 Basic Shapes Used in Bridge Construction

As a bridge construction material, the most basic shapes used for concrete members are either round or rectangular.

Round

Round shaped members are most commonly used in substructures and are cast-in-place. Common uses of round concrete members in bridge construction are piles or pier columns. (see Figure 6.2.1)

Figure 6.2.1 Round Concrete Members

Rectangular

Rectangular shaped members can be used for both superstructure and substructure bridge elements and are cast-in-place. Common uses of this shape in bridge construction can include beams/girders, pier caps, piles, and columns. (see Figure 6.2.2)

Figure 6.2.2 Rectangular Concrete Members

6.2.3 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 700 degrees Fahrenheit may cause damage
- Formability - concrete can be cast to almost 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 2500 psi to about 6000 psi. Higher strength concrete, with compressive strengths ranging from 6000 psi to about 20,000 psi, is also available and becoming more commonly used. However, its tensile strength is only about 10 percent of its compressive strength, its shear strength is about 12 percent to 13 percent of its compressive strength, and its flexural strength is about 14 percent of its compressive strength (see Figure 6.2.3).

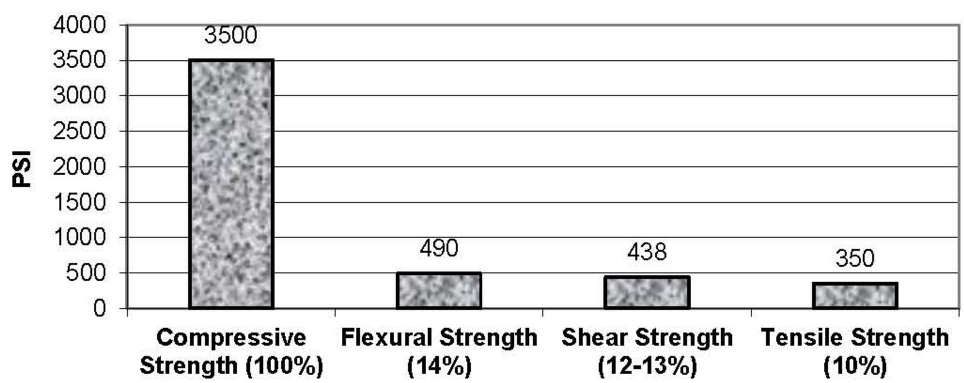
Six principal factors that increase concrete strength are:

- 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 5.1 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 percent to 200 percent of initial elastic deformation, depending on time.

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



Note: Percentages represent a comparison of various strength properties with the compressive strength of concrete.

Figure 6.2.3 Strength Properties of Concrete (3500 psi Concrete)

High Performance Concrete

High performance concrete (HPC) has been used for more than 30 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 Figure 6.2.4). 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 twice 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 or in construction plans and specifications.

HPC Type	Minimum Strength Criteria	Water-Cementitious Ratio	Minimum Durability Factor
Very Early Strength (VES)	2,000 PSI / 6 hours	≤ 0.4	80%
High Early Strength (HES)	5,000 PSI / 24 hours	≤ 0.35	80%
Very High Strength (VHS)	10,000 PSI / 28 days	≤ 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 Vanikar, and Ray Cook; *Concrete International*, February 1996, The American Concrete Institute.
- "Workshop Showcases High-Performance Concrete Bridges," *Focus* Newsletter, May 1996.

Figure 6.2.4 FHWA's Strategic Highway Research Program (SHRP) Implemented HPC Mix Designs

Ultra-High Performance Concrete

Ultra-high performance concrete (UHPC) has exceedingly high durability and compressive strength. This form of concrete is a high strength, ductile material that is formulated from a special combination of constituent materials which include Portland cement, silica fume, quartz flour, fine silica sand, high-range water-reducer, water, and either steel or organic fibers.

UHPC has compressive strengths of 18,000 psi to 33,000 psi and flexural strengths of 900 to 7,000 psi, which depends on the type of fibers that are being used and if a secondary treatment is used to help further develop compressive strength. UHPC also has the capability to sustain deformations and resist flexural and tensile stresses, even after it initially cracks.

6.2.4 Reinforced Concrete

Concrete is commonly used in bridge applications due to its compressive strength properties. However, in order to supplement the limited tensile, shear and flexural strengths of concrete, reinforcement is used.

Conventionally Reinforced Concrete

Current steel reinforcement has a tensile yield strength of 60 ksi or 75 ksi and therefore has approximately 100 times the tensile strength of commonly used concrete. Older structures use a reinforcement that has tensile yield strength of 40 ksi. Stainless steel reinforcement (40 ksi to 75 ksi) is gaining popularity due to estimates by manufacturers that is has a service life of 100 years. Therefore, in conventionally 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 conventionally 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 (see Figure 6.2.5).

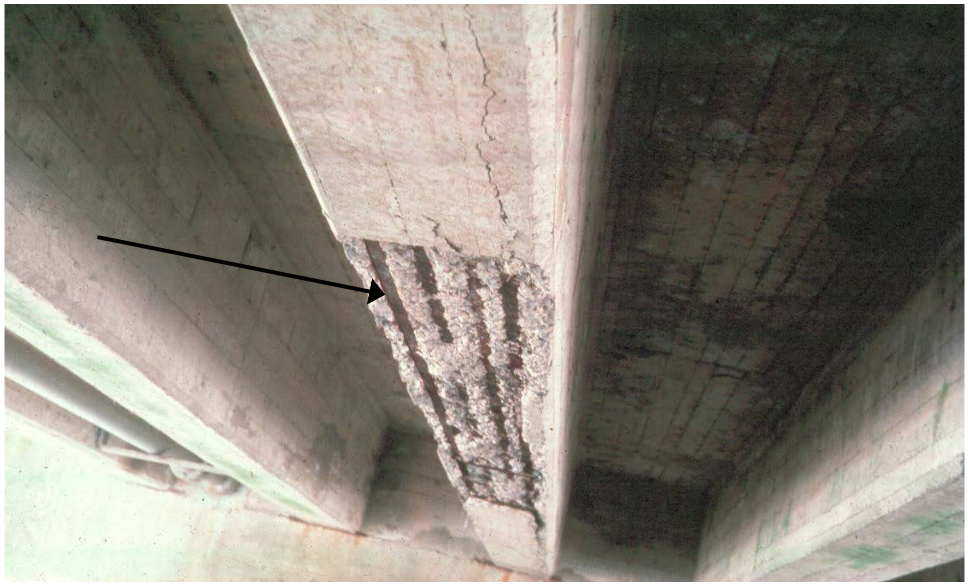
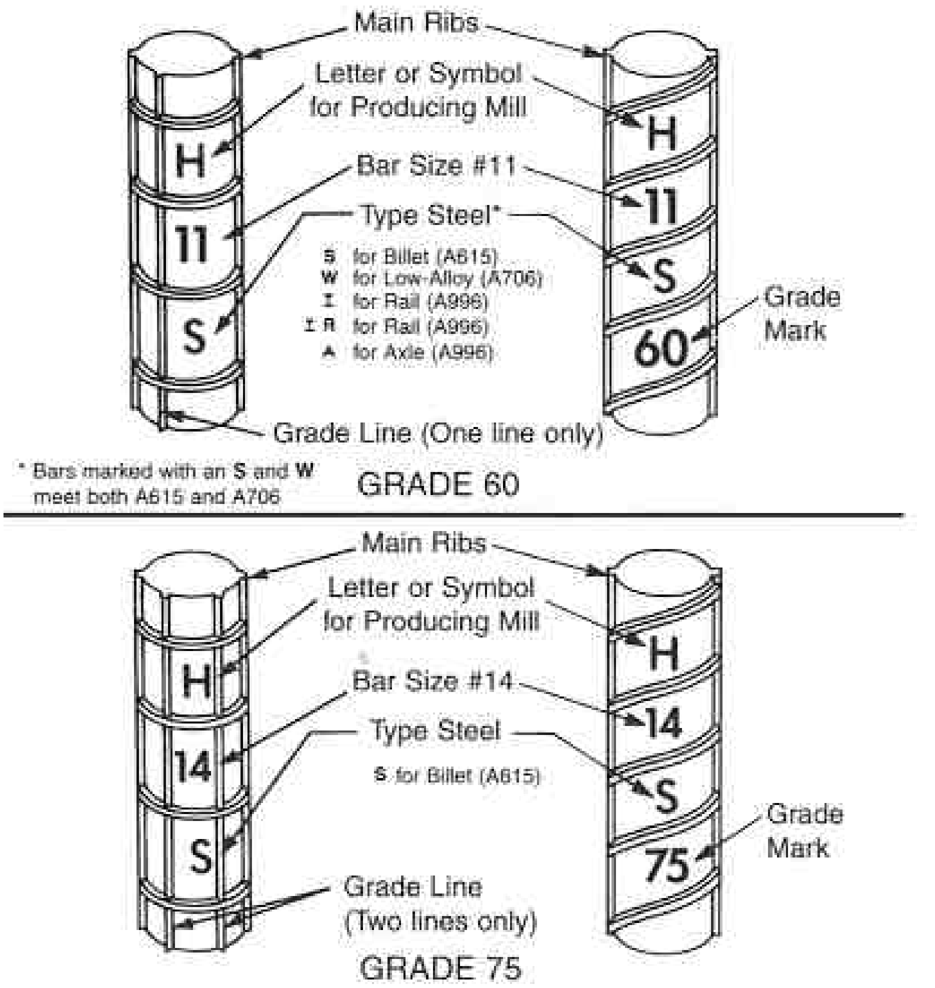


Figure 6.2.5 Concrete Member with Tensile Steel Reinforcement Showing

Shear reinforcement is also needed to resist diagonal tension (refer to Topic 3.1). Shear cracks start at the bottom of concrete members near the support and propagate upward and away from the support at approximately a 45 degree angle. Vertical or diagonal shear reinforcement is provided in this area to intercept the cracks and to stop crack initiation and propagation.

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 6.2.6). The gripping pattern improves bond with the surrounding concrete. Modern reinforced concrete bridges are constructed with "deformed" reinforcing steel.



INCH-POUND BAR SIZE	DIAMETER (in.)	AREA (in.^{2})
#3	0.375	0.11
#4	0.500	0.20
#5	0.625	0.31
#6	0.750	0.44
#7	0.875	0.60
#8	1.000	0.79
#9	1.128	1.00
#10	1.270	1.27
#11	1.410	1.56
#14	1.693	2.25
#18	2.257	4.00

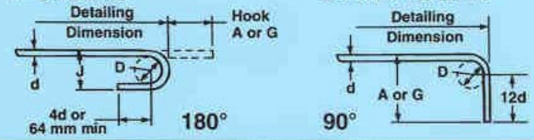
Figure 6.2.6 Standard Deformed Reinforcing Bars (Source: Concrete Reinforcing Steel Institute)

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).

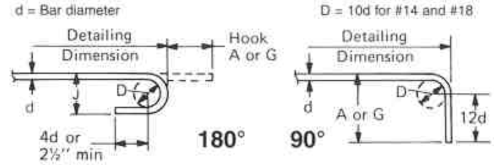
STEEL TYPE	BAR SIZE RANGE	GRADE	MINIMUM YIELD, MPa	MINIMUM TENSILE, MPa
Billet A615M	#10 - #19 #10 - #57 #19 - #57	300 420 520	300 420 520	500 620 690
Low-Alloy A706M	#10 - #57	420	420	550
Rail & Axle A996	#10 - #25 #10 - #25 #10 - #25	300 350 420	300 350 420	500 550 620

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 #3 - #18 #6 - #18	40 60 75	40 60 75	70 90 100
Low-Alloy A706	#3 - #18	60	60	80
Rail & Axle A996	#3 - #8 #3 - #8 #3 - #8	40 50 60	40 50 60	70 80 90

Figure 6.2.7 Standard Deformed Reinforcing Bars (Source: Concrete Reinforcing Steel Institute) (Continued)

In US units, reinforcing bars up to 1-inch 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 of an 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-inch square bars, respectively.

Reinforcing bars can also be used to increase the compressive strength of a concrete member. When reinforcing bars are properly incorporated 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.

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. Stainless steel reinforcement is another type of reinforcement bar that allows protection from the adverse environments. Stainless steel reinforcement has greater corrosion resistance than that of conventional reinforcement and has an estimated service life of 100 years.

Fiber Reinforced Polymer Applications

Deterioration in concrete members is primarily caused by corrosion of conventional reinforcement. Fiber reinforced polymer (FRP) composite reinforcement is becoming increasingly popular since it does not corrode like conventional reinforcement. See Topic 6.6.1 for a detailed description of fiber reinforced polymer reinforcement applications.

6.2.5 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 under primary live loads 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 6.2.8).

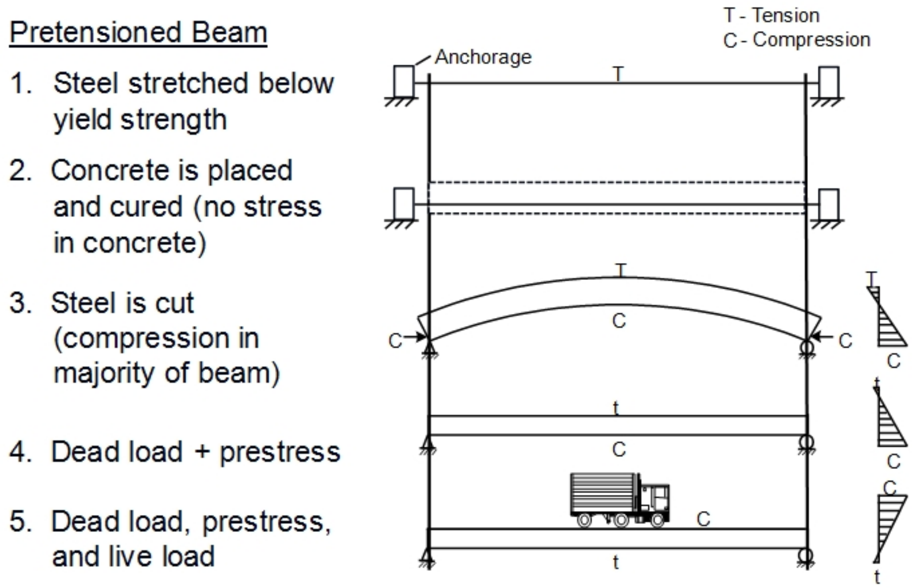


Figure 6.2.8 Prestressed Concrete Beam

Prestressing Methods

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 6.2.9)
- 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 6.2.10)
- Combination method - this is used for long members for which the required prestressing force cannot safely be applied using pretensioning only or construction method such as for a spliced bulb-T which would require post-tensioning

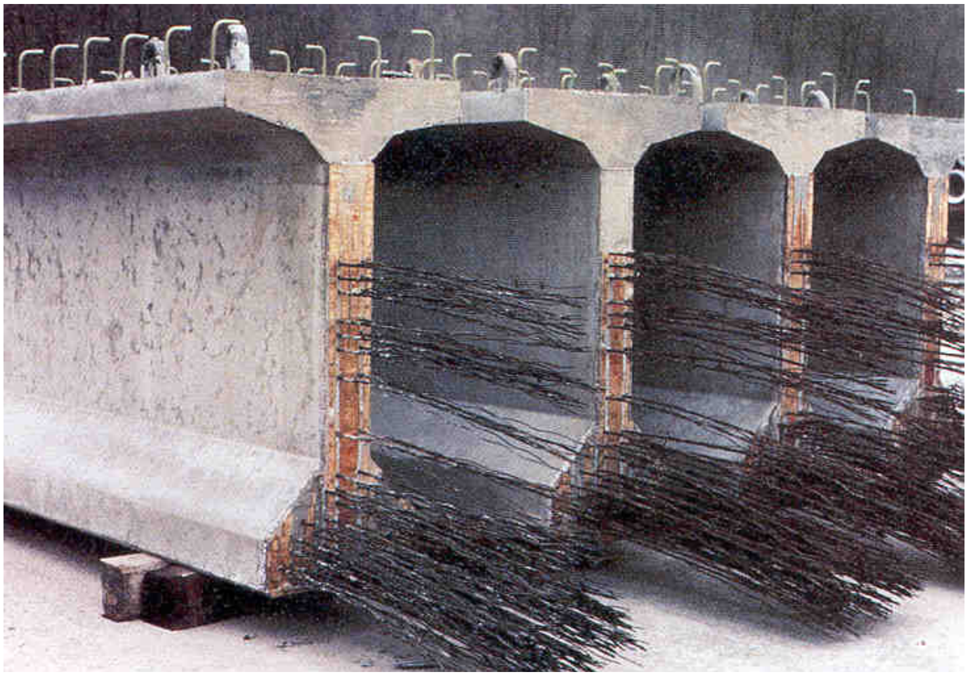


Figure 6.2.9 Pretensioned Concrete I-Beams

Figure 6.2.10 Post-tensioned Concrete Box Girders

Prestressing Reinforcement

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 prestressing operations; the most popular wire size is 1/4-inch diameter and the most common grade of steel is the 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 270 ksi grade is most commonly used today.

- Bars (ASTM A322 and A29) - high tensile strength bars typically have a minimum ultimate stress of 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 145 degrees Fahrenheit and above. This slip occurs because the epoxy material begins to melt at these temperatures. Since 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.

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. Crack control at the beam ends may also be obtained by using draped strands. A number of strands are draped from both ends of the beam to the beam’s third points resulting in end strand patterns with center of gravities near the beam center of gravity.

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 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.

6.2.6 Anticipated Modes of Concrete Deficiencies

In order to properly inspect a concrete bridge, the inspector needs to be able to recognize the various types of deficiencies associated with concrete. The inspector also needs to understand the causes of the deficiencies and how to examine them. There are many common deficiencies that occur on reinforced concrete bridges:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali-Silica Reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation
- Other causes (temperature changes, chemical attack, moisture absorption, differential foundation movement, design and construction deficiencies, unintended objects in concrete, fire damage)

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 conventionally 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. There are two types of structural cracks: flexure and shear (see Figure 6.2.11). Cracks caused by dimensional changes due to shrinkage or temperature are considered nonstructural cracks.

Flexure Cracks

Flexure cracks are considered structural cracks and 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 5.1.9), 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 6.2.12). If the beams are continuous span structures (refer to Topic 5.1.9), flexure cracks can also occur at the top of members at or near their interior supports.

Shear Cracks

Shear cracks are considered structural cracks and 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 6.2.13). 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.

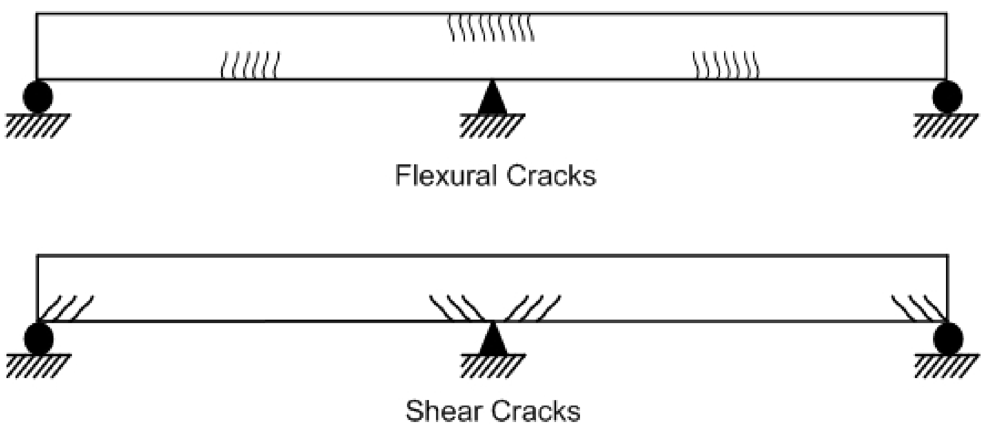


Figure 6.2.11 Structural Cracks

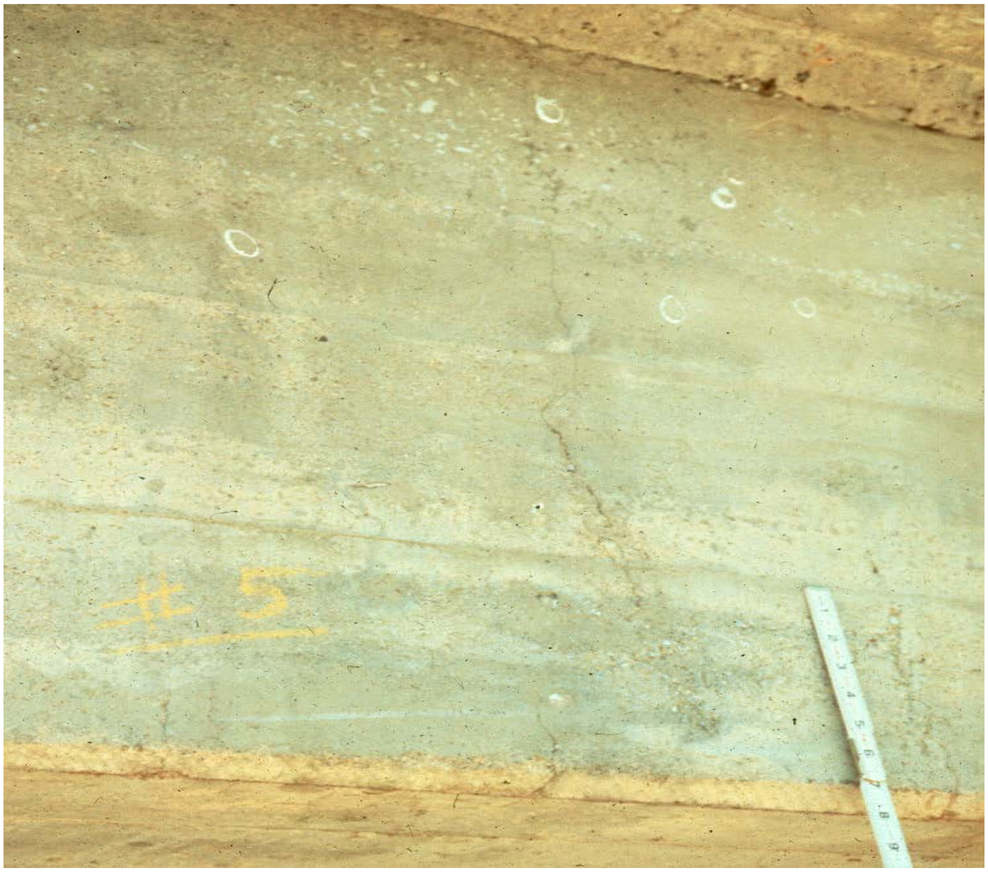


Figure 6.2.12 Flexural Crack on a Tee Beam

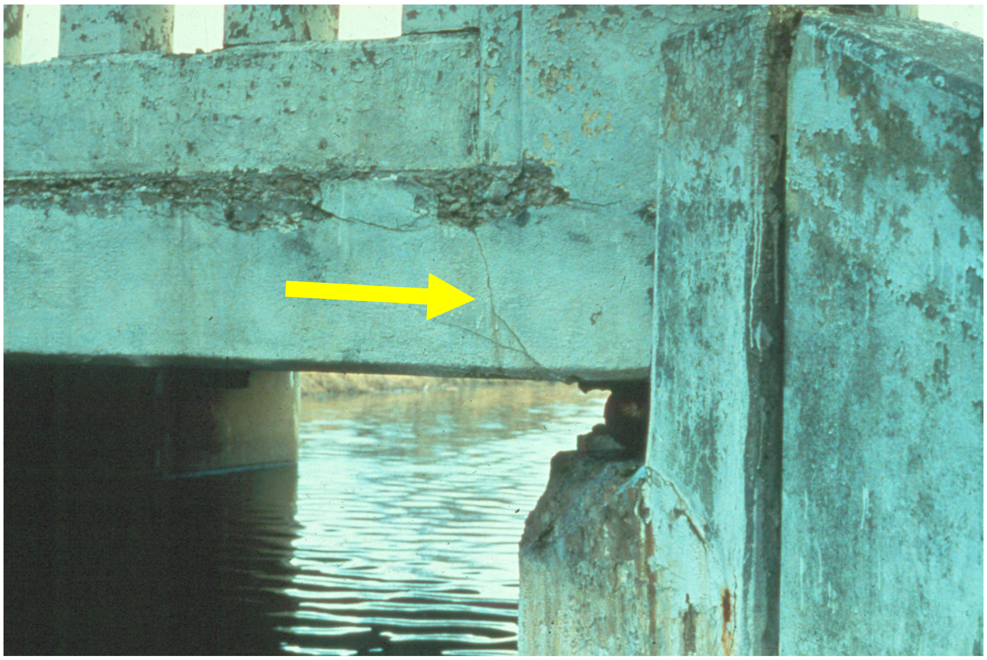


Figure 6.2.13 Shear Crack on a Slab

Crack Size

Crack size is very important in assessing the condition of an in-service bridge. A crack comparator card can be used to measure and differentiate cracks (see Figure 6.2.14).

Figure 6.2.14 Crack Comparator Card

According to the Unpublished Draft Guidelines for NCHRP Project 12-82, *Developing Reliability-Based Bridge Inspection Practices*, "engineering judgment [is] exercised in determining whether any present flexural cracking is moderate to severe. Crack widths in reinforced concrete bridges exceeding 0.006 inches to 0.012 inches reflect the lower bound of moderate cracking. The American Concrete Institute Committee Report 224R-01 presents guidance for what could be considered reasonable or tolerable crack widths at the tensile face of reinforced concrete structures for typical conditions. These range from 0.006 inches for marine or seawater spray environments to 0.007 inches for structures exposed to de-icing chemicals, to 0.012 inches for structures in a humid, moist environment.

In prestressed concrete bridge structural elements, tolerable crack […] criteria [has] been adopted in the Precast Prestressed Concrete Institute (PCI) MNL-37-06, *Manual for the Evaluation and Repair of Precast Prestressed Concrete Bridge Products*. The PCI Bridge Committee recommends that flexural cracks greater in width than 0.006 inches […] be evaluated to affirm adequate design and performance.

Generally, cracking in prestressed elements is more problematic than cracking in reinforced concrete elements.

In cases where flexural cracking is minor or appropriate assessment has indicated that the cracking is not affected the adequate load capacity of the element, the cracking provides pathways for the ingress of moisture and chlorides that may cause corrosion of the embedded steel. This attribute is intended to consider the increased likelihood of corrosion resulting from the cracking in the concrete."

Record the length, width, location, and orientation (horizontal, vertical, or diagonal) when reporting cracks. Carefully record cracks in main members or primary members. Document if the crack extends partially or completely through the member. Indicate the presence of rust stains or efflorescence or evidence of possible reinforcement section loss.

Nonstructural Cracks

Nonstructural cracks result from internal stresses due to dimensional changes. Nonstructural cracks are divided into three categories:

- Temperature cracks (see Figure 6.2.15)
- Shrinkage cracks (see Figure 6.2.16)
- 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 6.2.15 Temperature Cracks

Figure 6.2.16 Shrinkage Cracks

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

Structural cracks are usually oriented perpendicular to their stresses (i.e. tension or shear). Nonstructural cracks such as 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.

In addition to classifying cracks as either structural or nonstructural and recording their lengths and widths, 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 is to 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 6.2.17).

- Longitudinal cracks - These are fairly straight cracks that run parallel to the centerline of the bridge or a bridge member (see Figure 6.2.18).

- 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 6.2.19).

- 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 6.2.17 Transverse Cracks

Figure 6.2.18 Longitudinal Cracks

Figure 6.2.19 Pattern or Map Cracks

Scaling

Scaling, also known as surface breakdown, 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 1/4-inch deep, with surface exposure of coarse aggregates (see Figure 6.2.20)
- Medium or moderate scale - loss of surface mortar from 1/4- inch to 1/2-inch deep, with mortar loss between the coarse aggregates (see Figure 6.2.21)
- Heavy scale - loss of surface mortar from 1/2-inch to 1-inch deep; coarse aggregates are clearly exposed (see Figure 6.2.22)
- Severe scale - loss of coarse aggregate particles, as well as surface mortar and the mortar surrounding the aggregates; depth of the loss exceeds 1 inch; reinforcing steel is usually exposed (see Figure 6.2.23)

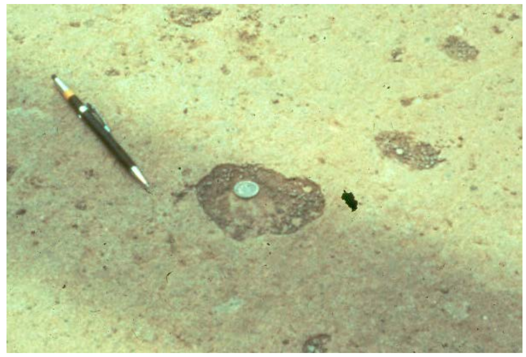


Figure 6.2.20 Light or Minor Scaling

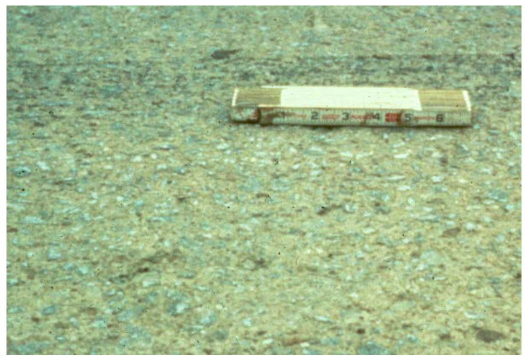


Figure 6.2.21 Medium or Moderate Scaling

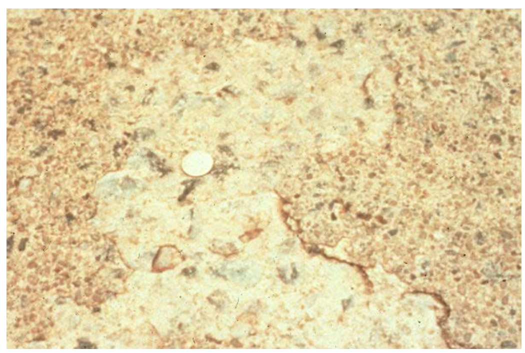


Figure 6.2.22 Heavy Scaling

Figure 6.2.23 Severe Scaling

When reporting scaling, note the location of the deficiency, the size of the affected area, and the scaling classification. For severe scale, the depth of penetration of the deficiency will also be recorded.

Delamination

Delamination occurs when layers of concrete separate at or near the level of the outermost layer of reinforcing steel. The major cause of delamination is expansion of corroding reinforcing steel causing a break in the bond between the concrete and reinforcement. 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 or chain drag. When a delaminated area completely separates from the member, the resulting depression is called a spall.

When reporting delamination, note the location and the size of the area.

Spalling

A spall is a depression in the concrete (see Figure 6.2.24). 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 1 inch deep or approximately 6 inches in diameter
- Large spalls - more than 1 inch deep or greater than 6 inches in diameter

Figure 6.2.24 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 are to 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 crush and then spall.

When reporting spalls, note the location of the deficiency, the size of the area, and the depth of the deficiency.

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. During the 1960's, salt was added to water to prevent it from freezing during mixing and fabrication. Practices like these are no longer acceptable. Various admixtures are incorporated to account for adverse weather condition. This practice causes accelerated reinforcement corrosion that leads to cracking of the concrete.

Freeze-Thaw

Freeze-thaw is the freezing water within the capillaries and pores of cement paste and aggregate resulting in internal overstressing of the concrete, which leads to deterioration including cracking, scaling, and crumbling. Pore pressure is a phenomenon that occurs during freeze-thaw which causes the deterioration and expansion 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 6.2.25). In order to estimate the percent of concrete contaminated by chloride, nondestructive testing is required (see Topic 15.2.2).

Figure 6.2.25 Efflorescence

Alkali-Silica Reaction

Alkali-Silica Reaction (ASR) is an expansive reaction, forming a gel, which will result in the swelling and expansion of concrete. (See Figure 6.2.26).

The process involves a reaction between potassium and sodium alkalis (common in cement) and silica (common in aggregates). Alkali found in soils, deicers, and chemical treatments could also contribute to ASR. In addition, salts have been also known to accelerate alkali-silica reactions. Moisture can also promote the expansion for a structure already affected by ASR.

Typical indicators are map cracking or scaling and in cases that are more advanced, closed joints and spalled concrete surfaces are typical indicators. Cracking may appear in areas where there is frequent moisture. There is no early indication of ASR visible, so lab testing will be required to confirm presence of ASR.

Even though there is no early detection, there is a process to confirm if ASR is present in concrete which consists of three different levels. First, there is a Level 1 investigation that is performed, which consists of a condition survey performed to evaluate distress. If further investigation is required, a Level 2 investigation will be conducted, which consists of documenting information, measuring the Cracking Index, obtain samples, and conduct a petrographic examination. After the first two levels of investigation are complete and further investigation is required, a Level 3 investigation is performed. Level 3 investigations will include determining the expansion to date, the current rate of expansion, and the potential for future expansion. It will be necessary to perform the test of the structure at the location of the ASR in order to gather all the information required to determine the growth of the ASR (Source: Report on Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures, Publication No. FHWA-HIF-09-004)

Figure 6.2.26 Alkali-Silica Reaction (ASR)

Ettringite Formation

Ettringite formation is an internal deficiency 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 methods 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

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. Honeycombs or construction voids are hollow spaces or voids that may be present within the concrete. Honeycombs are construction deficiencies caused by improper vibration during concrete placement, resulting in the segregation of the coarse aggregates from the fine aggregates and cement paste. This can be attributed to excessive vibration. In some cases, honeycombs are the result of insufficient vibration, where the entire concrete mix does not physically reach the formwork surface (see Figure 6.2.27).

Figure 6.2.27 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 Figure 6.2.28). 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.

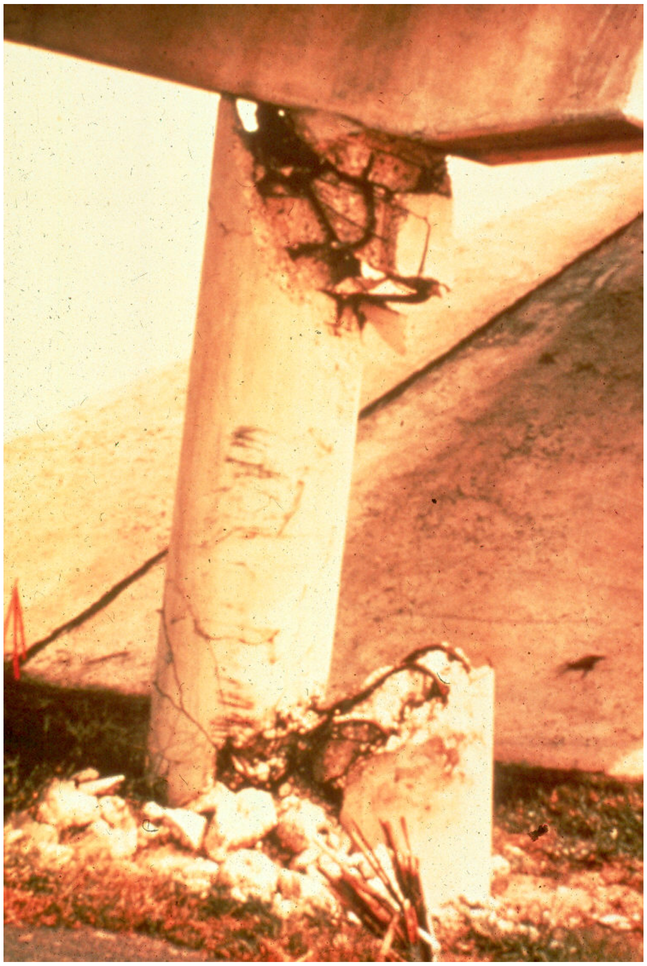


Figure 6.2.28 Concrete Column Collision Damage

Abrasion

Abrasion damage is the result of external forces acting on the surface of the concrete member and is similar to wear (see Figure 6.2.29). 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 (see Topic 13.2).

Figure 6.2.29 Substructure Abrasion

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 due to tension include excessive sagging, spalling, and/or cracking at the mid-span of simple span structures and at the supports of continuous span structures. Diagonal cracks close to support points may be an indication of overstress due to shear or torsion. Permanent deformation is another visual sign of overstress damage in a member. If overload damage is detected or suspected, notify the Chief Bridge Engineer immediately (see Figure 6.2.30).

Figure 6.2.30 Overload Damage

Internal 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 a rebar rust stain 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 3000 psi that will cause the concrete to yield, resulting in wider cracks, delaminations, and spalls (see Figure 6.2.31).

Figure 6.2.31 Loss of Bond: Concrete / Corroded Reinforcing Bar

Loss of Prestress

Prestressed concrete members deteriorate in a similar fashion to conventionally reinforced concrete members. However, the effects on prestressed concrete member performance are usually more detrimental. Significant deficiencies 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.

Forensic studies after a 2005 prestress beam failure determined that nominal tensile stress can be reduced to approximately 20 to 30 percent based on the condition of the prestress strands. Lightly corroded strands experienced a 20 percent reduction while heavy pitted strands experienced a 29 percent reduction in nominal tensile stress.

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 methods.

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.

Carbonation

Carbonation is a chemical reaction between carbon dioxide in the air with calcium hydroxide and hydrated calcium silicate in concrete. Carbon dioxide will react with the alkali in the cement which makes the pore water become more acidic, lowering the pH. The carbon dioxide will then start to carbonate the moment the concrete element is fabricated. Carbonation will start at the surface and move slowly deeper into the concrete, eventually reaching the reinforcement. Once it reaches the reinforcement, corrosion will begin to occur.

Other Causes of Concrete Deterioration

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.

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 substructures. Differential settlement induces stresses in the supported superstructure and can lead to concrete deterioration. Cracks due to differential foundation movement are normally oriented in a vertical or diagonal direction.

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 which will result in cracking, delaminations and spalls.
- 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) and excessive shrinkage cracking.
- 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.
- 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.

- Over finishing – This can lead to early scaling.

Unintended Objects in Concrete

Some items discovered in concrete: screws, nails, tools, trash, paper, soft drink cans, etc. Objects like these can create voids and collect water or corrode and deteriorate concrete.

Fire Damage

Extreme heat will damage concrete. High temperatures (above 700 degrees Fahrenheit) will cause a weakening in the cement paste and lead to cracking.

6.2.7
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 sealers.

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. Consult the paint manufacturers prior to covering concrete to determine the best possible paint based upon the expected weather/exposure conditions and concrete type.

Several classes of paint are used to coat 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, measure the concrete moisture 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 6.2.32). 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 6.2.32 Anti-Graffiti Coating on Lower Area of Bridge Piers

Water Repellent Sealers

Water repellent membranes (sealers) applied to concrete bridge decks, piers, abutments, columns, barriers, or aprons form a tight barrier to water and chlorides. The sealer penetrates up to 3/8 of an inch into the concrete to give strong adhesion. Sealers have good resistance to abrasion from weathering and traffic. Methyl methacrylate, silane, and silicone are three common water repellent sealers.

Types and Characteristics of Reinforcement Coatings

Because unprotected steel reinforcement corrodes and has adverse effects on concrete, some type of protective coating is used on steel reinforcement placed in concrete structures to ensure minimal steel corrosion. Steel reinforcement can be protected by the following methods:

- Epoxy coating
- Galvanizing
- Stainless steel cladding
- Cathodic protection
- Anodic 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.

Stainless Steel Cladding

Stainless steel cladding is another form of steel reinforcement protection that is resistant to corrosion. The chromium (Cr) in the coating will form a passivation layer chromium oxide (Cr_2O_3) when exposed to oxygen which is not visible to the naked eye. The coating will protect the reinforcement from water and air and the coating will quickly reform if the surface is scratched.

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.

Anodic Protection

Much like cathodic protection, this can help slow down the corrosion process of reinforcement. This is achieved by having a metal structure anode with a low voltage direct current so it can achieve and maintain an electrochemically passive state. Anodic protection can be more suitable than cathodic protection for reinforcement that is located in extremely corrosive environments. However, this will require careful monitoring and control, otherwise, it could hurry up the corrosion process.

6.2.8 Inspection Methods for Concrete and Protective Coatings

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

- Visual
- Physical
- Advanced inspection methods

Visual Examination

There are two types of visual inspections that may be required of an inspector. The first, called a routine inspection, involves reviewing the previous inspection report and visually examining the members from beneath the bridge. Give all concrete surfaces a thorough visual assessment to identify obvious surface deficiencies during a routine inspection.

The second type of visual inspection is called an in-depth inspection. An in-depth inspection is an inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine methods. Hands-on inspection may be necessary at some locations. This type of visual inspection requires the inspector to visually assess all 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 deficiencies. The hands-on inspection method may be supplemented by nondestructive testing.

Physical Examination

Physically examine areas of concrete or rebar deterioration that are identified visually by using an inspection hammer. This hands-on effort verifies the extent of the deficiency and its severity.

Sound high stress areas for deficiencies 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 (see Figure 6.2.33). 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.

Give special attention to The location, length and width of cracks found during the visual inspection and sounding methods. 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 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, use a crack gauge, which is a more accurate crack width-measuring device.

Figure 6.2.33: Inspector Using a Chain Drag

Advanced Inspection Methods

If the extent of the concrete deficiency cannot be determined by the visual and/or physical inspection methods described above, advanced inspection methods may be used. Nondestructive methods, described in Topic 15.2.2, include:

- Acoustic Wave Sonic/Ultrasonic Velocity Measurements
- Electrical Methods
- Delamination Detection Machinery
- Ground-Penetrating Radar
- Electromagnetic Methods
- Pulse Velocity
- Flat Jack Testing
- Impact-Echo Testing
- Infrared Thermography
- Laser Ultrasonic Testing
- Magnetic Field Disturbance
- Neutron Probe for Detection of Chlorides
- Nuclear Methods
- Pachometer

- Rebound and Penetration Methods
- Ultrasonic Testing
- Smart Concrete
- Carbonation

Other methods, described in Topic 15.2.3, include:

- Concrete Permeability
- Concrete Strength
- Endoscopes and Videoscopes
- Moisture Content
- Petrographic Examination
- Reinforcing Steel Strength
- Chloride Test
- Matrix Analysis
- ASR Evaluation

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 collision damage and coating removal 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.

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Bridge Materials

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Steel Description	Steel Designation		Years in Use
	American Society for Testing and Materials (ASTM)	American Association of State Highway and Transportation Officials (AASHTO)	
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)			
High Performance Steel	A709 (HPS 50W, HPS 70W, HPS 100W)	M270 (HPS 50W, HPS 70W, HPS 100W)	1996-Present

Topic 6.3 Steel/Metal

6.3.1 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 needs to be familiar with the various properties and types of steel.

6.3.2 Common Methods of Steel Member Fabrication

Rolled Shapes

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 36 inches in depth but are now available from some mills as deep as 44 inches.

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. Although rolled beams may not incorporate intermediate stiffeners, they will have connection plates for diaphragms or cross-frames.

Built-Up Shapes

Plate girders are often specified when the design calls for members deeper than 36 or 44 inches.

Plate girders are built-up shapes 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.

6.3.3 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 the most efficient form of steel for a tensile capacity per pound basis. Wires are typically used as prestressing strands or tendons in beams and girders (See Figure 6.3.1).

- Cables can be fabricated from steel wire rope, parallel wires or seven wire strands. Cable-stay and steel suspension bridges are primarily supported by steel cables (see Figure 6.3.2).

- Steel plates have a wide variety of uses. They are primarily used to construct built-up shapes (see Figures 6.3.3 and 6.3.4).

- Steel bars are generally placed in concrete to provide tensile reinforcement in the form of deformed round bars (see Figure 6.3.5). Steel bars can also be used as primary members such as eyebars in older trusses or arches (see Figure 6.3.6) or secondary tension members.

- 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 6.3.7). 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 6.3.8).

Figure 6.3.1 Prestressing Strands for a Box Beam

Figure 6.3.2 Steel Cables with Close-up of Cable Cross-Section

Figure 6.3.3 Steel Plate Welded to Girder

Figure 6.3.4 Welded I-Girder

Figure 6.3.5 Steel Reinforcement Bars

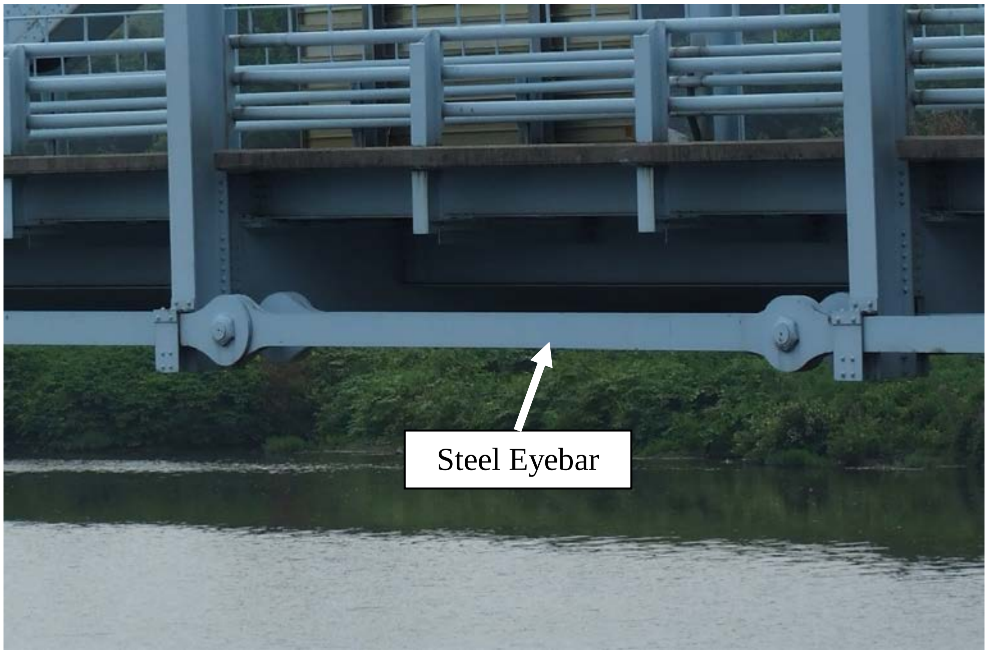

Figure 6.3.6 Steel Eyebar

Figure 6.3.7 Rolled Beams

Figure 6.3.8 Built-up Girder

6.3.4 Properties of Steel

Physical Properties

When compared with iron, steel has greater strength characteristics, it is more elastic, and can withstand the effects of impact and vibration better.

Iron consists of small amounts of carbon. However, when the carbon content is between 0.l% and 2.1%, the material is classified as steel. Steel has a unit weight of about 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 - first used in 1960 which features good weldability and improved strength and replaced A7 as the “workhorse” bridge steel; now specified as A709, Grade 36
- A709 steel – the current overall umbrella specification used in bridge construction which was developed in 1974

  A709 steel covers carbon and high-strength alloy steel structural shapes, plates and bars, and quenched and tempered alloy steel for structural plates intended for use in bridges. There are six grades 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.

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 - a "weathering steel," was developed to be left unpainted, which develops a protective oxide coating (patina) upon exposure to the atmosphere under proper design and service conditions (refer to Topic 6.3.6 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.

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 6.3.

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 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 6.3.5 – for specific temperature information)

- Corrosion resistance - unprotected carbon steel corrodes (i.e., rusts) readily; however, steel can be protected by coating, plating or adding weathering components to the alloy
- Weldability – today’s 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

- Fracture toughness for mechanical property

High Performance Steel

In 1996, a new steel type, High Performance Steel (HPS), was introduced to bridge construction (see Figure 6.3.9). 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 A709 HPS 70W, which was produced by Thermo-Mechanical-Controlled Processing (TMCP). The new high performance steels exhibit enhanced weldability, fracture toughness, and corrosion resistance properties over the more common low carbon steels. Currently the HPS grades available are HPS 50W, HPS 70W, and HPS 100W and like low carbon steels, the “W” stands for weathering steel. Bridge girders may be constructed as ‘hybrid’ members (70 ksi flanges and 50 ksi webs). This results in a more shallow girder which helps with low clearance problems.

Figure 6.3.9 High Performance Steel Bridge

6.3.5 Anticipated Modes of Steel Deficiencies

Corrosion

To properly inspect a steel bridge, the inspector will have to be able to recognize the various types of steel deficiencies and deterioration. The inspector will also have to understand the causes of the deficiencies and how to examine them. The most recognizable type of steel deficiency is corrosion (see Figure 6.3.10). Bridge inspectors will have to be familiar with corrosion since it can lead to a substantial section reduction resulting in decreased 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 corrosion is accelerated.

Figure 6.3.10 Steel Corrosion and Complete Section Loss on Girder Webs

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 chemical 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
- Pack rust – occurs between two mating surfaces of elements; an increase in volume of rust over the original steel will create localized distortion and possibly cracking
- Crevice corrosion – occurs between adjacent surfaces, but the rust may not expand

Fatigue Cracking

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 (see Figure 6.3.11). This type of cracking can lead to sudden and catastrophic failure on certain bridge types. Therefore, the bridge inspector needs to know where to look and how to recognize early stages of fatigue crack development.

Figure 6.3.11 Fatigue Crack

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 may be referred to as distortion induced fatigue. Out-of-plane distortion is common in beam webs where transverse members, such as floorbeams, connect and can lead to fatigue cracking (see Figure 6.3.12).

There is a distinction between fatigue that is caused from in-plane (as designed) bending and out-of-plane distortion.

Additional information about fatigue and fracture in steel bridges is presented in Topic 6.4.

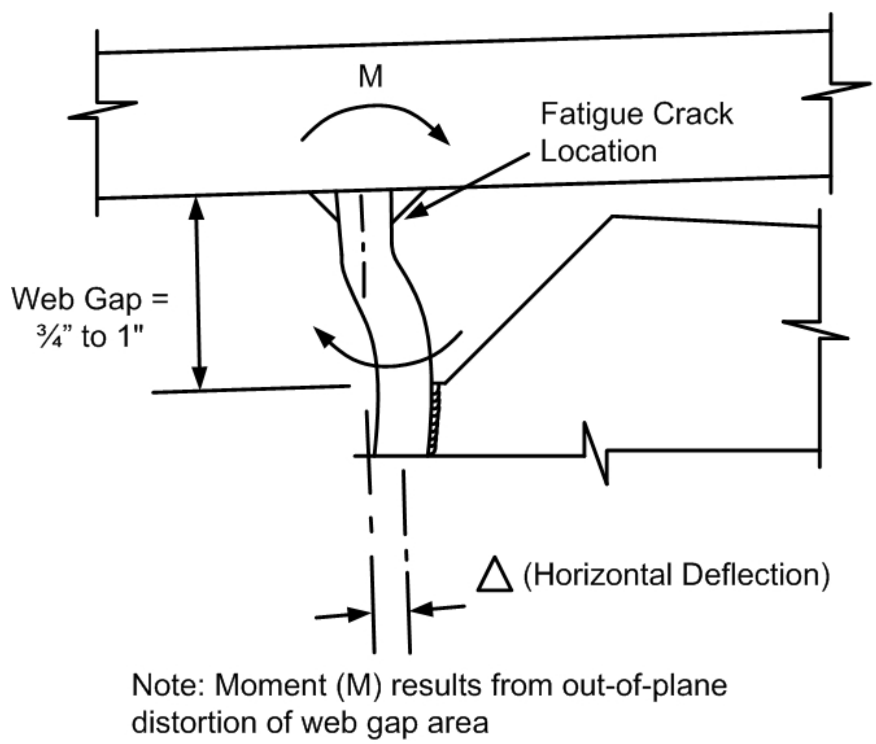

Figure 6.3.12 Distortion Induced Fatigue

Overloads

Loads that exceed member or structure design are known as overloads. 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 5.1). When yield occurs, steel will bend or elongate and remain distorted 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 near 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 traffic are susceptible to collision damage. Indications of collision damage include broken, dislocated or distorted members (see Figure 6.3.13).

Figure 6.3.13 Collision Damage on a Steel Bridge

Heat Damage

Steel members undergo serious deformation when exposed to extreme heat (see Figure 6.3.14). 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 6.3.14 Heat Damage

Temperatures affecting steel strength commonly used in bridges are as follows:

- 600°-1000°F - starts to affect strength
- Above 1000°F - major loss of strength

Once steel is subjected to heat, the yield strength and modulus of elasticity are relatively constant and can be reduced to approximately 90% of its value up to 600°F. Between 600°F and 1000°F, the yield strength will then further be reduced down to approximately 75% of its yield strength. The modulus of elasticity for steel will be reduced down to 75% at 1000°F. Temperatures above 1000°F will significantly reduce the strength properties of steel.

Coating Failures

The following coating failures are common on steel:

- Chalking, erosion, checking, cracking, and wrinkling caused by too much paint (see Figure 6.3.15), as described in ASTM D-3359.
- Blisters are caused by painting over surface contaminants such as: 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 6.3.16). 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 6.3.17). 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 6.3.18). 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.

- 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 6.3.19). Mudcracking is often a phenomenon of inorganic zinc-rich primers, which have been applied too thick 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.

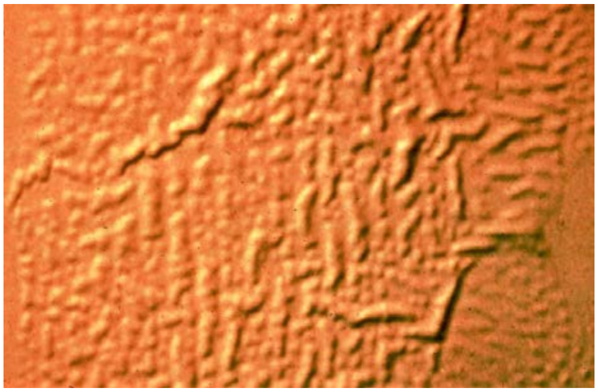

Figure 6.3.15 Paint Wrinkling

Figure 6.3.16 Rust Undercutting at Scratched Area

Figure 6.3.17 Pinpoint Rusting

Figure 6.3.18 Paint Peeling from Steel Bridge Members

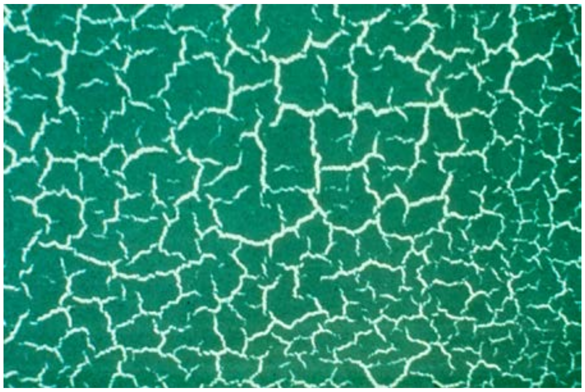

Figure 6.3.19 Mudcracking Paint

6.3.6 Protective Systems

Function of Protective Systems

Protective systems, when applied properly, provide active or passive protection against rust or corrosion by providing a barrier, a sacrificial anode (galvanic action) or both**.** Protective systems may be passive or active.

Passive systems include paint, metalizing or galvanizing through galvanic action, or weathering steel patina. These systems are self-functioning and operate through a protective film that either isolates the base metal (paint and weathering steel) or accelerates corrosion of the coating, thereby slowing or stopping corrosion of the base metal (galvanic action).

Active systems include cathodic protection systems. These systems incorporate an external current source and artificial anode mesh or coating, together which reverses the current of the system and prevents electron loss of the steel or metal. Because corrosion only occurs at the electron-losing anode (steel or metal), the reversed current turns the steel or metal into a giant cathode, which does not corrode. The anode mesh or coating is also spared from corrosion, since the system utilizes artificially created electrons instead of electrons from the mesh or coating.

A thorough understanding of the steel corrosion process will help in the inspection of protective systems on bridge members.

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.

The following are required to promote corrosion in steel:

- 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.

Paint

Paint is the most common passive system coating used to protect steel bridges. Paint is composed of four basic compounds: pigments, resin (also called binder), solvents (also called thinners), and additives (such as thickeners and mildewcides). 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 deficiency by sunlight, color, and hardness. Pigments are generally powders before being mixed into paint. The resin 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 resin (binder). 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 (oxidation)
- Polymerization - two or more resin molecules combine to form a single more complex molecule. Components when mixed react chemically to form a solid, rigid, resistant coating film.
- Moisture cured - reaction between resin system and atmospheric moisture.

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, mildewcides reduce mildew problems, and thickeners lengthen the drying time for application in hot weather. Polymerization - Two or more resin molecules combine to form a single more complex molecule. Components when mixed react chemically to form a solid, rigid, resistant coating film.

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 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 is designed to 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 are not designed to be used in water immersion service or in alkaline environments.
A disadvantage of this paint type is the offensive odor during application. 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 heavily regulated activity in all states and can make maintenance activities very costly.

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 should be removed prior to top coating 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 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 should 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.

Less durable than some other coatings and less flexibility in application temperature tolerances.

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 maintenance can be chosen to provide long lasting results.

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.

Metalizing

Metalizing is a thermal spray application of a protective coating, typically zinc or zinc/aluminum. The coating can be applied in more extreme temperature ranges than most paints, by generally requires a higher degree of surface cleanliness and irregular surface profile. The coating is generally top-coated with a sealer to provide longer protection.

Galvanizing

Galvanizing is a technique of coating metal generally accomplished by hot-dipping the metal. The coating is primary zinc, which then reacts with the environment to form a protective zinc oxide that prevents corrosion of the steel.

Weathering Steel Patina

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 patina 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.

It is common to find coating systems applied to the ends of weathering steel members near expansion joints and over substructure units. These systems minimize staining that may be associated with weathering steel.

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, to accumulate on the superstructure caused by high-speed traffic passing under a low clearance bridge
- 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

Protection of Suspension Cables and Stayed 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 should 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 40%.

6.3.7 Inspection Methods for Steel

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

- Visual
- Physical
- Advanced inspection methods

Visual Examination

Steel Members

Inspect steel members for corrosion, section loss, buckling, and cracking.

In the inspection records, identify the location of Fracture Critical Members (FCMs) and describe the FCM inspection frequency and procedures. Inspect FCMs according to the AASHTO *Manual for Bridge Evaluation*. See Topic 6.4 for detailed description of inspection procedures of fracture critical members.

Some common steel bridge inspection locations and signs of distress include:

- Bent or deficiency members - determine the type of deficiency (e.g., collision, overload, or fire), inspect for proper alignment, and check for cracks, tears, and gouges near the deficiency 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 6.3.20)
- Fatigue prone details - fatigue cracks may occur at certain locations on a bridge, and certain inspection procedures need to be followed when fatigue cracks are observed (see Figure 6.3.21 and Topic 6.4 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
- Problematic details: welded or mechanical connections; look for cracks in the paint, cracks in the steel
- Repairs that show indiscriminate welding or cutting procedures
- Areas of excessive vibrations or twisting

Inspection procedures for observed 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
- If a suspect area is located, a more detailed advanced inspection method may be required (see Topic 15.3).

Figure 6.3.20 Corrosion of Steel

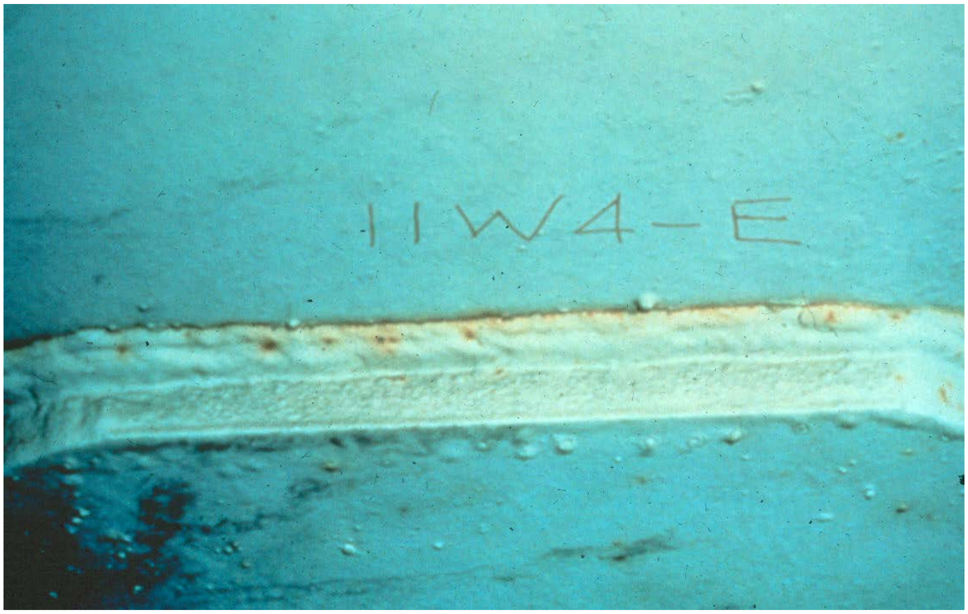

Figure 6.3.21 Fatigue Crack

Protective Coatings

Rust typically starts in a few characteristic places such as horizontal surfaces where water, dirt and debris accumulate, then spreads to larger areas.

Examine sharp edges and square corners of structural members (see Figure 6.3.22). 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 paint 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.

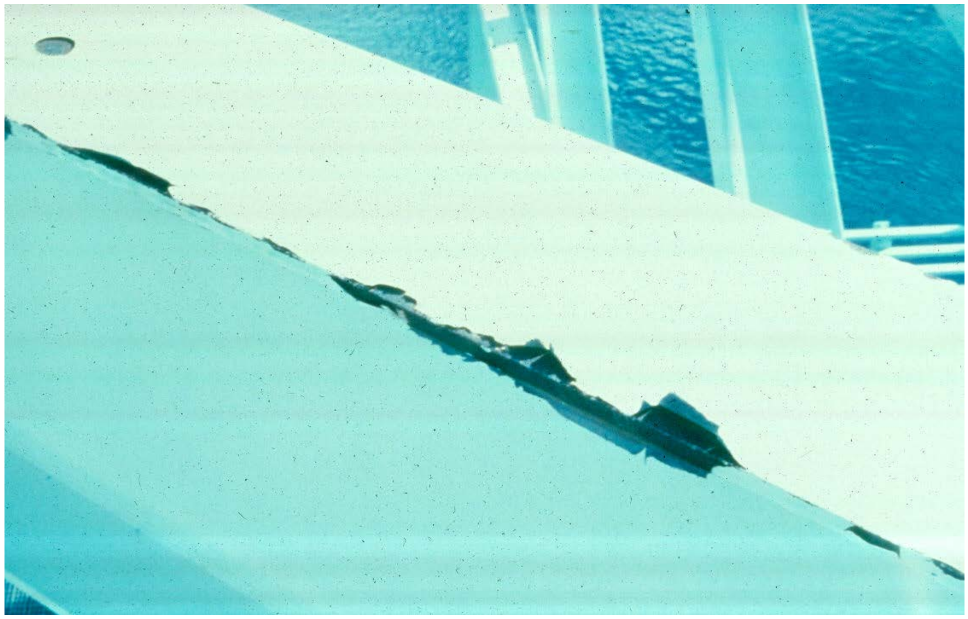

Figure 6.3.22 Paint Failure on Edge of Steel Truss Member

Examine all areas that retain moisture and salt. Check under scuppers and beneath downspouts and under expansion joints. Check horizontal surfaces under the edge of bridge decks and under expansion dams, where roadway deicing salt runoff collects (see Figure 6.3.23). Examine the top surfaces of girder bottom flanges.

Inspect inaccessible or hard-to-reach areas that may have been missed during painting. A flashlight and inspection mirror may be needed to visually inspect these areas. Examine the inside surfaces of lattice girders and beams. Examine all horizontal surfaces. These areas trap water and are susceptible to paint failure, corrosion, and section loss.

Inspect around bolts, rivets, and pins (see Figure 6.3.24). 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 or spray zones, where debris and corrosive deicing salt-laden water are directly deposited on painted members by passing traffic (see Figure 6.3.25). 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 6.3.23 Water and Salt Runoff Near Expansion Joint

Figure 6.3.24 Corroding Rivet Head

Figure 6.3.25 Roadway Spray Zone Deficiency

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 6.3.26). The color changes as the oxide film matures to a fully protective coating.

A yellow-orange, for new steel with initial exposure, is acceptable (see Figures 6.3.27 and 6.3.28). For bridges that have been in service for several years, purple brown color is acceptable (see Figure 6.3.29), while flaking steel or black color indicates the improper formation of the protective oxide film (see Figure 6.3.30).

Figure 6.3.26 Color of Oxide Film is Critical in the Inspection of Weathering Steel; Dark Black Color in an Indication of Non-protective Oxide

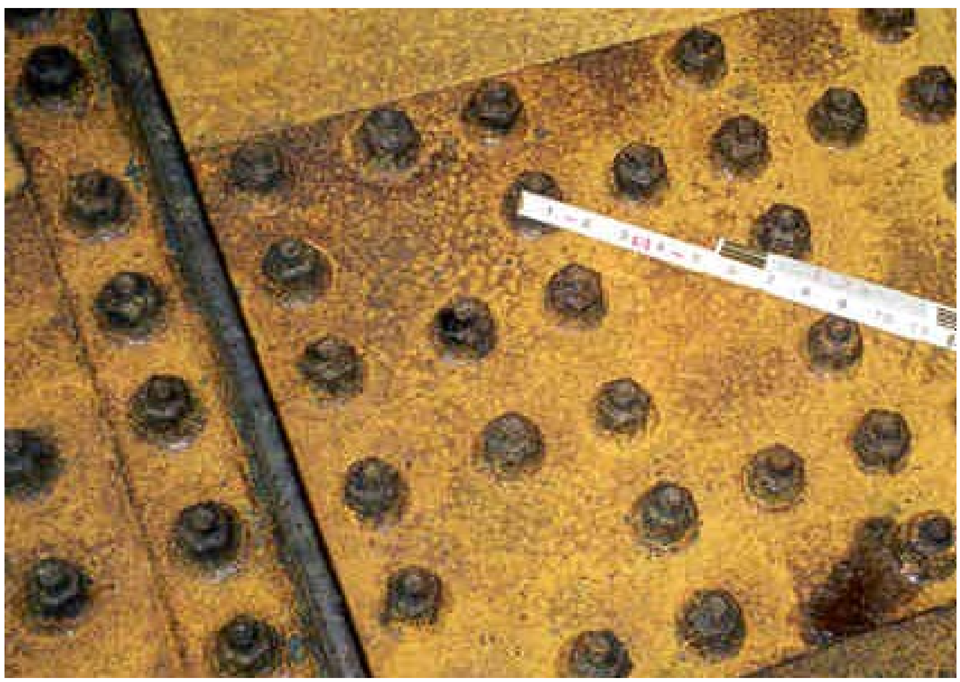

Figure 6.3.27 Yellow Orange – Early Development of the Oxide Film (Patina)

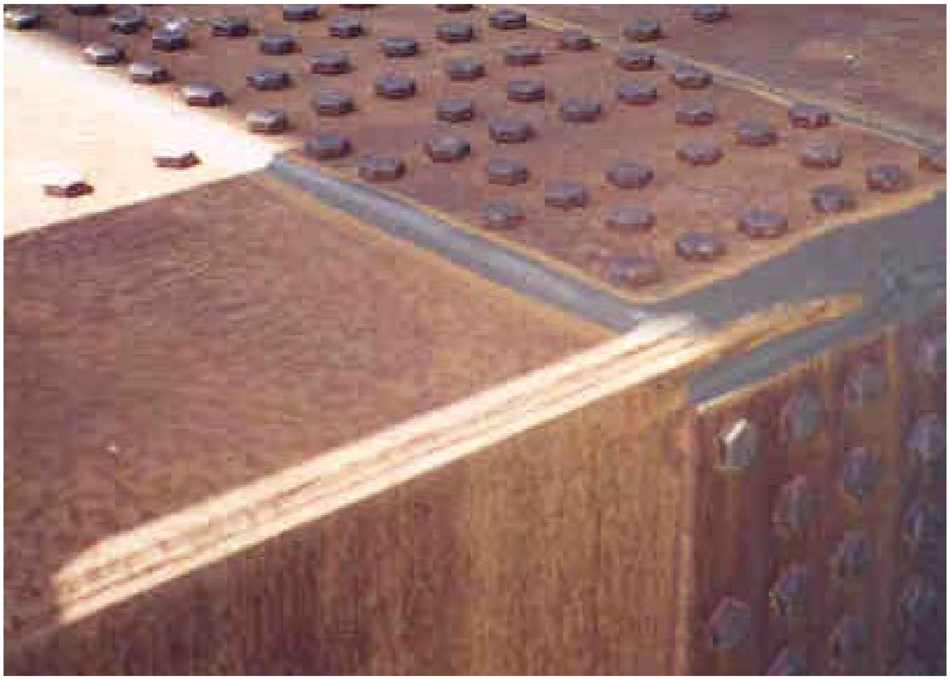

Figure 6.3.28 Light Brown – Early Development of the Oxide Film (Patina)

Figure 6.3.29 Chocolate Brown to Purple Brown - Fully Developed Oxide Film

Figure 6.3.30 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 changing condition of the weathering steel over time. A color coupon may 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. Figure 6.3.31 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, 1/4 inch in diameter	Initial indication of non-protective oxide
Large flakes, 1/2 inch in diameter or greater	Non-protective oxide
Laminar sheets or nodules	Non-protective oxide, severe conditions

Figure 6.3.31 Correlation Between Weathering Steel Texture and Condition

Physical Examination

Steel Members

Once the deficiencies are identified visually, physical methods are used to verify the extent of the deficiency. For steel members, the main physical inspection methods involve the measurement of deficiencies identified visually. An inspection hammer or wire brush is used to remove loose paint or rust flakes so accurate measurements of remaining section can be made. During the removal process, personal protective measurements are taken to ensure the inspector is protected against exposure to potentially hazardous materials. Excessive hammering, brushing or grinding may close surface cracks and make the cracks difficult to find. Corrosion causes in loss of member material. This loss of cross section due to corrosion is known as section loss. Section loss may be measured using a straight edge and a tape measure. However, a more exact method of measurement, such as calipers or an ultrasonic thickness gauge (D-meter), may be used to measure the remaining section of steel. Corrosion products need to be removed to obtain more accurate measurements.

Bridge member dimensions can be field measured and recorded to verify the accuracy of bridge member dimensions shown in plans or sketches. If incorrect member sizes are reported, the load rating analysis for safe load capacity of the bridge will be inaccurate.

Protective Coatings

The degree of coating failure can be assessed during the inspection. There are a variety of proprietary methods which use a set of photographic standards to evaluate and categorize the degree and extent of coating failure. A simple method entails evaluation of painted surfaces in accordance with the Society for Protective Coatings Guide to Visual Standard Number 2 (SSPC-Vis 2) “Standard Method for

Evaluating Degree of Rusting on Painted Steel Surfaces". Vis 2 is a pictorial standard for evaluating the degree of rusting on painted steel surfaces. Other visual comparison manuals can also be used to evaluate coating defects.

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 areas. 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 chemicals 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 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 6.1. The tape test is still qualitative since there is not an actual measurement of adhesion (psi) such as with a pull-off test.

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.

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. The Society for Protective Coatings Paint Application Standard Number 2 (SSPC-PA2) "Measurement of Dry Paint Thickness With Magnetic Gages" provides a detailed description of how to calibrate and take measurements using magnetic gages.

Another method for measuring dry film thickness using the gage described in Topic 6.1. An advantage of this method is that it can be used close to edges. While the magnetic gages measure the combined thickness of all paint layers, the instrument 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 easily 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 6.1 can be used to determine the type of in-service paint.

Weathering Steel Patina

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 (1/8 inch in diameter) texture
- Flakes (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 observed, the following steps may 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 remaining section thickness 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 will 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 Methods

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

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

Other methods for determining material properties described in Topic 15.3.3, include:

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

6.3.8 Other Bridge Materials

Cast Iron

Iron is an elemental metal smelted from iron ore. Iron 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 older 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 approximately 450 pcf.

Properties of Cast Iron

Some of the mechanical properties of cast iron include:

- Strength - tensile strength varies from 25,000 psi to 50,000 psi, while compressive strength varies from 65,000 psi to 150,000 psi
- Elasticity - cast iron has an elastic modulus of 13,000,000 psi to 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 cannot 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 cyclical stresses

Cast Iron Deficiencies

The primary forms of deficiency in cast iron are similar to those in steel.

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 members still exist on some older bridges, and were 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 35,000 psi, while tensile strength varies between 36,000 psi and 50,000 psi
- Elasticity - modulus of elasticity ranges from 24,000,000 psi to 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

Wrought Iron Deficiencies

The primary forms of deficiency in wrought iron are similar to those in steel.

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. However, aluminum has been used to replace iron or steel members for rehabilitation projects. Aluminum weighs less than steel and may allow greater live loads on the rehabilitated structures.

Properties of Aluminum

The properties of aluminum are generally similar to those of steel. However, a few notable differences exist:

- Weight - aluminum alloy has a unit weight of about 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

Aluminum Deficiencies

The primary forms of deficiency in aluminum are:

- Fatigue cracking - the combination of high stresses and vibration caused by cyclic loading
- Pitting - aluminum can pit slightly, but this condition rarely becomes serious
- Corrosion - corrosion in direct contact with fresh concrete if not coated or otherwise protected. Aluminum reacts principally with alkali hydroxides from cement. Aluminum in contact with plain concrete can corrode, and the situation is worse if the concrete contains calcium chloride as an admixture or if the aluminum is in contact with dissimilar metal.

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Chapter 6
Bridge Materials

Topic 6.4 Fatigue and Fracture in Steel

6.4.1 Introduction

Since the 1960's, many steel bridges have developed fatigue induced cracks. Although these localized failures have been extensive, only a few U.S. bridges have actually collapsed as a result of steel fatigue fractures. These collapses have helped shape the National Bridge Inspection Program.

The first collapse was the Silver Bridge over the Ohio River at Point Pleasant, West Virginia on December 15, 1967. This structure was an eyebar chain suspension bridge with a 700-foot main span that collapsed without warning and forty-six people died (see Figure 6.4.1). The collapse was due to stress corrosion and corrosion fatigue that allowed a minute crack, formed during casting of an eye-bar, to grow. The two contributing factors, over the years continued to weaken the eye-bar. Stress corrosion cracking is the formation of brittle cracks in a normally sound material through the simultaneous action of a tensile stress and a corrosive environment. Corrosion fatigue occurs as a result of the combined action of a cyclic stress and a corrosive environment. The bridge's eye-bars were linked together in pairs like a chain. A huge pin passed through the eye and linked each piece to the next. The heat-treated carbon steel eye-bar broke, placing undue stress on the other members of the bridge. The remaining steel frame buckled and fell due to the newly concentrated stresses.

Figure 6.4.1 Silver Bridge Collapse

The second collapse occurred on June 28, 1983, when a suspended two-girder span carrying I-95 across the Mianus River in Greenwich, Connecticut failed (see Figure 6.4.2). The pin and hanger assembly failed at one location on one of the two girders. The forces were redistributed and caused an overstress that lead to the bridge collapse.

Figure 6.4.2 Mianus River Bridge Collapse

On August 1, 2007, the I-35W Mississippi River Bridge in Minneapolis, Minnesota collapsed. The cause of this deck truss collapse was due to a failed gusset plate (see Figure 6.4.3).

Figure 6.4.3 I-35W Mississippi River Bridge Collapse

The above catastrophes resulted due to a failure of a fracture critical member. For bridge inspectors, understanding the causes of the common member failure modes is important. This understanding permits the inspector to use more time evaluating problematic areas of a bridge and less time on other portions of the bridge.

When inspecting steel bridges, the inspector identifies a fracture critical member by sight or based on previous reports and drawings. The National Bridge Inspection Standards (NBIS) require that all fracture critical members on a bridge be identified, an inspection frequency be described and the inspection methods be listed prior to an inspection.

Fracture Critical Member

A fracture critical member (FCM) is a steel member in tension or with a tension element, whose failure probably causes a portion of or the entire bridge to collapse. Bridges that contain fracture critical members are considered fracture critical bridges.

Fatigue

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

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

Reviewing Member Forces

Two criterions exist for a bridge member to be classified as fracture critical. The first criterion deals with the forces in the member. Members that are in tension or members that have fibers or elements that are in tension meet the first criterion. The five types of member forces are presented in Topic 5.1.3 and include:

- Axial tension – Acts along the longitudinal axis of a member and tends to "pull" the member apart
- Axial compression – Acts along the longitudinal axis of a member and tends to "push" the member together
- Shear – Equal but opposite transverse forces which tend to slide one section of a member past an adjacent section producing diagonal tension force oriented 45 degrees to the longitudinal axis
- Bending moment – Develops when an external load applied transversely to a bridge member causes it to bend and produces both compression and tension forces at different locations in the member and can be positive or negative
- Torsion – A type of shear force resulting from externally applied moments that tend to twist or rotate the member about its longitudinal axis producing diagonal tension present on all surfaces of the member

Redundancy

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

Redundancy is defined as a structural condition where there are more elements of support than are necessary for stability.

Redundancy means that if a member or element fail, the load previously carried by the failed member is redistributed to other members or elements. These other members have the capacity to temporarily carry additional load, and collapse of the structure may be avoided. On structures without redundancy, the redistribution of load may cause additional members to also fail, resulting in a partial or total collapse of the structure.

There are three basic types of redundancy to consider in bridge design:

- Load path redundancy
- Structural redundancy
- Internal redundancy

Load Path Redundancy

Bridge designs that have three or more main load-carrying members or load paths between supports are considered load path redundant. If one member were to fail, the bridge load is redistributed to the other members, and bridge failure may not occur. An example of load path redundancy is a multi-girder bridge (see Figure 6.4.4).

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

Figure 6.4.4 Load Path Redundant Multi-Girder Bridge

Structural Redundancy

Bridge designs which provide continuity of load path from span to span are referred to as structurally redundant. Interior spans of a continuous span bridge designs are considered structurally redundant (see Figure 6.4.5). In the event of an interior member failure, loading from that span can be redistributed to the adjacent spans, and bridge failure may not occur.

Figure 6.4.5 Structurally Redundant Continuous Span Bridge

Continuous spans are structurally redundant except for the end spans, where the development of a fracture effectively causes two hinges, one at the abutment and one at the fracture itself. This situation leads to structural instability.

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

Internal Redundancy

Internal redundancy exists when a bridge member contains three or more elements that are mechanically fastened together so that multiple independent load paths are formed. Mechanical fasteners include rivets and bolts. Failure of one member element might not cause total failure of the member. Examples of internally redundant members are shown in Figures 6.4.6 and 6.4.7.

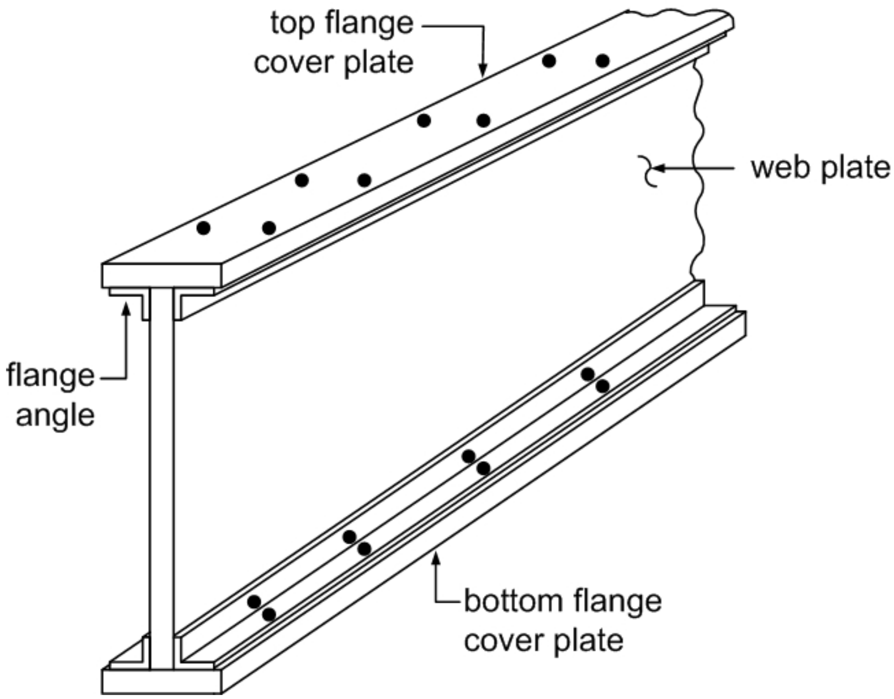


Figure 6.4.6 Internally Redundant Riveted I-Beam

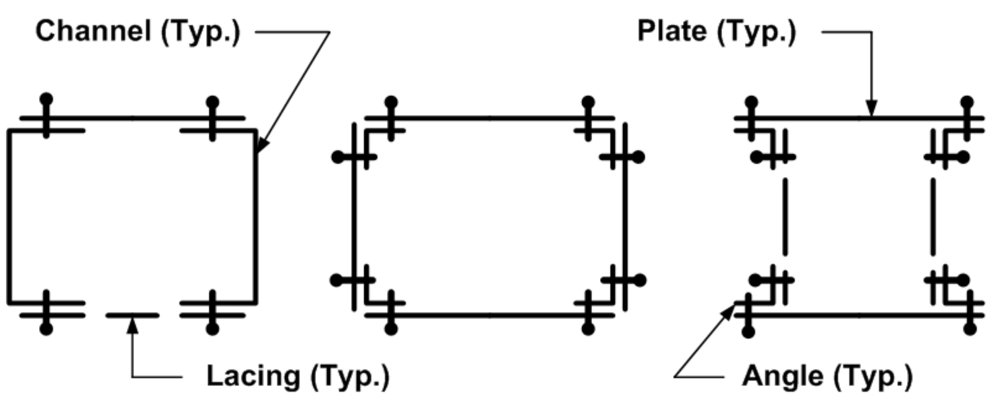


Figure 6.4.7 Internally Redundant Riveted Box Shapes

Internal redundancy of a member can be decreased or eliminated by repairs that involve welding. The welds provide paths for cracks to propagate from one element to another (see Figure 6.4.8).

Figure 6.4.8 Patch Plate Welded on Riveted Girder Web along Flange Angle

Non-redundant Configuration

Bridge inspectors are concerned primarily with load path redundancy. Neglect structural and internal redundancy and classify all bridges with less than three load paths as nonredundant (see Figure 6.4.9). Non load path redundant bridge configurations in tension contain fracture critical members.

AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, Section 10.3.1 states that main load-carrying components subjected to tensile stresses may be considered nonredundant load path members if failure of a single element could cause collapse.

AASHTO *LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Section 1.2, defines multiple load path structures as structures capable of supporting specified loads following loss of a main load-carrying component or connection. If a structure cannot support the specified loads following loss of a main load-carrying member, the consequence is "collapse" as defined in the *AASHTO LRFD Specifications.* Section 1.2 defines collapse as a major change in geometry of the bridge rendering it unfit for use.

AASHTO *LRFD Specifications*, Section 1.3.4, discusses redundancy. Main elements and components whose failure is expected to cause collapse of the bridge are designated as failure-critical and the associated structural system is considered nonredundant. Failure-critical members in tension may be designated as fracture-critical. Those elements and members whose failure is not expected to cause collapse of the bridge are nonfailure-critical and the associated structural system is considered redundant.

Figure 6.4.9 Non-redundant Two-Girder Bridge

6.4.2 Failure Mechanics

Failure mechanics involves describing the process by which a member fails when subjected to fatigue.

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

- Crack initiation
- Crack propagation
- Fracture

Crack Initiation

Cracks most commonly initiate from points of stress concentrations in structural or connection details. Stress concentrations can result from weld flaws, fatigue prone design and fabrication details, or out–of–plane distortions. The most critical conditions for crack initiation at structural details are those combining a flaw with a detail in a high stress concentration area.

Crack Propagation

Once a fatigue crack has initiated, applied cyclic stresses cause propagation, or growth, of a crack across the section of the member until it reaches a critical size.

Fracture

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

Bridge structures, particularly those that are welded, cannot be fabricated without some flaws and details with high stress concentrations.

Good detailing can reduce the number and severity of stress concentrations, but connecting the girders, stringers, floorbeams, diaphragms, and other members makes it impossible to avoid stress concentrations

Fatigue Life

The fatigue life of a member is the number of load cycles required to initiate and propagate a fatigue crack to critical size. The number of cycles used to determine fatigue life is based on truck traffic. Cars and buses do not create stresses large enough to contribute to fatigue life. Each load cycle or truck passage causes one or more major stress cycles. Wind and temperature changes may also cause stress cycles, but are not normally considered for fatigue life calculations for primary bridge members.

The number of cycles required to initiate a fatigue crack is the fatigue-crack-initiation life. The number of cycles required to propagate a fatigue crack to a critical size is called the fatigue-crack-propagation life. The total fatigue life is the sum of the initiation and propagation lives.

Bridge engineers use estimations of total fatigue life in predicting the fatigue crack potential of new and existing steel bridge members.

Types of Fractures

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

- Brittle Fracture **-** Occurs with no warning and without prior plastic deformation (see Figure 6.4.10). Once a brittle fracture occurs, the surface of the fracture is flat.
- Ductile Fracture **-** Generally proceeded by local plastic deformation of the net uncracked section. This plastic deformation results in distortion of the member, providing some visual warning of the impending failure which is the distorted shape which appears when the specimen stretches and necks down in diameter. Once a ductile fracture occurs, the surface of the fracture has shear lips at a 45 degree angle (see Figure 6.4.11).

Figure 6.4.10 Brittle Fracture of Cast Iron Specimen

Figure 6.4.11 Ductile Fracture of Cold Rolled Steel Specimen

Factors that Determine Fracture Behavior

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

Service temperature – Different steel types have different transition temperatures. Bridge members exposed to temperatures below their transition temperature, may experience a brittle fracture if they fail. For steels used in fracture critical members the transition temperature is defined as the minimum service temperature for which the Charpy V–notch test value is at least 25 ft–lbs.

Loading rate – Rapid loading of a steel member, as may occur from a truck collision or an explosion, can create sufficient energy to cause a member to fail in brittle fracture. Truck loading normally stresses the member at an intermediate loading rate which does not create a high energy level. Variations in the speed at which the truck crosses the bridge do not significantly alter the rate of loading.

Degree of constraint – Thick welded plates or complex joints can produce a high degree of constraint that limits the steel's ability to deform plastically. Thinner plates are less prone to fracture, given the same conditions, than are thicker plates.

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

- Cold service temperature
- Rapid truck loading rates
- High degree of constraint (stiff)

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

- Warm service temperature
- Slow truck loading rates

- Low degree of constraint (flexible)

The adverse combination of these three factors greatly enhances the likelihood of a brittle fracture. The transition from ductile behavior to brittle is a matter of degree. In either case, when it occurs, the failure of a fracture critical member is sudden and catastrophic.

Fracture Toughness

The fracture toughness is a quantitative method of expressing of a material's resistance to brittle fracture when a crack is present. Fracture toughness can be defined as the ability of a material to resist crack propagation while under load. Fracture toughness is dependent upon the chemical composition of the material. Steel has greater fracture toughness than iron. Fracture toughness generally depends on the steel type, temperature, together with geometric effects such as constraint. In general, thick welded members made of steel with low toughness are more likely to fracture in low temperatures.

An impact test that is used to determine the fracture toughness of a steel specimen or coupon is called the Charpy V-notch test (see Figure 6.4.12). This test measures the amount of energy absorbed by a test specimen prior to failure. The Charpy V-notch test requirements vary depending on the type of steel, type of construction, whether welded or mechanically fastened, and the applicable minimum service temperature.

Figure 6.4.12 Charpy V-notch Testing Machine

6.4.3

Factors Affecting Fatigue Crack Initiation

Most critical conditions for fatigue crack initiation are those which involve a combination of flaws and stress concentrations. Girders, stringers, floorbeams, diaphragms, bracing, truss members, hangers, and other members are structurally connected. Bridge structures, particularly those that are welded, cannot be fabricated without details that cause some level of stress concentrations. Good detailing can reduce the number and severity of these stress concentrations in connections.

Welds

Welds are the connections of metal parts formed by heating the surfaces to a plastic (or fluid) state and allowing the parts to flow together and join with or without the addition of filler material. The term base metal refers to the metal parts that are to be joined. Filler material, or weld material, is the additional metal generally used in the formation of welds. The complete assembly is referred to as a weldment. Conditions of stress concentration are often found in weldments and can be prone to crack initiation.

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

Groove Welds **–** Groove welds, which are sometimes referred to as butt welds, are used when the members to be connected are lined up edge to edge or are in the same plane (see Figure 6.4.13). Full penetration groove welds extend through the entire thickness of the piece being joined, while partial penetration groove welds do not. Weld reinforcement is the added filler material that causes the throat dimension to be greater than the thickness of the base metal. This weld reinforcement is sometimes ground flush with the base metal to qualify the joint for a better fatigue strength category (see Topic 6.4.5 for descriptions of AASHTO Fatigue Categories).

Fillet welds – Fillet welds connect members that either overlap each other or are joined edge to face of plate, as in plate girder assembly of web and flange plates (see Figure 6.4.14). Fillet welds are the most common type of weld because large tolerances in fabrication are allowable when members are lapped over each other instead of fitted together as in groove welds.

Plug welds **-** Plug welds pass through holes in one member to another, with weld metal filling the holes and joining the members together (see Figure 6.4.15). Plug welds have sometimes been used to fill misplaced holes. These repairs are very likely to contain flaws and microcracks that can result in the initiation of fatigue cracking. Plug welds are no longer permitted by AASHTO for bridge construction because they are fatigue prone due to the high degree of constraint and the prominence of weld flaws and slag inclusions. However, AASHTO does permit limited use of plug welds in bridge construction, such as web reinforcement plates (doubler plates) on girder webs at pin and hangers locations.

Tack welds **-** Tack welds are small welds commonly used to temporarily hold pieces in position during fabrication or construction (see Figure 6.4.16). They are often made carelessly, without proper procedures or preheating, and can be a problematic detail when located on a tension member.

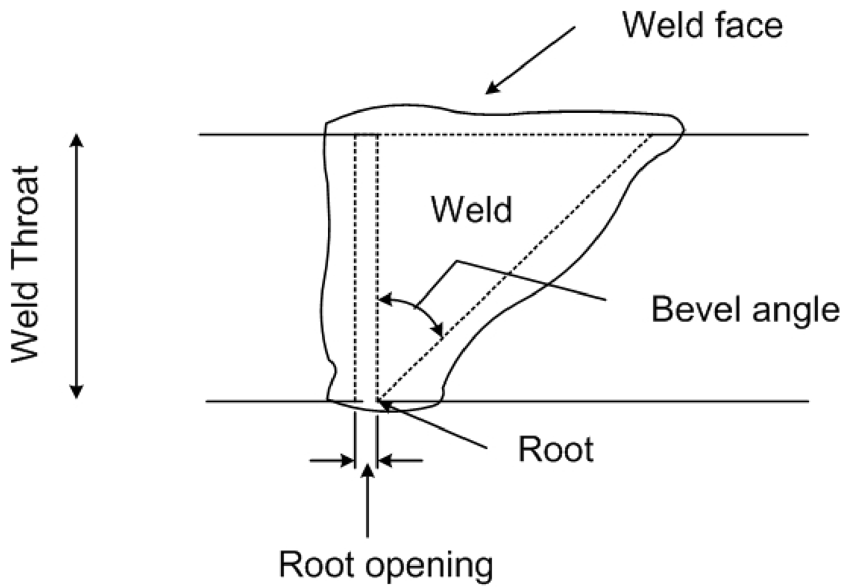


Figure 6.4.13 Groove Weld Nomenclature

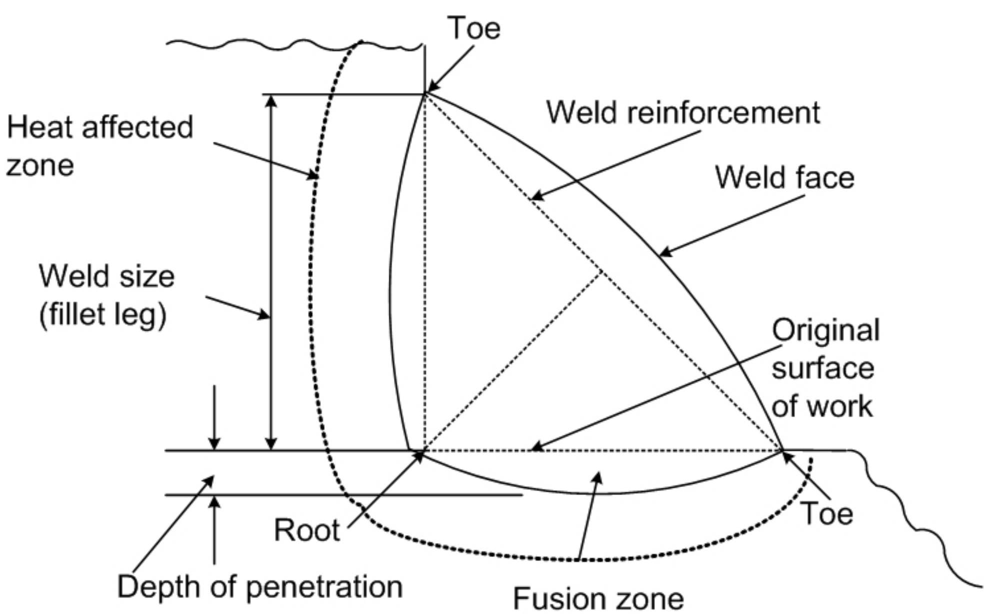


Figure 6.4.14 Fillet Weld Nomenclature

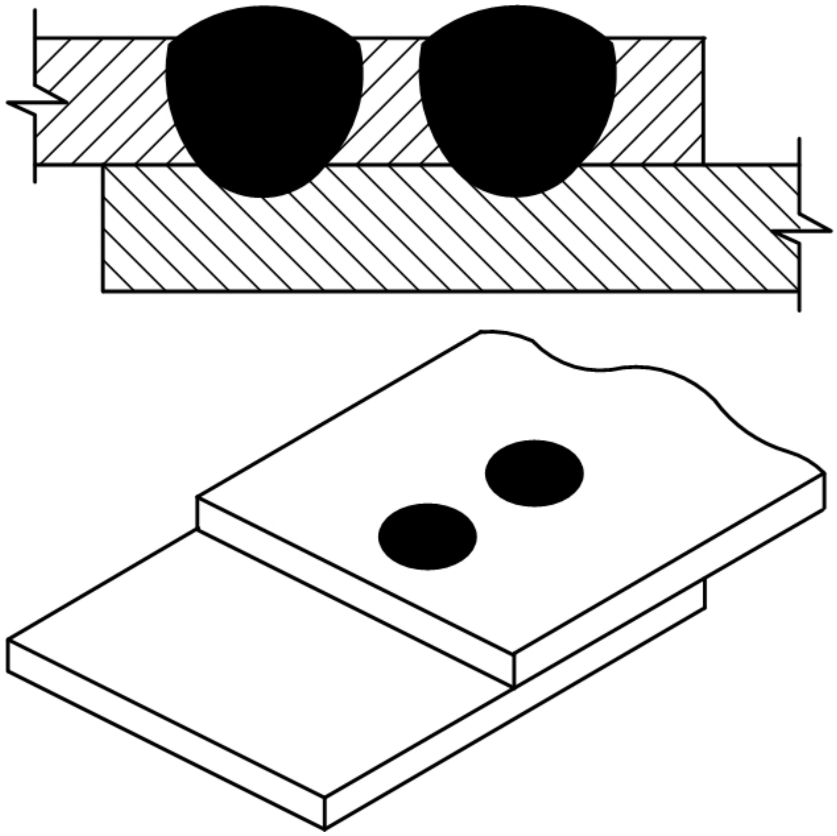


Figure 6.4.15 Plug Weld Schematic

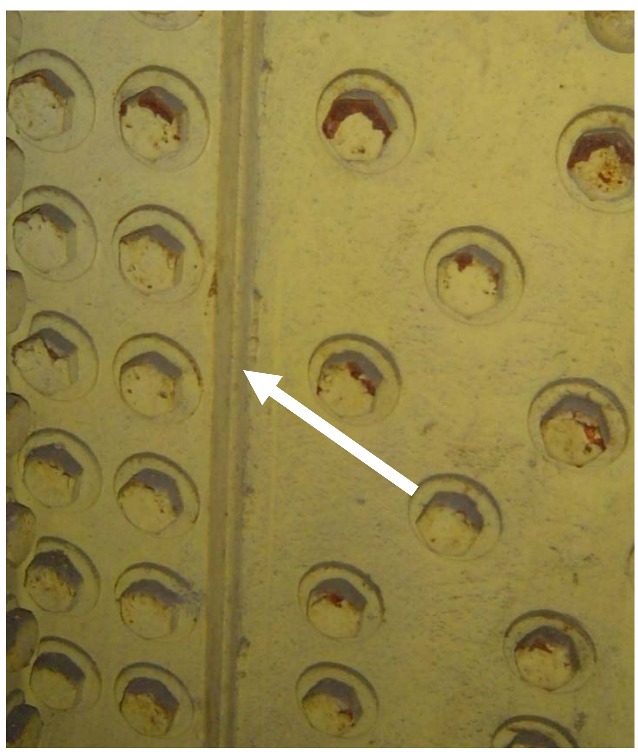


Figure 6.4.16 Tack Weld

Both plug and tack welds are smaller than fillet and groove welds but they can be the source of serious problems to bridges. They tend to be more problematic than groove and fillet welds. Cracks normally initiate at the weld toes or at any imperfections that may exist in the weld. Cracks from the tack welds often propagate into the base metal.

The joint geometry is also used to describe the weld. Some common weld joints include (see Figure 6.4.17):

- Butt
- Lap
- Tee
- Edge
- Corner

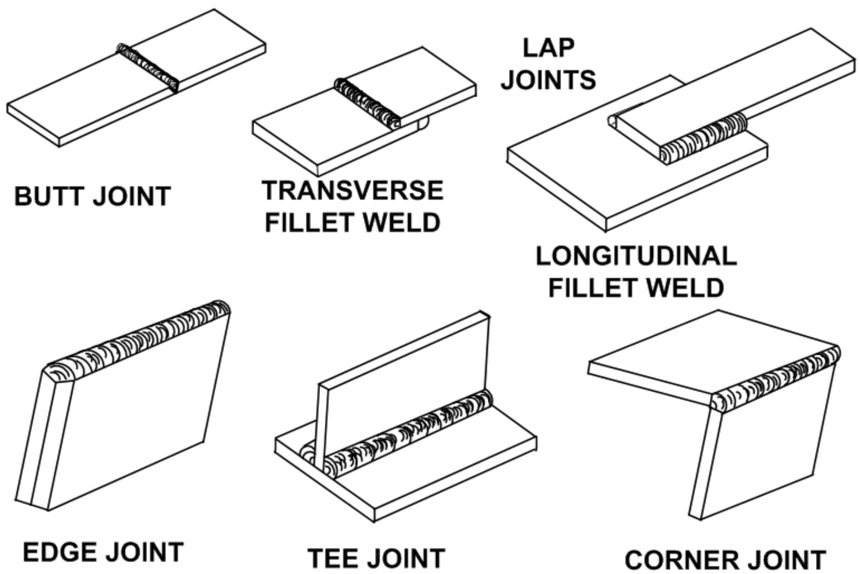


Figure 6.4.17 Types of Welded Joints

All welding processes result in high built-in residual tension stresses, which are at or near the yield point in the weldment and in the base metal adjacent to it. Load-induced stress concentrations also often occur at welded bridge connections, where these residual tensile stresses are high. This combination of stress concentration and high residual tensile stress is conducive to fatigue crack initiation. Such cracks typically begin either at the weld periphery, such as at the toe of a fillet weld, where there typically can be sharp discontinuities, or else at an internal discontinuity such as a slag inclusion or porosity (explained later). In the initial stages of fatigue crack growth, much of the fatigue life is expended by the time a crack has propagated out of the high residual tensile stress zone.

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

- Quality of weld and base material
- Fabrication methods

- Erection techniques
- In-service conditions

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

Material Deficiencies

Material deficiencies can occur when there is an incorrect proportion of steel assembled and rolled. This may cause the carbon content or the grain structure to be not conductive in producing today's ductile materials. Material deficiencies may exist in different forms:

- External flaws (e.g., surface laps)
- Internal flaws (e.g., nonmetallic inclusions, laminations and "rolled-in" plate deficiencies (see Figures 6.4.18 & 6.4.19)).

Figure 6.4.18 Exposed Lamination in Steel Slab

The centerline crack in Figure 6.4.18 may have resulted from a shrinkage cavity like that shown in Figure 6.4.19 which was not forged and melded completely in the hot rolling process.

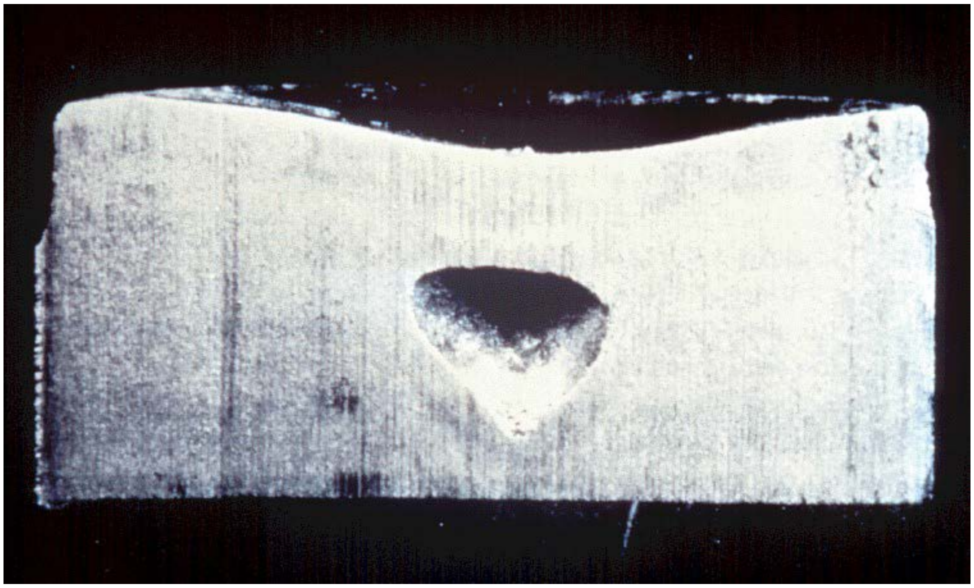


Figure 6.4.19 Shrinkage Cavity in Steel Billet

Fabrication Flaws

Fabrication flaws occur when members are joined together to produce elements designed to carry the primary stress. Welding deficiencies are potential discontinuities in welded members that could lead to a fatigue crack.

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

Incomplete Penetration – Incomplete penetration occurs when the weld metal fails to penetrate the root of a joint or fails to fuse completely with the root face of the base metal (see Figure 6.4.20). Incomplete penetration is not permitted for most bridge applications. Incomplete penetration welds cause a local stress riser at the root of a weld and can reduce the load-carrying capacity of the member. A stress riser is a detail that causes stress concentration.

Lack of fusion – Lack of fusion is a condition in which boundaries of unfused metal exist either between the base metal and weld metal or between adjacent layers of weld metal (see Figure 6.4.21). Lack of fusion is generally a result of poor welding techniques, can seriously reduce the load-carrying capacity of the member, and could be a point of crack initiation at a lower stress.

Slag inclusions – Slag inclusion occurs when nonmetallic matter is inadvertently trapped between the weld metal and the base metal (see Figure 6.4.22). Slag from the welding rod shield may be forced into the weld metal by the arc during the welding operation. If large, irregular inclusions or lengthy lines of inclusions are present, crack initiation at a lower stress could begin and the strength of the weld may be considerably reduced. However, small isolated globe shaped inclusions do not seriously affect the strength of a weld, but can be a point of crack initiation.

Porosity – Porosity is the presence of cavities in the weld metal caused by entrapped gas and takes the form of small spherical cavities, either scattered throughout the weld or clustered in local regions (see Figure 6.4.23). It is tolerated if the amount does not exceed specified quantities relative to weld size. Sometimes, porosity is visible on the surface of the weld.

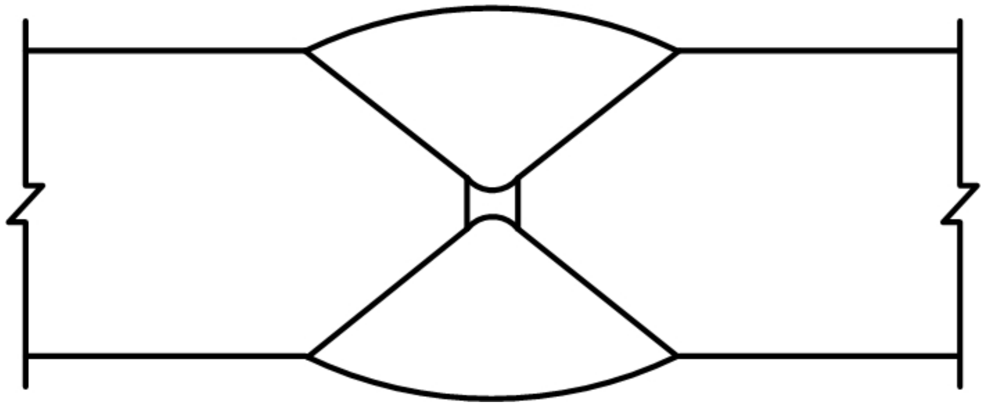


Figure 6.4.20 Incomplete Penetration of a Double V-Groove Weld

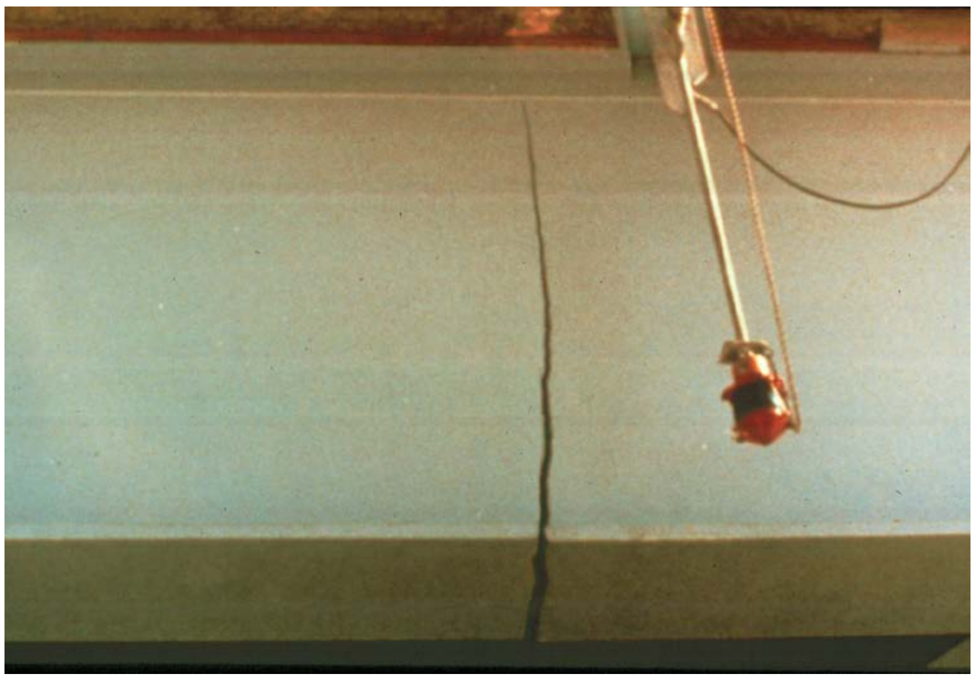


Figure 6.4.21 Crack Initiation from Lack of Fusion in Heat Affected Zone of Electroslag Groove Weld of a Butt Joint

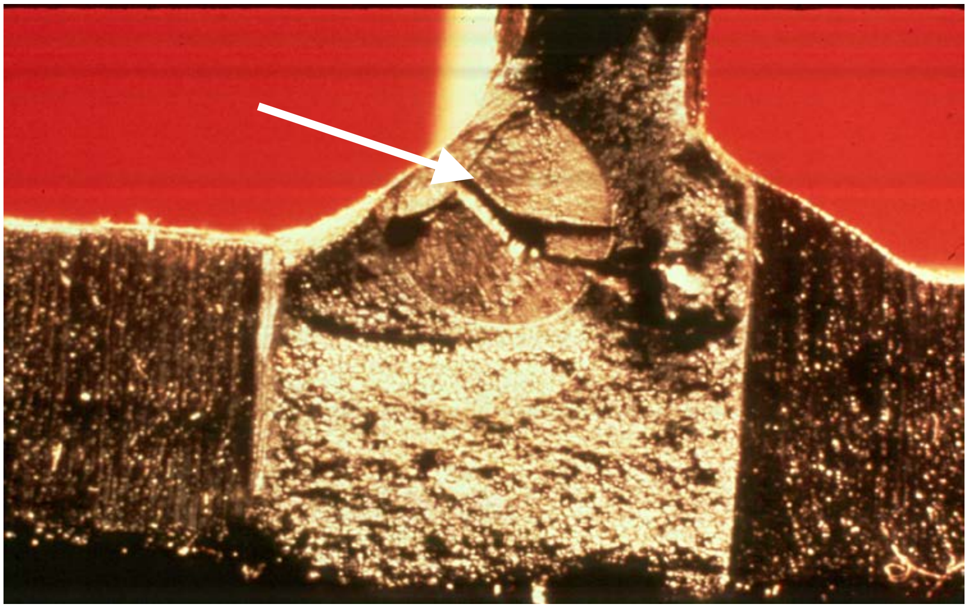


Figure 6.4.22 Web to Flange Crack due to Fillet Weld Slag Inclusion

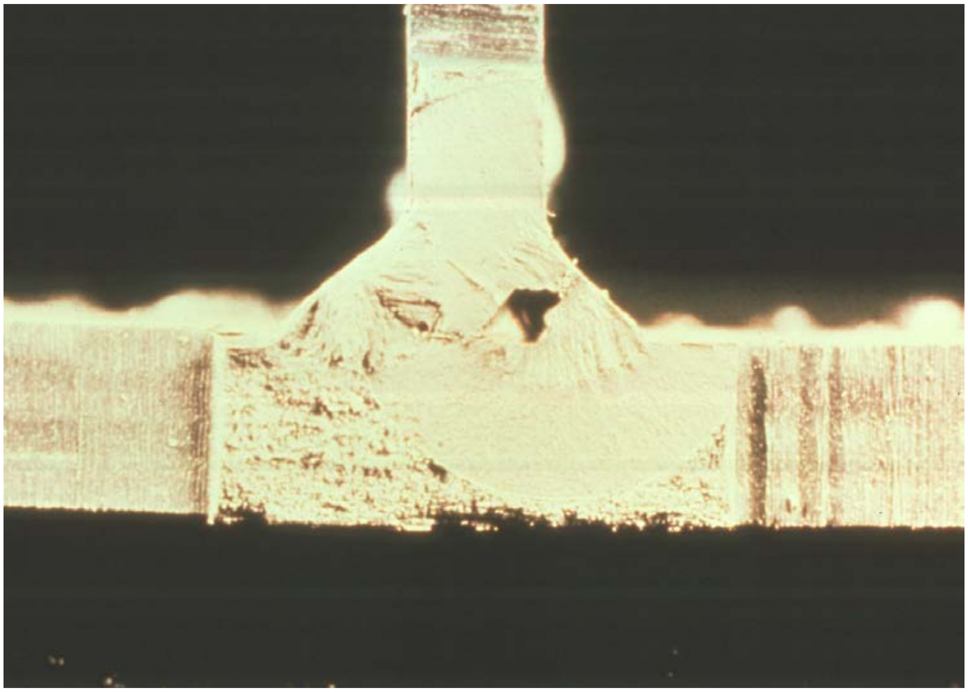


Figure 6.4.23 Crack Initiation from Porosity in Longitudinal Web-to-Flange Fillet Weld of Plate Girder

Plug welds are sometimes found in bridge members. In most cases, they were made to fill mislocated bolt holes. Such welds are highly restrained and often contain incomplete penetration, lack of fusion, slag inclusions, and porosity. There have been many instances where a crack and fracture have occurred because of a plug weld (see Figure 6.4.24).

Figure 6.4.24 Crack Resulting from Plug Welded Holes

Visible weld deficiencies include:

Improper welding practices

- Improper type and size of electrode - Electrodes are to suit the metal being joined, the welding position, the function of the weld, the plate thickness, and the size of the joint.
- Improper welding current and polarity - Welding current and polarity are to suit the type of electrode used and the joint to be made.
- Improper preheat and interpass temperature - Preheating and the required temperature level depends on the plate thickness, the grade of steel, the welding process, and ambient temperatures. Where these conditions dictate the need, make periodic checks to ensure adherence to requirements.

Undercut – The condition in which a local reduction in a section of base metal occurs alongside the weld deposit. This may happen either on the surface of the base metal at the toe of the weld, or in the fusion face of multiple pass welds due to overheating. This groove creates a mechanical notch, which is a stress riser (see Figure 6.4.25). When an undercut is controlled within the limits of specifications and does not constitute a sharp or deep notch, it is not seen as a serious deficiency.

Overlap – Overlap is a weld flaw at the toe of a weld in which the weld metal overflows onto the surface of the base metal without fusing to it due to insufficient heat (see Figure 6.4.26). This condition may exist intermittently or continuously along the weld joint. Discontinuity at the toe of a weld acts as a stress riser and reduces the fatigue strength of the member.

Cleanliness of the joint - Joint and plate surfaces are to be cleaned of dirt, rust, and moisture. This is especially important on those surfaces to be fused with the deposited weld metal. Mill scale during fabrication may interfere with surfaces fused together properly.

Incomplete penetration – When incomplete penetration occurs due to the failure of the weld material to fuse completely with the root face of the base material, a deficiency results.

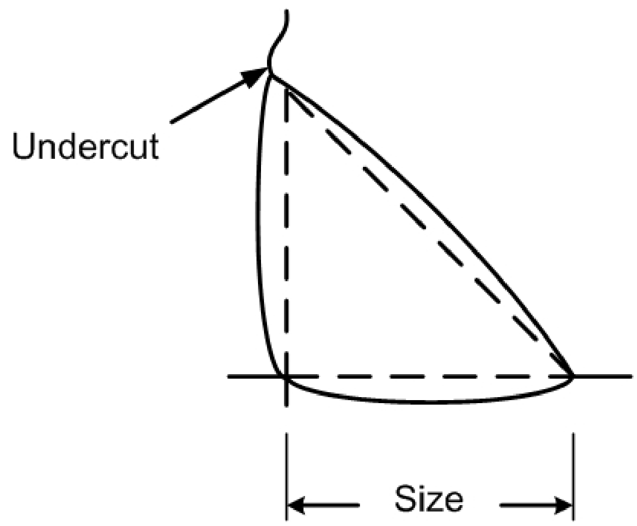


Figure 6.4.25 Undercut of a Fillet Weld

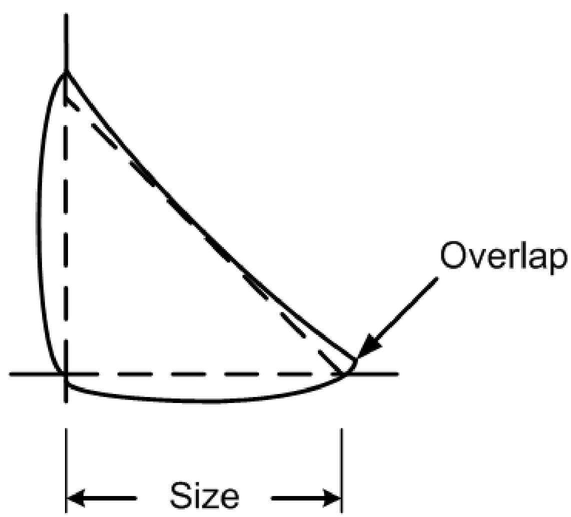


Figure 6.4.26 Overlap of a Fillet Weld

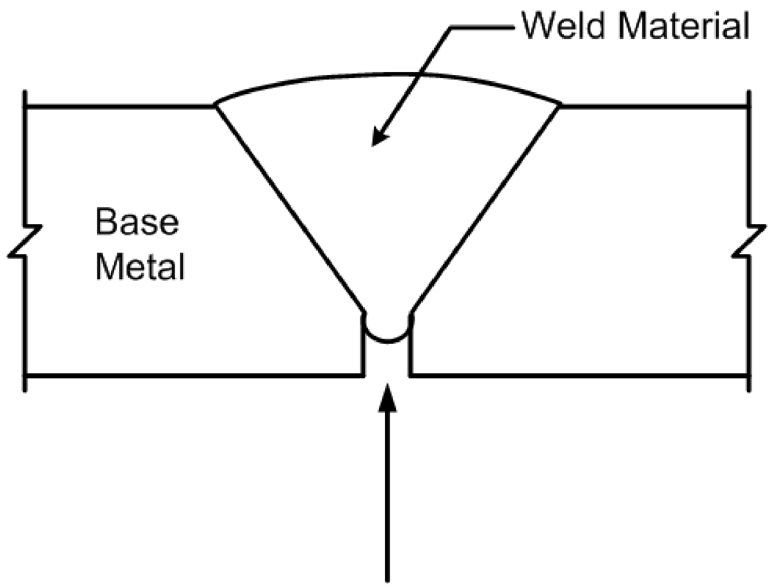


Figure 6.4.27 Incomplete Penetration of a V-Groove Weld

Other fabrication flaws include:

Craters – Craters are a depression at the termination of an arc weld. They may be a problem if they are undersized (i.e., not full throat) and/or they are concave, since they might crack upon cooling. Normally, on continuous fillet welds, there is no crater problem because each crater is filled by the next weld. The welder starts the arc at the outer end of the last crater and momentarily swings back into the crater to fill it before going ahead for the next weld.

Cracks – There are to be no cracks of any kind, either in the weld or in the heat-affected zone of the welded member.

Bolt and rivet holes – Holes of any kind in the base metal create a stress riser. Punched holes for rivets, without reaming, contain gouges that can initiate a crack. The stresses can increase when going around a hole. Burrs generated during the drilling process are additional risers and have to be removed.

Beam coping – When flange/web copings do not have the proper radius as per AASHTO specifications a stress riser is created (see Figure 6.4.28).

Flame cuts – Flame cutting, although fast, creates large surface discontinuities that are stress risers (see Figure 6.4.29). The surfaces of flame cut plates in tension are to be ground smooth in the direction of the tensile stress.

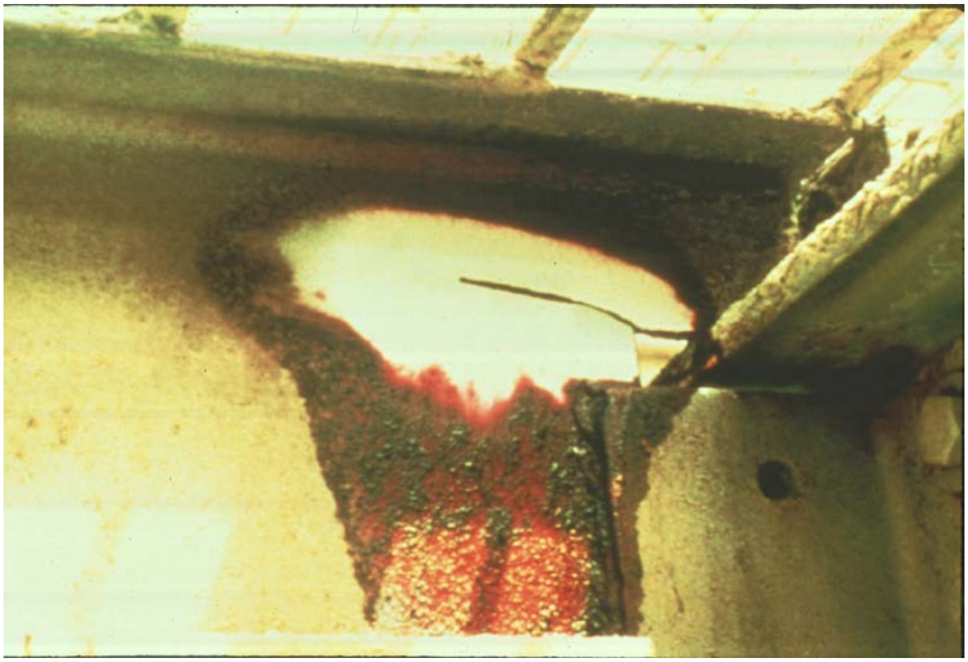


Figure 6.4.28 Crack Initiation at Coped Web in Stringer to Floorbeam

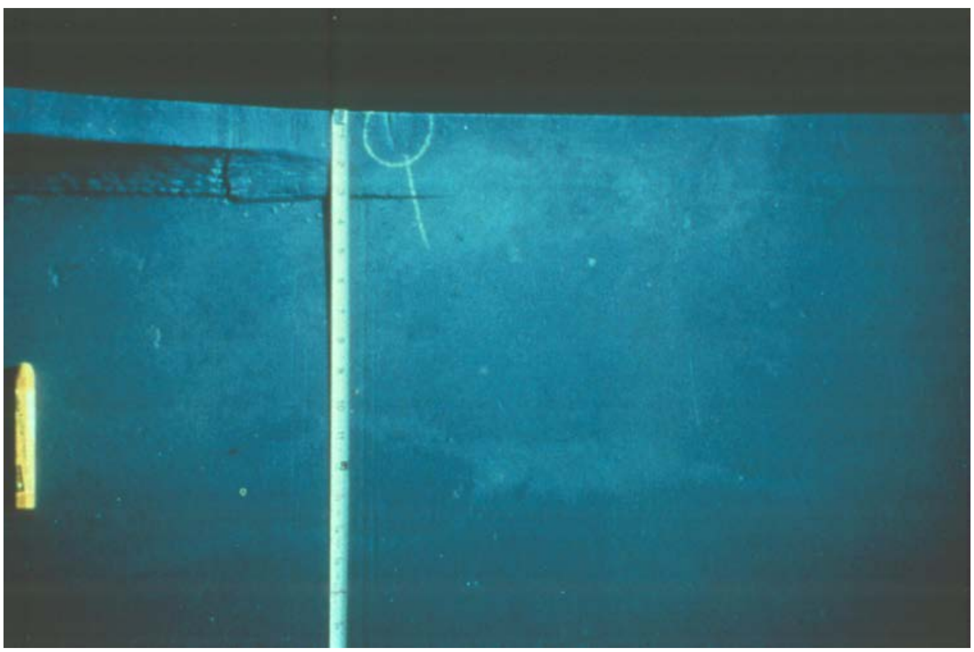


Figure 6.4.29 Insufficiently Ground Flame Cut of Gusset Plate for Arch to Tie Girder Connection

Lamellar tear - Applied tensile stress across the thickness of a plate due to weld quenching (cooling down) can induce internal lamellar tearing of the plate which is produced during fabrication (see Figure 6.4.30).

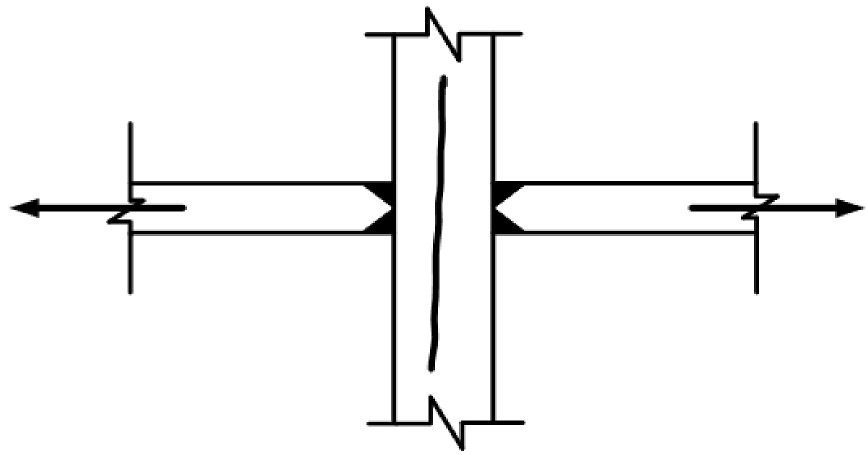


Figure 6.4.30 Thick plate with Two Plates Welded to it and Showing a Lamellar Tear

Transportation and Erection Flaws

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

Out-of-Plane Bending Forces – Sometimes during transport, beams are supported in a manner not accounted for in the original design. Horizontal and vertical deflections can cause out-of-plane bending about an unintended axis. Beams are to be securely blocked to resist cyclic side-sway movement during truck, barge or rail transport. There have been extreme cases where cracks have initiated in beams before they have been erected.

Nicks, Notches, and Indentations – Beam handling devices such as lifting tongs develop intense pressure at the point of contact and can cause measurable indentations and gouges. When transporting steel beams, chains are commonly used to secure the beam to the truck or railroad car which can create notches on the corners of steel members. These notches can lead to stress concentrations.

Tack Welds – Tack welding was a common practice in the mid 1950's and through the early 1960's and was applied to hold members together during erection. When left in place and exposed to tensile stress, they are a potential crack initiation location. Tack welds are to be avoided if possible. But if they are used, they are to be small and long so they won't interfere with subsequent submerged-arc welds and incorporated into the final weld.

In-Service Flaws

Once the structure is placed in service, environmental conditions, traffic and retrofits can contribute to fatigue crack initiation. The most common in–service flaws include:

Impact damage - Some members may be prone to collision damage by errant vehicles which may nick, tear, and excessively stress the steel (see Figure 6.4.31).

Indiscriminate welds - Indiscriminate application of welded attachments such as conduit supports, lighting attachments, and ladder brackets to steel members can cause stress risers in the base metal. Field conditions do not support high quality welds which can typically lead to weld flaws and lead to cracking.

Corrosion - Deep corrosion pits can develop in structures that are improperly detailed for corrosion control, poorly maintained, or left unpainted.

Improper heat straightening – When insufficient heat is applied during straightening (as in fixing collision damage), physical manipulation of the steel can induce plastic deformation which can strain harden the affected area.

Figure 6.4.31 Severe Collision Damage on a Fascia Girder

In summary, bridges can contain significant flaws or problematic details that can be the point of initiation of fatigue cracking and possibly result in fracture. Problematic details are identified prior to a fracture critical inspection.

6.4.4 Factors Affecting Fatigue Crack Propagation

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

- Stress range
- Number of cycles
- Type of details

Stress Range

The stress range is defined as the algebraic difference between the maximum stress and the minimum stress calculated at the detail under consideration. In other words, it is the value of the cyclic stress caused by a truck crossing the bridge (see Figure 6.4.32). The weight or dead load of the bridge produces a constant stress instead of a cyclic stress. Therefore, it does not affect the crack propagation life. Only stress ranges in tension or stress reversal can drive fatigue cracks to failure. Stress ranges in compression may cause cracks to grow to some extent at weldments where there are high residual tensile stresses. However, these "compression" cracks eventually arrest, and they do not induce fracture of the member. Only stress ranges in tension or stress reversal can drive fatigue cracks.

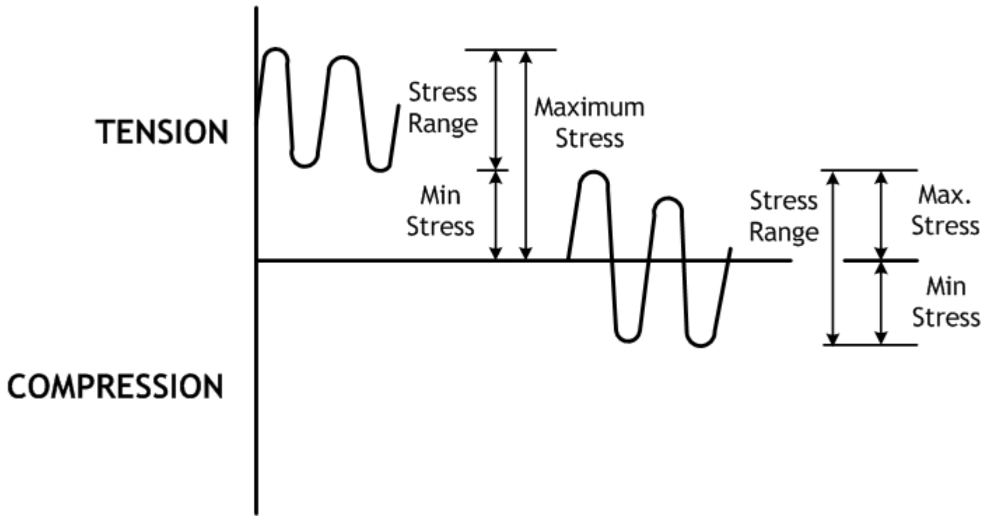


Figure 6.4.32 Applied Tensile and Compressive Stress Cycles

Number of Cycles

The number of stress cycles (frequency) is proportional to the number of trucks that cross the bridge during its service life. Each truck passage causes one or more major stress cycles. The number of cycles a bridge is subjected to is related to the age, location and span configuration of the structure. The number of cycles may eventually lead to fatigue cracks.

Types of Details

"Type of details" refers to the connection configuration in a particular area of the bridge. There are many problematic details used in the connections of bridges. AASHTO has chosen some typical details, or Illustrative Examples (see Figure 6.4.43). These Illustrative Examples are used to help determine AASHTO Fatigue Categories (see Topic 6.4.5).

Various details have different fatigue strengths associated with them. This is usually determined by the quality of the fabricated detail or the weld quality. It is common practice among bridge engineers to group steel bridge structural details into several AASHTO categories (A through E') of fatigue resistance. By doing this, the bridge engineer can design against risk levels of fatigue failure of the various details (i.e., details of higher fatigue strength categories are allowed higher stress ranges than the lower category details). In other words, details of higher fatigue strength categories (A & B), are allowed higher stress ranges than those in the lower category details (D through E').

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

Flange Crack Failure Process

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

One or more cracks can initiate from microscopic flaws or deficiencies at the weld toe of the transverse end weld (see Figure 6.4.33). Such cracking may then advance in three stages:

Figure 6.4.33 Part-Through Crack at a Cover Plated Flange

Stage 1

In the first stage, a part-through surface crack is only barely visible as a hairline on the bottom of the flange at the toe of weld. As stress is applied, the small cracks that have initiated join each other and begin to form a larger part-through surface crack (see Figure 6.4.34).

Figure 6.4.34 Part-Through Crack Growth at Cover Plate Welded to Flange

The crack front develops a thumbnail or half penny shape as it propagates in the thickness direction of the flange until reaching the inside surface. Once it breaks through the thickness of the flange, the shape rapidly changes into that of a three-ended crack.

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

Stage 2

During the second stage, the crack then propagates with two fronts moving across the flange width and one front moving into the web until it reaches a critical size, at which time the member may fracture (see Figure 6.4.35).

The crack is readily visible as a through-the-thickness crack on both the top and bottom surfaces of the flange (see Figure 6.4.36).

Approximately 5% of the fatigue life is left for growing the Stage 2 through crack (see Figure 6.4.37).

Stage 3

When a crack propagates to a critical size, the member fractures. Fracture is the separation of the member into two parts. When the member is fracture critical, the span, or a portion of it, likely collapses.

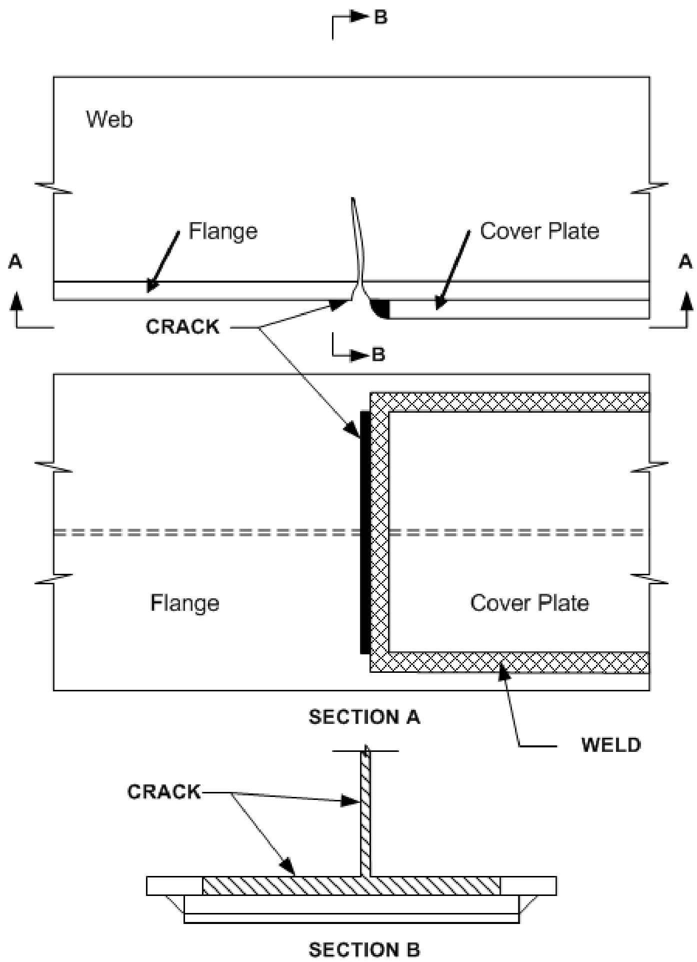


Figure 6.4.35 Through Crack Growth at Cover Plate Welded to Flange

Figure 6.4.36 Through Crack at a Cover Plated Flange

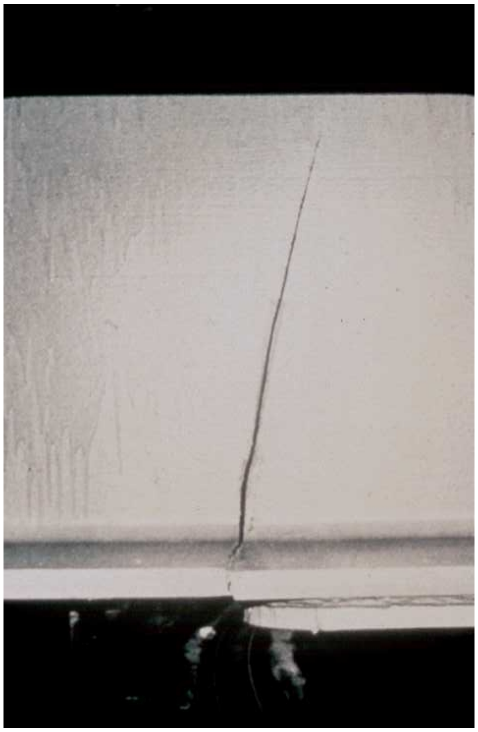


Figure 6.4.37 Through Crack has Propagated into the Web

Inspection

It is important the inspector realizes that cracks are only readily detectable visually as a through crack after most of the fatigue life of the detail is gone. Therefore, notify the bridge owner immediately whenever cracks are found in a flange.

When the fatigue life is finally used up, that is the fatigue crack has grown to a critical size and stress intensity, the fracture then occurs. The brittle fracture surface appears crystalline or uneven, and often reveals a herringbone pattern oriented toward the point of fracture initiation (see Figure 6.4.38).

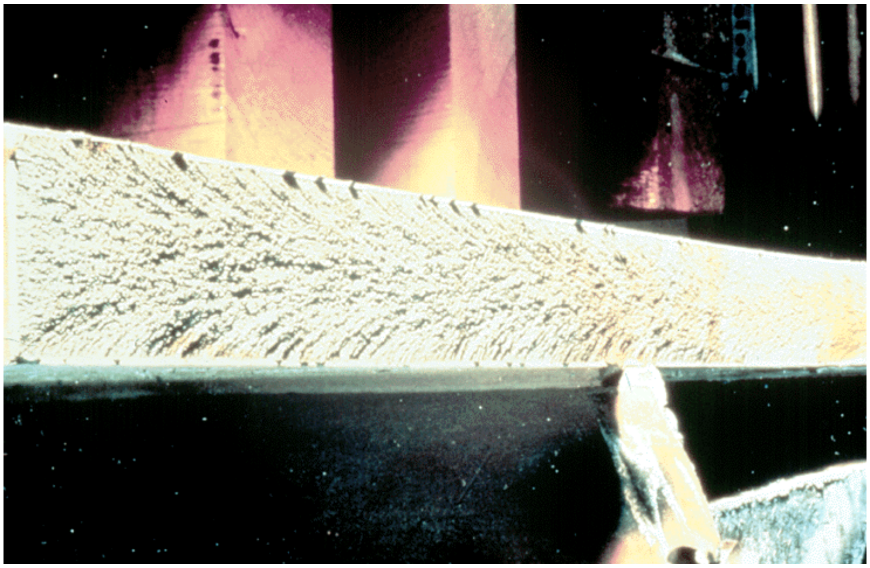


Figure 6.4.38 Brittle Fracture – Herringbone Pattern

Web Crack Failure Process

A common location for initiation of a web crack is at the weld toe of a transverse stiffener that is welded to the web of a beam. This type of crack grows in three stages (see Figure 6.4.39):

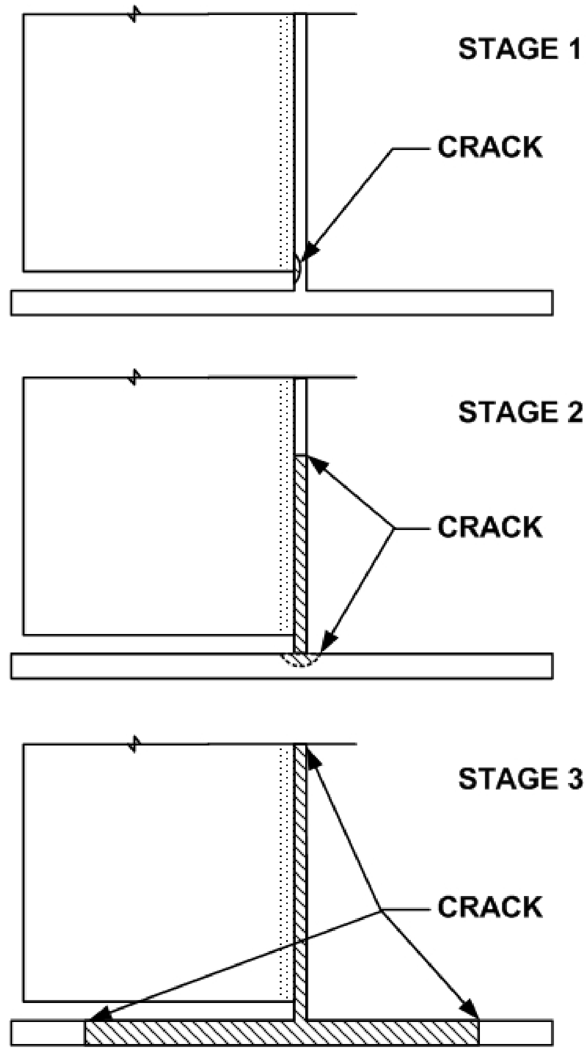


Figure 6.4.39 Crack Growth at Transverse Stiffener Welded to Web

Stage 1

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

A part-through stiffener crack is often just barely visible as a hairline along the toe of the weld (see Figure 6.4.40).

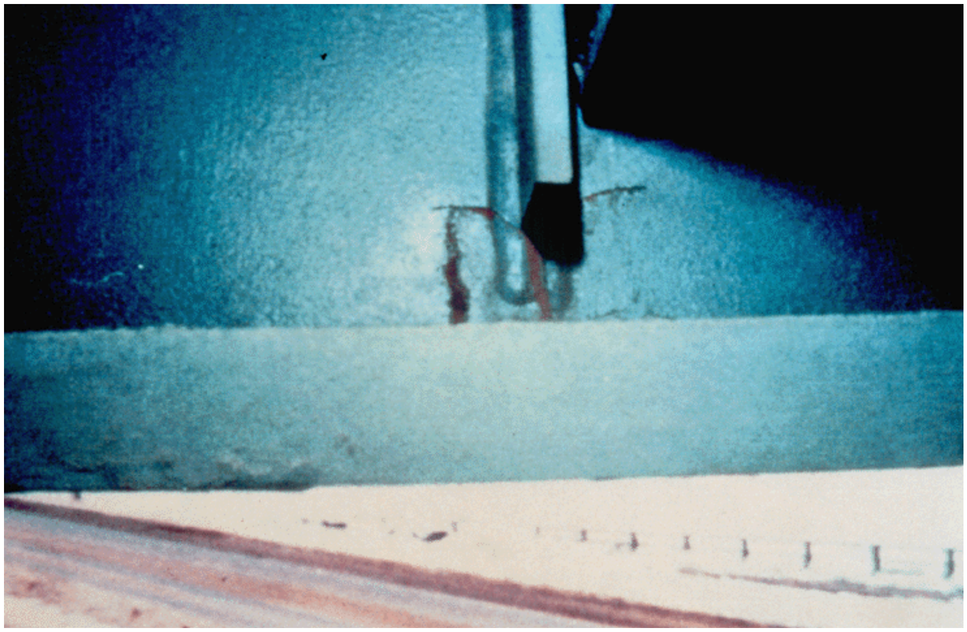


Figure 6.4.40 Part-Through Web Crack

This type of stiffener crack expends about 95% of its fatigue life propagating in Stage 1.

Stage 2

After it breaks through the web, the shape changes into a two-ended (or more) through crack which propagates up and down the web (see Figure 6.4.41). The through crack can be readily seen on both sides of the web. The stiffener crack expends about 5% of the fatigue life propagating in Stage 2.

Figure 6.4.41 Through Crack in the Web

Stage 3

Eventually, the lower crack front reaches the bottom flange, and the three-ended crack then propagates with two fronts moving across the flange and one front moving farther up the web, until the member fractures (see Figure 6.4.42).

Figure 6.4.42 Through-Crack Ready to Propagate into the Flange

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

Bring any web cracks discovered to the immediate attention of a bridge owner.

6.4.5 AASHTO Detail Categories for Load-Induced Fatigue

For purposes of designing bridges for fatigue caused by in-plane bending stress, the details are grouped into categories labeled A to E′. These categories are presented in the AASHTO *LRFD Bridge Design Specifications* Table 6.6.1.2.3-1 - Details for Load-Induced Fatigue (see Figure 6.4.43). For existing bridges these categories provide a method for the inspector to classify fatigue prone details. AASHTO Fatigue Categories are based on the load induced fatigue. Load-Induced fatigue is due to "in-plane" bending. In-plane bending occurs parallel to the longitudinal axis. The classification of details by category does not apply to details that crack due to out-of-plane distortion.

Each letter represents a rating given to a detail which indicates its level of fatigue strength, level "A" offering the highest and level " E′ " having the lowest resistance. Note that the 1998 AASHTO *Bridge Design Specifications* (2nd edition) have eliminated Category F. Category E can be conservatively applied in place of Category F. The details assigned to the same category have about equally severe stress concentrations and comparable fatigue lives. The alphabetical classification by the severity of the stress concentration is a useful method of identifying fatigue strength.

When used in fracture critical inspections, these fatigue categories serve as a reminder of which details are prone to fatigue cracking. They also prioritize the level of effort expanded to inspect each detail. The categories are defined as follows.

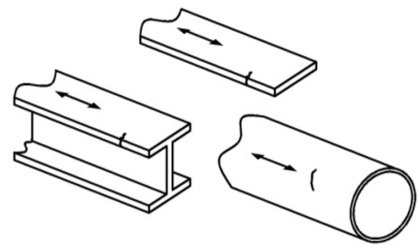
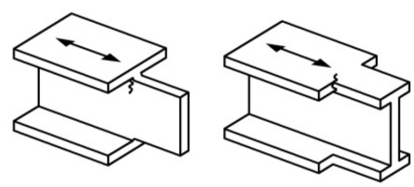
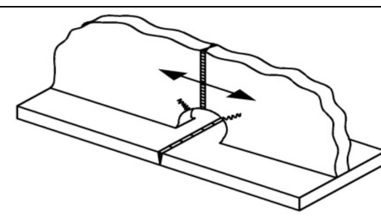
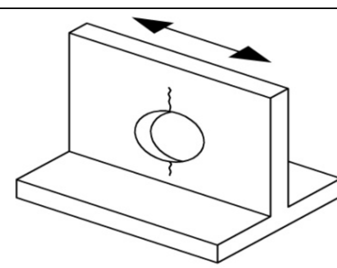
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 1—Plain Material away from Any Welding					
1.1 Base metal, except noncoated weathering steel, with rolled or cleaned surfaces. Flame-cut edges with surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	A	250×10^8	24	Away from all welds or structural connections	
1.2 Noncoated weathering steel base metal with rolled or cleaned surfaces designed and detailed in accordance with FHWA (*1989*). Flame-cut edges with surface roughness value of 1,000 μ-in. or less, but without re-entrant corners.	B	120×10^8	16	Away from all welds or structural connections	
1.3 Member with re-entrant corners at copes, cuts, block-outs or other geometrical discontinuities made to the requirements of AASHTO/AWS D1.5, except weld access holes.	C	44×10^8	10	At any external edge	
1.4 Rolled cross sections with weld access holes made to the requirements of AASHTO/AWS D1.5, Article 3.2.4.	C	44×10^8	10	In the base metal at the re-entrant corner of the weld access hole	
1.5 Open holes in members (*Brown et al. 2007*).	D	22×10^8	7	In the net section originating at the side of the hole	

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue
Source: American Association of State Highway and Transportation Officials

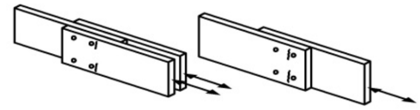
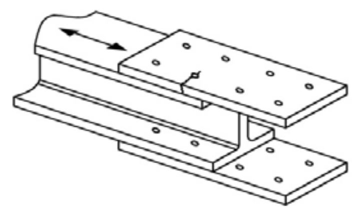
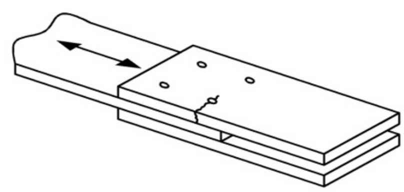
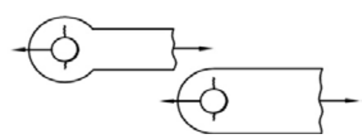
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 2—Connected Material in Mechanically Fastened Joints					
2.1 Base metal at the gross section of high-strength bolted joints designed as slip-critical connections with pre-tensioned high-strength bolts installed in holes drilled full size or subpunched and reamed to size – e.g. bolted flange and web splices and bolted stiffeners. (Note: see Condition 2.3 for bolt holes punched full size.)	B	120×10^8	16	Through the gross section near the hole	
2.2 Base metal at the net section of high-strength bolted joints designed as bearing-type connections, but fabricated and installed to all requirements for slip-critical connections with pre-tensioned high strength bolts installed in holes drilled full size or subpunched and reamed to size. (Note: see Condition 2.3 for bolt holes punched full size.)	B	120×10^8	16	In the net section originating at the side of the hole	
2.3 Base metal at the net section of all bolted connections in hot dipped galvanized members (*Huhn and Valtinat 2004*); base metal at the appropriate section defined in Condition 2.1 or 2.2, as applicable, of high strength bolted joints with pretensioned bolts installed in holes punched full size (*Brown et al. 2007*), and base metal at the net section of other mechanically fastened joints, except for eyebars and pin plates; e.g., joints using A307 bolts or non pretensioned high strength bolts.	D	22×10^8	7	In the net section originating at the side of the hole or through the gross section near the hole, as applicable	
2.4 Base metal at the net section of eyebar heads or pin plates (Note: for base metal in the shank of eyebars or through the gross section of pin plates, see Condition 1.1 or 1.2, as applicable).	E	11×10^8	4.5	In the net section originating at the side of the hole	

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

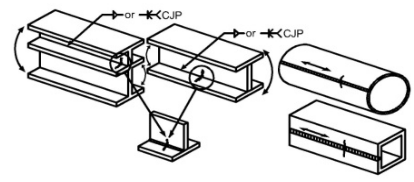
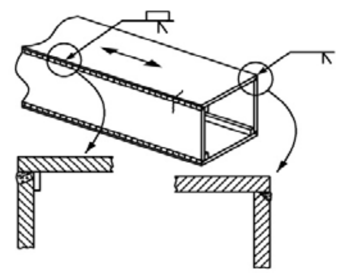
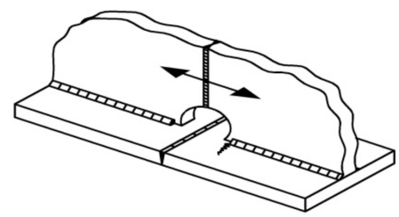
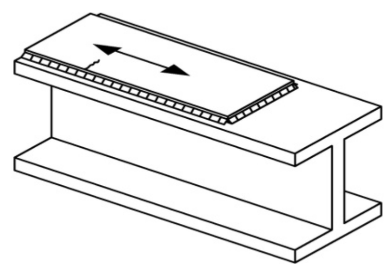
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 3—Welded Joints Joining Components of Built-Up Members					
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete joint penetration groove welds back-gouged and welded from the second side, or by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From surface or internal discontinuities in the weld away from the end of the weld	
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete joint penetration groove welds with backing bars not removed, or by continuous partial joint penetration groove welds parallel to the direction of applied stress.	B′	61×10^8	12	From surface or internal discontinuities in the weld, including weld attaching backing bars	
3.3 Base metal and weld metal at the termination of longitudinal welds at weld access holes made to the requirements of AASHTO/AWS D1.5, Article 3.2.4 in built-up members. (Note: does not include the flange butt splice).	D	22×10^8	7	From the weld termination into the web or flange	
3.4 Base metal and weld metal in partial length welded cover plates connected by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From surface or internal discontinuities in the weld away from the end of the weld	

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

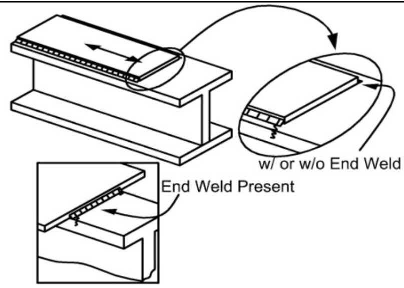
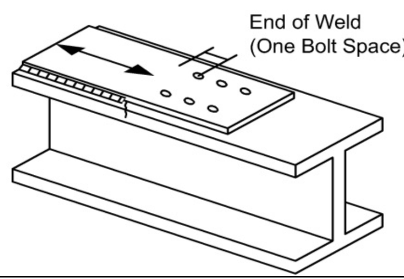
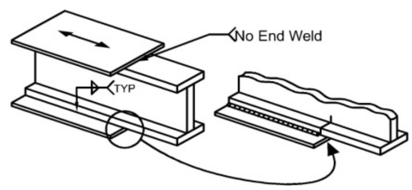
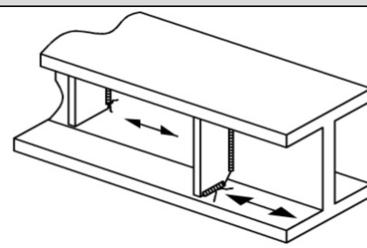
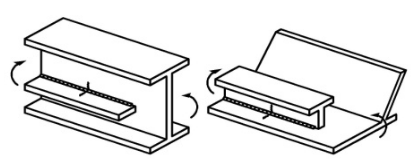
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
3.5 Base metal at the termination of partial length welded cover plates having square or tapered ends that are narrower than the flange, with or without welds across the ends, or cover plates that are wider than the flange with welds across the ends:				In the flange at the toe of the end weld or in the flange at the termination of the longitudinal weld or in the edge of the flange with wide cover plates	
Flange thickness ≤ 0.8 in.	E	11×10^8	4.5		
Flange thickness > 0.8 in.	E′	3.9×10^8	2.6		
3.6 Base metal at the termination of partial length welded cover plates with slip-critical bolted end connections satisfying the requirements of Article 6.10.12.2.3.	B	120×10^8	16	In the flange at the termination of the longitudinal weld	
3.7 Base metal at the termination of partial length welded cover plates that are wider than the flange and without welds across the ends.	E′	3.9×10^8	2.6	In the edge of the flange at the end of the cover plate weld	
Section 4—Welded Stiffener Connections					
4.1 Base metal at the toe of transverse stiffener-to-flange fillet welds and transverse stiffener-to-web fillet welds. (Note: includes similar welds on bearing stiffeners and connection plates).	C′	44×10^8	12	Initiating from the geometrical discontinuity at the toe of the fillet weld extending into the base metal	
4.2 Base metal and weld metal in longitudinal web or longitudinal box-flange stiffeners connected by continuous fillet welds parallel to the direction of applied stress.	B	120×10^8	16	From the surface or internal discontinuities in the weld away from the end of the weld	

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

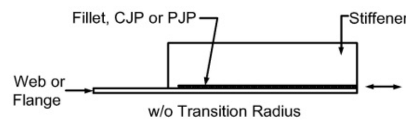
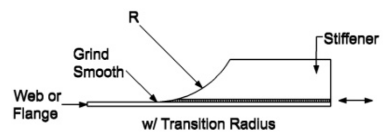
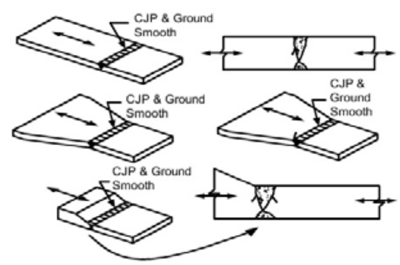
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
4.3 Base metal at the termination of longitudinal stiffener-to-web or longitudinal stiffener-to-box flange welds:					
With the stiffener attached by fillet welds and with no transition radius provided at the termination:				In the primary member at the end of the weld at the weld toe	
Stiffener thickness < 1.0 in.	E	11×10^8	4.5		
Stiffener thickness ≥ 1.0 in.	E′	3.9×10^8	2.6		
With the stiffener attached by welds and with a transition radius *R* provided at the termination with the weld termination ground smooth:					
$R \geq 24$ in	B	120×10^8	16	In the primary member near the point of tangency of the radius	
24 in. > $R \geq$ 6 in.	C	44×10^8	10		
6 in. > $R \geq$ 2 in.	D	22×10^8	7		
2 in. > R	E	11×10^8	4.5		
Section 5—Welded Joints Transverse to the Direction of Primary Stress					
5.1 Base metal and weld metal in or adjacent to complete joint penetration groove welded butt splices, with weld soundness established by NDT and with welds ground smooth and flush parallel to the direction of stress. Transitions in thickness or width shall be made on a slope no greater than 1:2.5 (see also Figure 6.13.6.2-1).				From internal discontinuities in the filler metal or along the fusion boundary or at the start of the transition	
F_y < 100 ksi	B	120×10^8	16		
F_y ≥ 100 ksi	B′	61×10^8	12		

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

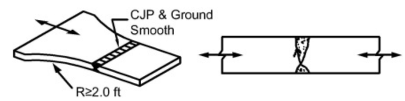
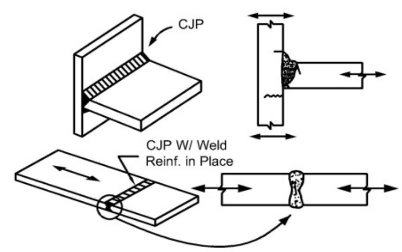
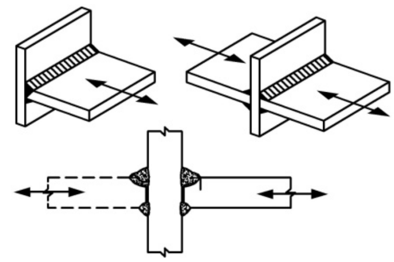
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
5.2 Base metal and weld metal in or adjacent to complete joint penetration groove welded butt splices, with weld soundness established by NDT and with welds ground parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft with the point of tangency at the end of the groove weld (see also Figure 6.13.6.2-1).	B	120×10^8	16	From internal discontinuities in the filler metal or discontinuities along the fusion boundary	
5.3 Base metal and weld metal in or adjacent to the toe of complete joint penetration groove welded T or corner joints, or in complete joint penetration groove welded butt splices, with or without transitions in thickness having slopes no greater than 1:2.5 when weld reinforcement is not removed. (Note: cracking in the flange of the 'T' may occur due to out-of-plane bending stresses induced by the stem).	C	44×10^8	10	From the surface discontinuity at the toe of the weld extending into the base metal or along the fusion boundary	
5.4 Base metal and weld metal at details where loaded discontinuous plate elements are connected with a pair of fillet welds or partial joint penetration groove welds on opposite sides of the plate normal to the direction of primary stress.	C as adjusted in Eq. 6.6.1.2.5-4	44×10^8	10	Initiating from the geometrical discontinuity at the toe of the weld extending into the base metal or, initiating at the weld root subject to tension extending up and then out through the weld	

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

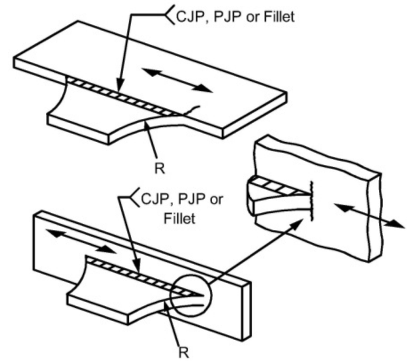
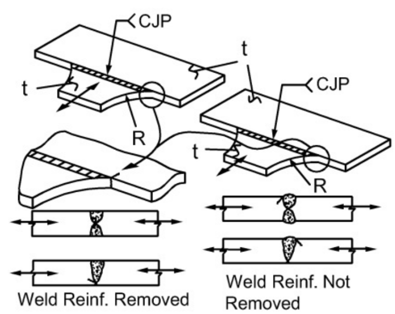
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 6—Transversely Loaded Welded Attachments					
6.1 Base metal in a longitudinally loaded component at a transversely loaded detail (e.g. a lateral connection plate) attached by a weld parallel to the direction of primary stress and incorporating a transition radius *R* with the weld termination ground smooth.				Near point of tangency of the radius at the edge of the longitudinally loaded component	
$R \geq 24$ in.	B	120×10^8	16		
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
(Note: Condition 6.2, 6.3 or 6.4, as applicable, shall also be checked.)					
6.2 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component of equal thickness by a complete joint penetration groove weld parallel to the direction of primary stress and incorporating a transition radius *R*, with weld soundness established by NDT and with the weld termination ground smooth:				Near points of tangency of the radius or in the weld or at the fusion boundary of the longitudinally loaded component or the transversely loaded attachment	
With the weld reinforcement removed:					
$R \geq 24$ in.	B	120×10^8	16		
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
With the weld reinforcement not removed:				At the toe of the weld either along the edge of the longitudinally loaded component or the transversely loaded attachment	
$R \geq 24$ in.	C	44×10^8	10		
24 in. $> R \geq 6$ in.	C	44×10^8	10		
6 in. $> R \geq 2$ in.	D	22×10^8	7		
2 in. $> R$	E	11×10^8	4.5		
(Note: Condition 6.1 shall also be checked.)					

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

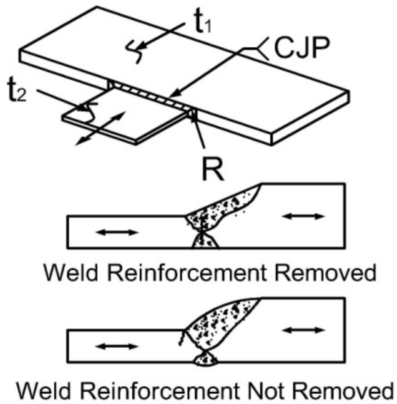
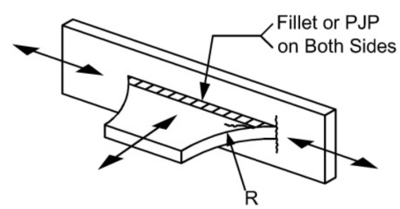
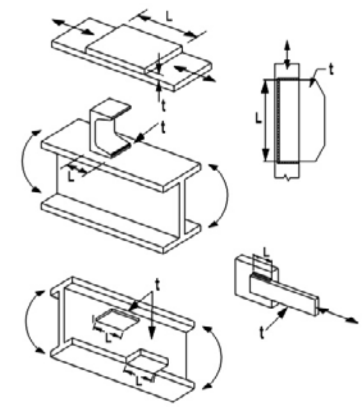
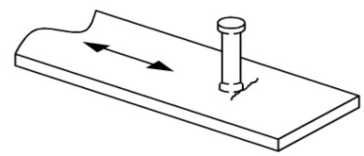
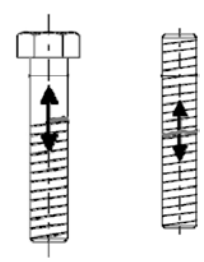
Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
6.3 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component of unequal thickness by a complete joint penetration groove weld parallel to the direction of primary stress and incorporating a weld transition radius R, with weld soundness established by NDT and with the weld termination ground smooth: With the weld reinforcement removed:				At the toe of the weld along the edge of the thinner plate In the weld termination of small radius weld transitions At the toe of the weld along the edge of the thinner plate	
$R \geq 2$ in.	D	22×10^8	7		
$R < 2$ in.	E	11×10^8	4.5		
For any weld transition radius with the weld reinforcement not removed:	E	11×10^8	4.5		
(Note: Condition 6.1 shall also be checked.)					
6.4 Base metal in a transversely loaded detail (e.g. a lateral connection plate) attached to a longitudinally loaded component by a fillet weld or a partial joint penetration groove weld, with the weld parallel to the direction of primary stress (Note: Condition 6.1 shall also be checked.)	See Condition 5.4				

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

Description	Category	Constant A (ksi^3)	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 7—Longitudinally Loaded Welded Attachments					
7.1 Base metal in a longitudinally loaded component at a detail with a length *L* in the direction of the primary stress and a thickness *t* attached by groove or fillet welds parallel or transverse to the direction of primary stress where the detail incorporates no transition radius:				In the primary member at the end of the weld at the weld toe	
$L < 2$ in.	C	44×10^8	10		
2 in. $\leq L \leq 12t$ or 4 in.	D	22×10^8	7		
$L > 12t$ or 4 in.					
$t < 1.0$ in	E	11×10^8	4.5		
$t \geq 1.0$ in.	E′	3.9×10^8	2.6		
Section 8—Miscellaneous					
8.1 Base metal at stud-type shear connectors attached by fillet or automatic stud welding	C	44×10^8	10	At the toe of the weld in the base metal	
8.2 Nonpretensioned high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Use the stress range acting on the tensile stress area due to live load plus prying action when applicable.				At the root of the threads extending into the tensile stress area	
(Fatigue II) Finite Life	E′	3.9×10^8	N/A		
(Fatigue I) Infinite Life	D	N/A	7		

Footnote:
TH = Threshold

Figure 6.4.43 *AASHTO LRFD Bridge Design Specifications*, 5th Edition, with 2010 Interim Revisions, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue, continued
Source: American Association of State Highway and Transportation Officials

6.4.6 Fracture Critical Bridge Types

The following is a list of steel bridge superstructure members and connections which are susceptible to fatigue cracking and possible failure:

- Two-girders systems (see Topic 10.2)
- Box beams and girders (see Topic 10.3)
- Trusses (see Topic 10.4)
- Arches (see Topic 10.5)
- Rigid frames (see Topic 10.6)
- Pin and hanger assemblies (see Topic 10.3)
- Gusset plates (see Topic 10.8)
- Eyebars (see Topic 10.9)
- Cable supported bridges (see Topic 16.1)
- Movable bridges (see Topic 16.2)
- Floating bridges (see Topic 16.3)

Fatigue cracks can develop in steel bridges as a result of repeated loading. Generally, the stress range, number of cycles and type of detail are the most important factors that influence fatigue cracking. Recognizing and understanding the behavior of connections and details is crucial if the inspector is to properly inspect FCMs. Connections and details are often the locations of highest stress concentrations.

6.4.7 Fracture Criticality

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

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

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

- The predicted number and size of loads

Details and Deficiencies

Carefully inspect more susceptible low fatigue strength details, such as AASHTO Fatigue Category D through E′ details on fracture critical bridges (see Figure 6.4.44). Be aware of bridges that may have experienced large truck volumes and/or magnitude of stress cycles.

Figure 6.4.44 Riveted Gusset Plate Connection

Initial Deficiencies

Initial deficiencies, in many cases, are cracks resulting from poor quality welds between attachments and base metal (see Figure 6.4.45). Many of these cracks occurred because the groove-welded element was considered a “secondary” attachment with no established weld quality criteria (e.g., splices in longitudinal web stiffeners or back-up bars). Intersecting welds can provide a path for the crack to travel between steel members (see Figure 6.4.46).

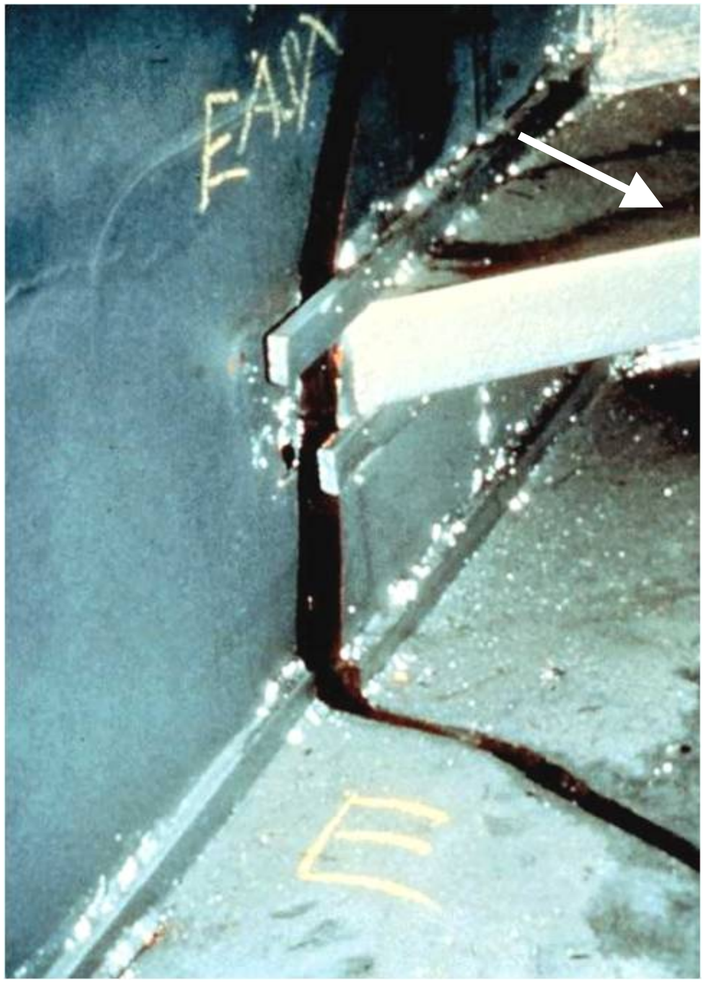


Figure 6.4.45 Poor Quality Welds Inside Cross Girder

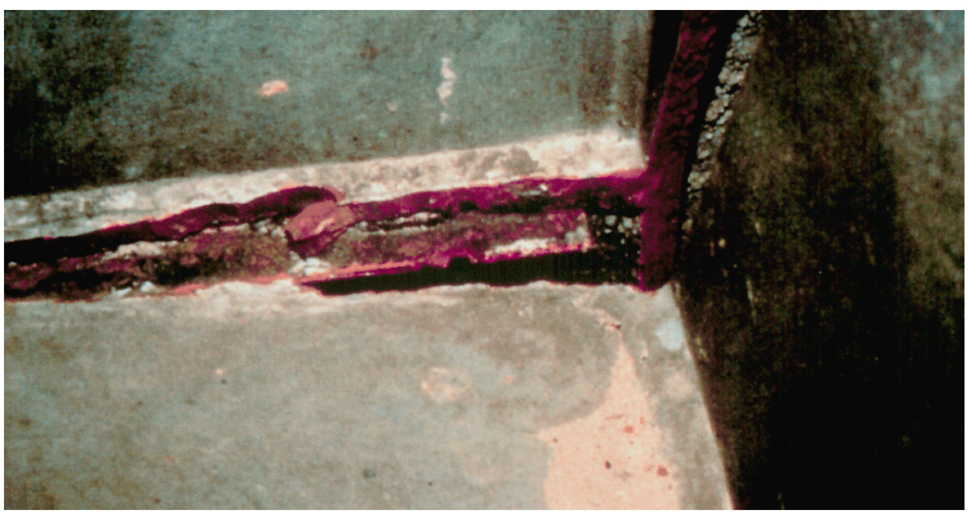


Figure 6.4.46 Intersecting Welds

6.4.8 Inspection Methods and Locations

While FCMs are to be given special attention, take care to assure that the remainder of the bridge or non-fracture critical members are not ignored and that they are also inspected thoroughly. Bridge plans and shop drawings for bridges designed after about 1980 are to have FCMs clearly identified. If FCMs are not clearly identified, use the guidelines previously described in this Topic along with the aid of a structural engineer to determine FCMs.

According to the National Bridge Inspection Standards, a fracture critical member inspection is defined as a hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation. FCM inspections are to be performed at intervals not to exceed twenty-four months. Certain FCMs may require a frequency of less than twenty-four months. Establish criteria to determine the level and frequency of these inspections based on such factors as age, traffic characteristics, AASHTO fatigue categories, and known deficiencies.

Methods

Visual

The inspection of steel bridge members for deficiencies is primarily a visual activity. Most deficiencies in steel bridges are first detected by visual inspection. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being inspected, is required. More exact visual observations can also be employed by cleaning suspect areas, removing paint when necessary, and using a magnifying unit.

Physical

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Removal of paint can be done using a wire brush, grinding, or sand blasting, depending on the size and location of the suspected deficiency. Take care in cleaning when the suspected deficiency is a crack. When cleaning steel surfaces, avoid any type of cleaning process, such as blasting or excessive grinding which may tend to close the crack. The use of degreasing spray before and after removal of the paint may help in revealing the deficiency.

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

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

Advanced Inspection Methods

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

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

Other methods for determining material properties, described in Topic 15.3.3, include:

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

Inspection of Details

Focus fracture critical member inspection on those details that are most susceptible to fatigue problems.

Of all the details, those of AASHTO fatigue Categories D through E′ are the most susceptible to fatigue crack growth. If Category D, E or E′ details are found on a FCM, carefully inspect them during each inspection and their presence clearly documented in the inspection report.

Recordkeeping and Documentation

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

When a deficiency is encountered in a FCM, record relevant information carefully and thoroughly, including:

- Method of inspection: visual, physical or advanced (see Topics 6.3 and

15.3)

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

Label the member using paint or other permanent markings: mark the ends of the crack, the date, and compare to any previous markings. Coordinate with the bridge owner to see what is acceptable to mark on the bridge. Be sensitive to aesthetics at prominent areas. Photograph and sketch the member and the deficiency.

Refer to Topics 4.4 and 4.6 for general record keeping, documentation and inspection report writing.

Recommendations

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

Urgent repairs - repairs that are required immediately in order to maintain the life of the structure or to keep the bridge open; these repairs are for bridge-threatening deficiencies. Urgent repairs are considered to be a critical finding. See Topic 4.5 for detail explanation of critical findings and appropriate plans of action.

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

Locations

Problematic Details

Problematic details may exist on a variety of steel bridges such as girder, frame, truss superstructures, and substructure components. The following are problematic details, which can lead to fatigue cracking:

- Triaxial constraint
- Intersecting welds
- Cover plates
- Cantilevered suspended span
- Insert plates
- Field welds: patch and splice plates
- Intermittent welds
- Out-of-plane bending
- Pin and hanger assemblies
- Back-up bars
- Mechanical fasteners and tack welds

Triaxial Constraint

Triaxial constraint leads to plastic constraint and brittle fracture. This fracture condition can be produced by a narrow gap between the gusset plate and transverse connection/stiffener plate (see Figure 6.4.47). Elastic stress results indicate that triaxial constraint will prevent yielding of the steel until the stress exceeds approximately 1.3 times the yield strength of the material. Under high plastic constraint, local stresses can reach 2 to 3 times the average stress.

Figure 6.4.47 Local Triaxial Constraint Condition Resulting in Fracture on the Hoan Bridge, Milwaukee, Wisconsin

Intersecting welds

Intersecting welds are defined as welds that run through each other, overlap, touch, or have a gap between their toes of less than 1/4 inch (see Figure 6.4.48). This problematic detail allows for alternate, unanticipated stress paths that may act as stress risers, leading to crack initiation. Intersecting welds are not fatigue related or material dependent and may consequently occur under low stress levels in a ductile material with good toughness properties. Additionally, intersecting welds may leave large residual stresses after welding, leading to possible cracking and reduced fatigue strength. Welds are terminated short of the intersection by at least 1/4 inch to avoid intersecting welds. In most cases, it is desirable to allow the longitudinal weld (parallel with the applied stress) to be continuous. This avoids Category E type detail at the weld termination if it is interrupted. The end termination of a transverse weld does not directly affect its fatigue strength and is classified as Category C' for plates.

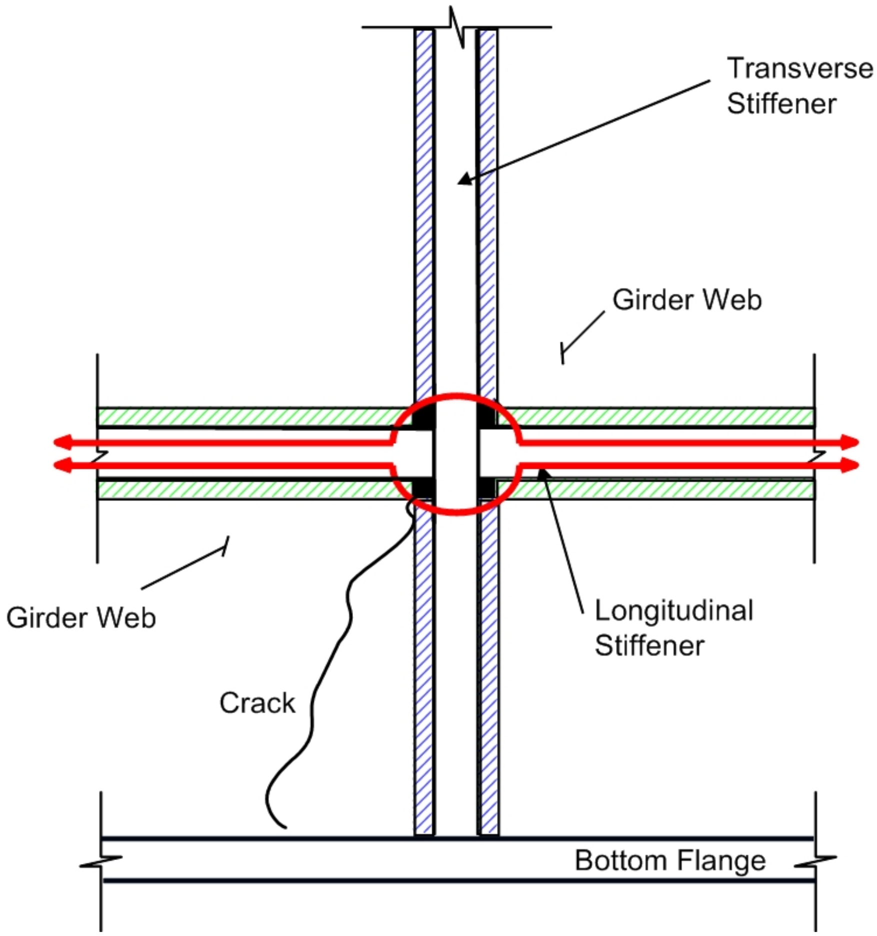


Figure 6.4.48 Potential Crack Formation due to Intersecting Welds

Cover Plates

Partial length cover plates, popular from the 1940s to 1970s, allowed designers to increase the flexural capacity of a beam by welding plates onto the flanges to increase the flange section, typically at the midspan of the beam or over interior supports of continuous spans. This detail combines the fatigue problems associated with a sudden change in cross-sectional area, residual stresses that accumulate at the end of a welded plate, and welding across a tension flange. Despite several attempts to eliminate crack initiation through the use of different end treatments, the cracks normally initiate at the weld toe and then propagate into the base metal flange and finally into the web (see Figure 6.4.49).

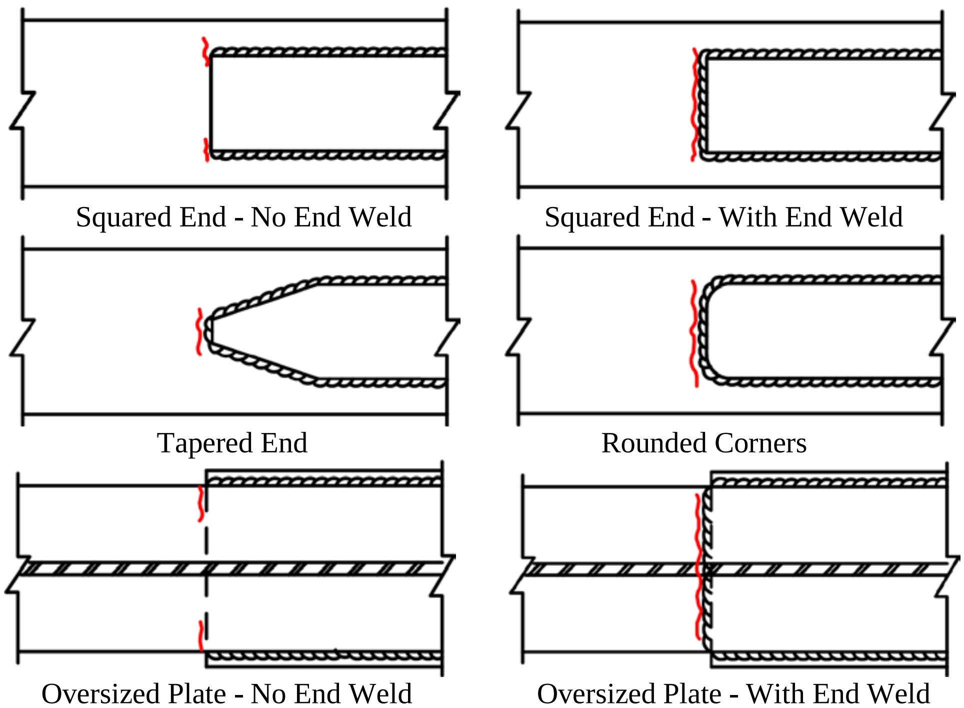


Figure 6.4.49 Potential Crack Formation for Various Cover Plate End Treatments

Some bridge owners have peened the ends of cover plates to induce residual compressive stresses to deter the formation of cracks.

Cantilevered-Suspended Span

This type of span configuration utilizes one or two cantilever arms to support a suspended (simple) span. This practice allowed designers of the 1960s to dictate a zero-moment condition (or hinge) while moving the deck joints away from substructure piers and bearing devices. As a result of this configuration, the top flange of the cantilevered span and the bottom flange of the suspended span are in tension. Examine these areas closely. Inspect this detail for horizontal and vertical alignment and accelerated corrosion due to drainage from expansion joints in the deck. Potential crack locations are illustrated in Figure 6.4.50. Closely examine the connections between the supports and stiffeners since many cracks initiate at the weld toe or root of connecting members.

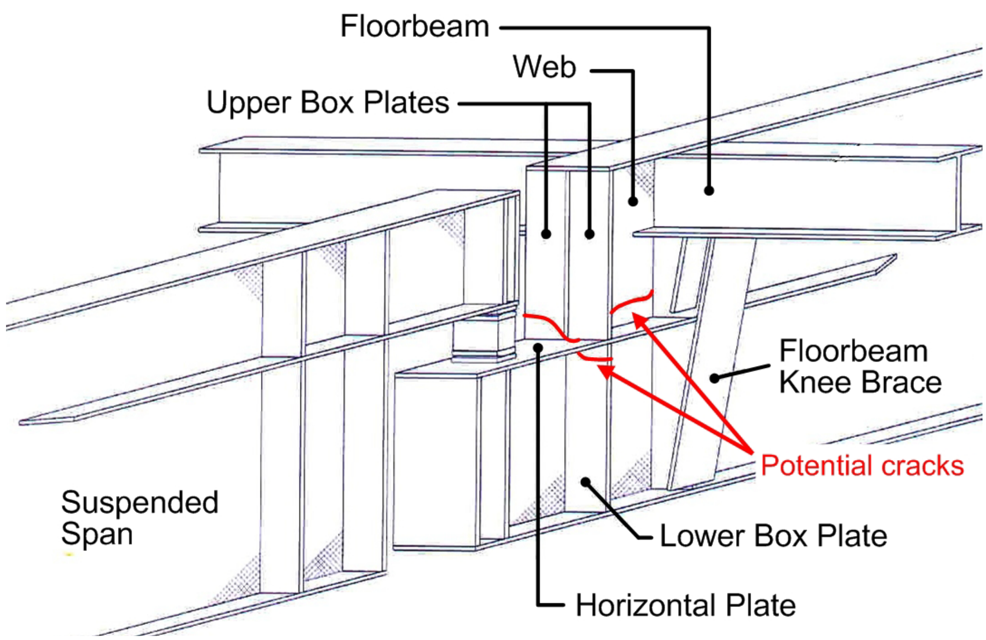


Figure 6.4.50 Potential Crack Formation for Cantilever Suspended Span (Potential Cracks Shown in Red)

Insert Plates

Insert plates are sometimes used to vary the depth of a girder. This detail may contain a vertical weld, which is subject to crack development similar to that found in a full width web splice. Both longitudinal and transverse welds are used to connect the insert plate to the girder. Transverse (vertical) welds are perpendicular to the bending stresses in the flange and web and may see stress reversal due to live load. In some cases, the weld is ground flush only on the fascia side of the exterior beam, leaving stress risers on the interior side. Cracks may initiate in the vertical web weld and propagate through the width of the flange and up the web base metal (see Figure 6.4.51). Insert plate vertical welds at the ends of spans are also susceptible to crack initiation. These low stress regions can crack due to lack of fusion in the weld connecting them to the girder (see Figure 6.4.52).

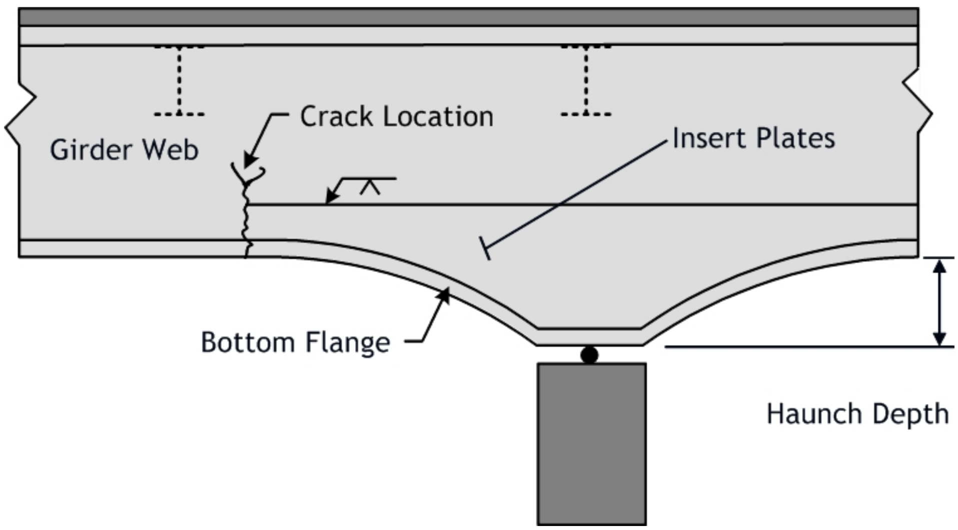


Figure 6.4.51 Potential Crack Formation in Vertical Web Weld at Haunch

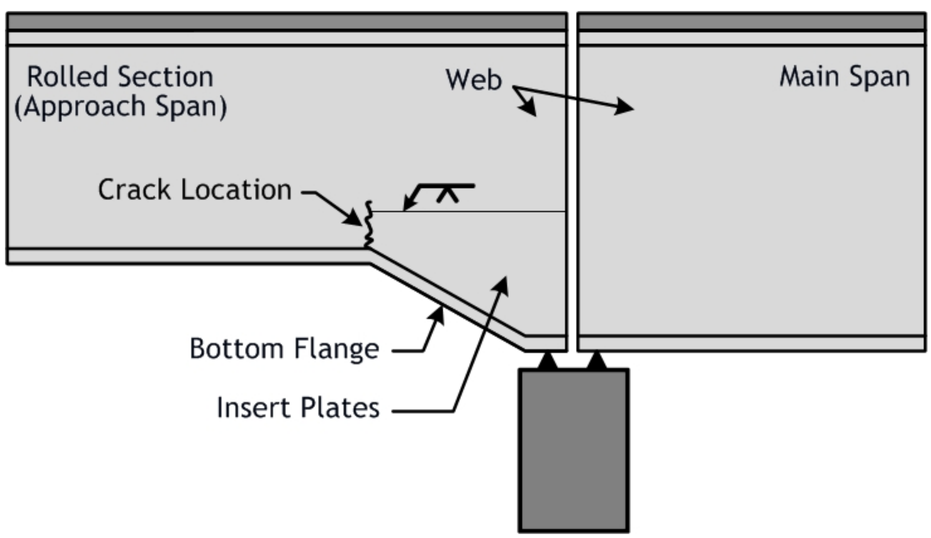


Figure 6.4.52 Potential Crack Formation in Vertical Web Weld at End of Span

Field Welds: Patch and Splice Plates

Patch plates may be added to increase the total (or static) strength of the girder, repair corroded areas, or correct fabrication errors. For older steels, welding patch plates are often problematic since the welds may be perpendicular to the primary stresses and the chemical composition of older steels leads to brittleness when welded. Retrofits such as welding patch plates to flanges and webs or welding stringer ends to floorbeams are examples of potential problematic areas (see Figure 6.4.53). Aside from patch plates, closely examine splice plate welds perpendicular to tensile stress caused by axial or bending forces.

Figure 6.4.53 Field Welds Perpendicular to Bending Stresses are Susceptible to Cracking

Intermittent Welds

Intermittent welds, also referred to as stitch welds, are discontinuous welds used to connect steel bridge members. The nature of stop and start nature of these non-continuous fillet welds are susceptible to lack of fusion (see Figure 6.4.54). This practice contributed to stress riser locations and was abandoned by the mid 1970s. However, many stitch-welded member bridges are still in-service today.

Figure 6.4.54 Intermittent or Stitch Welded Transverse Stiffeners

Out-of-Plane Bending

Deflection of floorbeams or diaphragms can cause out-of-plane distortion in the girder webs. Out-of-plane distortion occurs across a small web gap between the flanges and end of vertical connection plates (see Figure 6.4.55). Two very common instances of out-of-plane distortion are in the web gap floorbeam connection and the lateral bracing gusset plate connection. The deck prevents rotation at the top gap, while the bearing prevents rotation at the bottom gap. Cracks caused by out-of-plane distortion are not covered in the AASHTO Fatigue Categories A - E'.

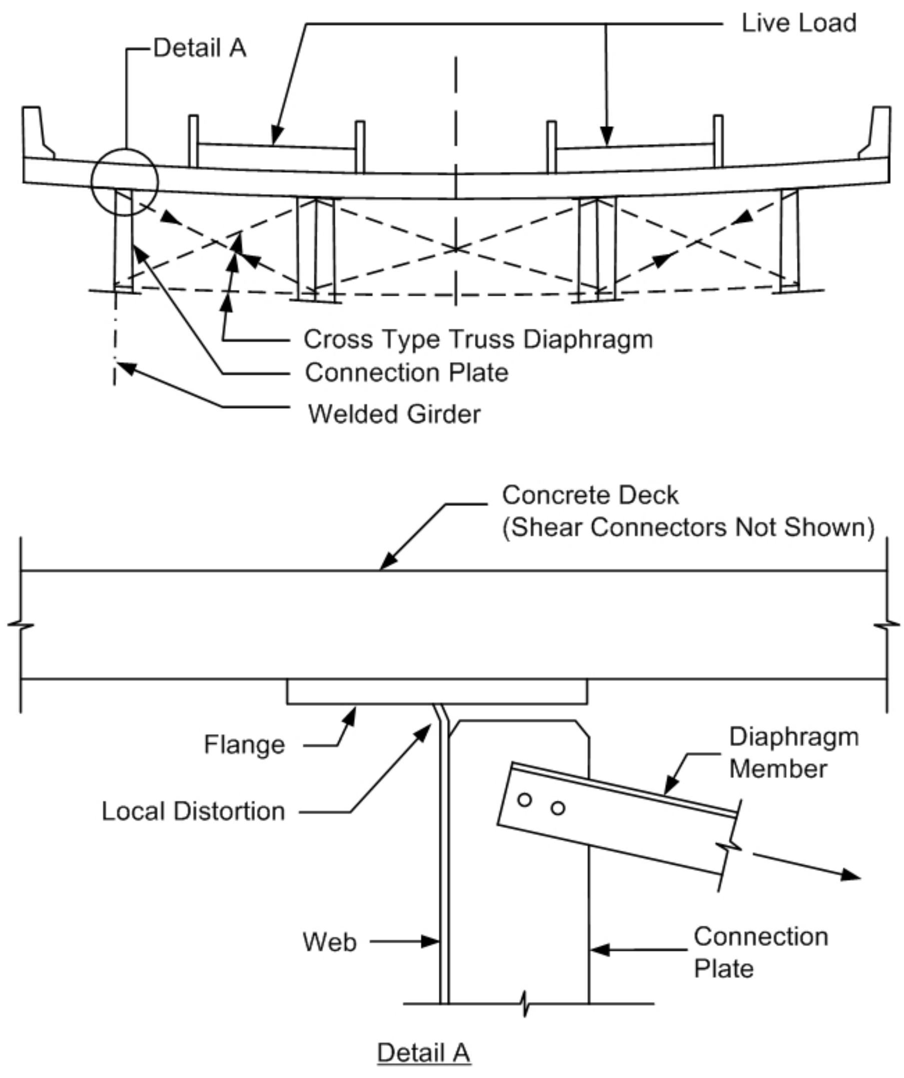


Figure 6.4.55 Out-of-Plane Distortion in Web Gap at Diaphragm Connections

Bridge members are normally designed to resist axial tension or compression, shear, or in-plane bending (parallel to the longitudinal axis). A loading or distortion may occur to produce torsion or twisting about the member's longitudinal axis. Out-of-plane bending results from this torsion. The following examples are some common areas for out-of-plane bending:

- Girder web connections for diaphragms and floorbeams - Girder web connections may exhibit out-of-plane bending due to floorbeam/diaphragm end rotation from live load. Vertical connection plates used to transmit the out-of-plane forces to the girders are often sufficient. However, the structural details at the ends of the connection plates are sometimes inadequate to accommodate the deflections and rotations. This semi-flexible connection is commonly referred to as a "web-gap" problematic detail (see Figure 6.4.56).

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

  Connection plate at bottom flange - At the lower end of diaphragm connection plates which are not welded to the tension flange of girders, the condition of local out-of-plane distortion and bending of the web plate usually is less severe. This is because the tension flange is not restrained from lateral movement, which is sufficient to reduce the web plate bending. However, if the bottom flange is restrained from lateral deflection (e.g., at bearings), fatigue cracks will develop along the web to flange weld (see Figure 6.4.57).

  Connection plate at bottom flange for skewed bridges - Fatigue cracking may also develop at the unattached lower end of diaphragm connection plates for skewed bridges. Most of these diaphragms are perpendicular to the girders and thus are subjected to large differential vertical deflections which in turn cause out-of-plane distortion at the lower end of the diaphragm connection plate. If the girder flange is relatively thick and stiff against lateral displacement, most of the deflection is accommodated by bending of the web plate within the gap between the flange and the end of the connection plate welds. Fatigue cracks may initiate at the bottom of the vertical plate and grow upward in a U-shape before propagating horizontally into the web. "Bleeding" of the crack indicates that there is relative movement of the crack surface, and moisture will combine with the oxide to streak down the surface. Frequently inspect severely skewed bridges with relatively heavy flanges at the lower ends of diaphragm connection plates if these connection plates are not attached to the bottom flange.

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

- Staggered floorbeams or lateral gusset plate locations - Skewed bridges often use staggered floorbeams or lateral bracing gusset plate locations, which may be susceptible to out-of-bending similar to unattached lower ends of diaphragm connection plates for skewed bridges (see Figure 6.4.58).

- Lateral bracing gussets and diaphragm connection plates - Many fatigue cracks resulting from out-of-plane distortion of girder webs have been detected in web plates at the junction of lateral bracing gussets and diaphragm connection plates. The unequal lateral forces from the bracing members introduce lateral deflection and twisting of the junction in the direction perpendicular to the web. If the gusset plate is not attached to the vertical connection plate, the web plate in the small horizontal gap between the gusset plate and the connection plate is subjected to relative out-of-plane distortion and development of fatigue cracking. The vertical deflection of the lateral bracing causes stresses in the lateral bracing gusset plates. The welds connecting the lateral bracing gusset plates to the girder web may experience fatigue cracking (see Figure 6.4.59).

- Diaphragm connections to gusset plates - The diaphragm components may be connected to gusset plates, which are welded to the vertical connection plates. The ends of the groove weld between the gusset plate and the connection plate have an abrupt change in plate geometry with re-entrant corners at the top of the connection plates. Fatigue cracks have developed in this region and unless these fatigue cracks are accompanied by movement and by oxide powder, their existence may not be obvious. Careful inspection from both sides of the diaphragm is necessary.

- Cantilevered floorbeams may also produce out-of-plane bending as the stringer attempts to deflect more than the main superstructure girder (see Figures 6.4.60 and 6.4.61).

Figure 6.4.56 Web Crack due to Out-of-Plane Distortion at Top Flange

Figure 6.4.57 Web Crack due to Out-of-Plane Distortion at Bottom Flange

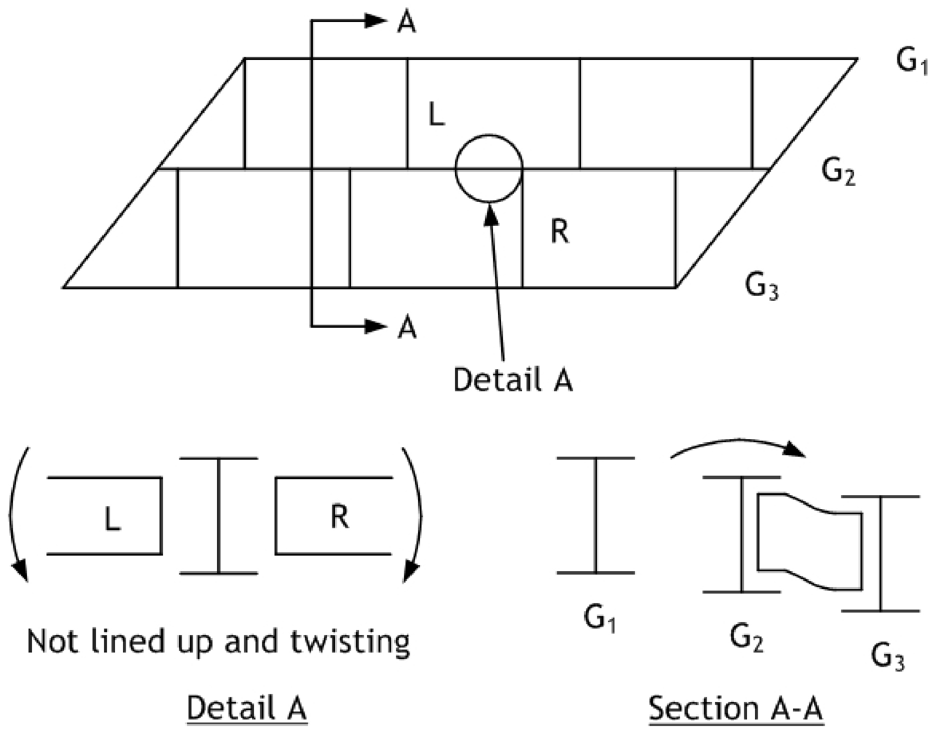


Figure 6.4.58 Skewed Bridge Producing Out-of-Plane Bending due to Differential Deflection of Floorbeams and Girders

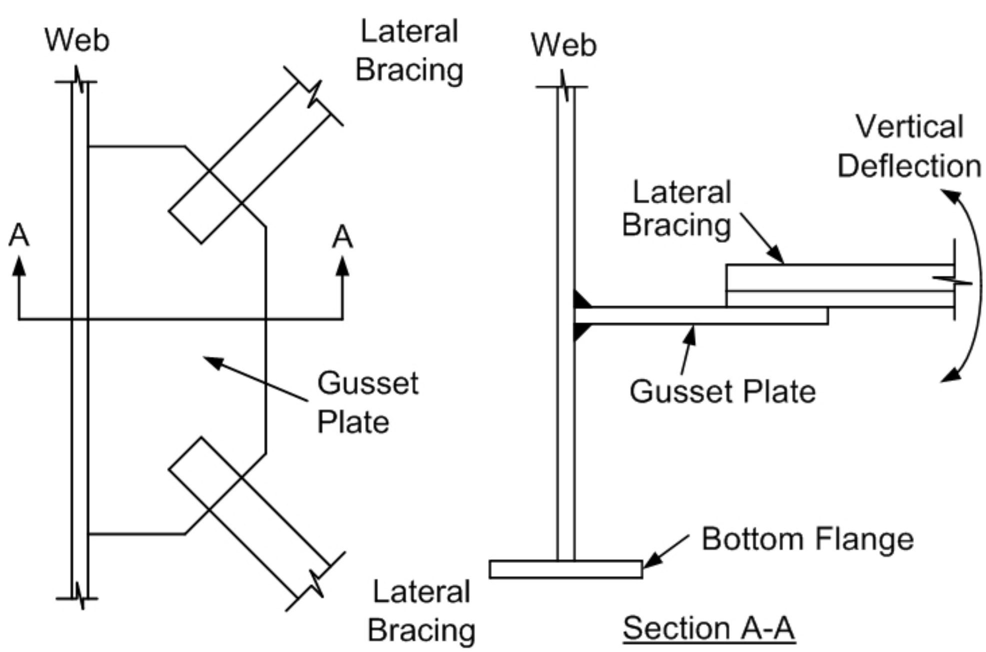


Figure 6.4.59 Lateral Bracing Deflections Producing Out-of-Plane Bending

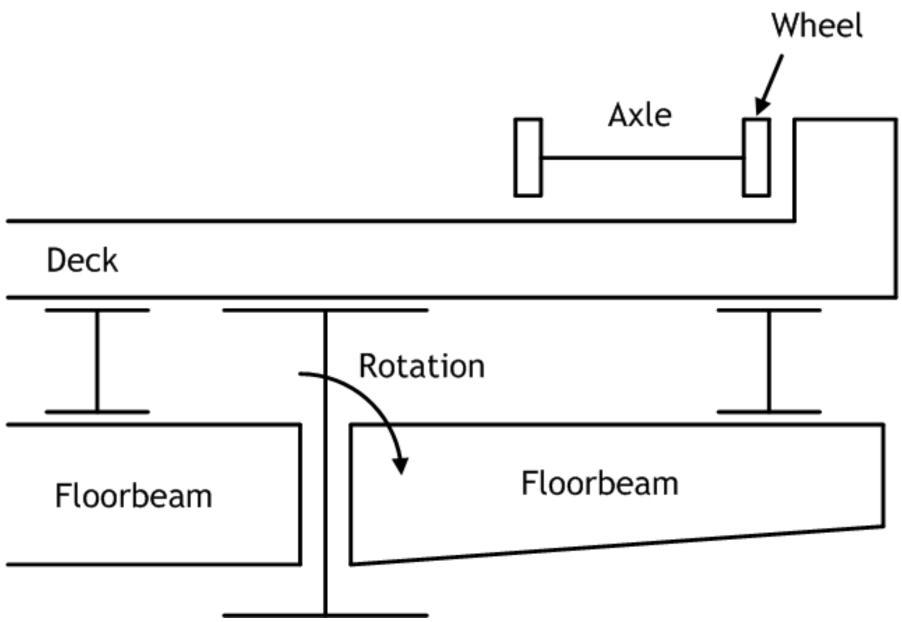


Figure 6.4.60 Cantilevered Floorbeam Producing Out-of-Plane Bending due to Differential Deflection of Stringer

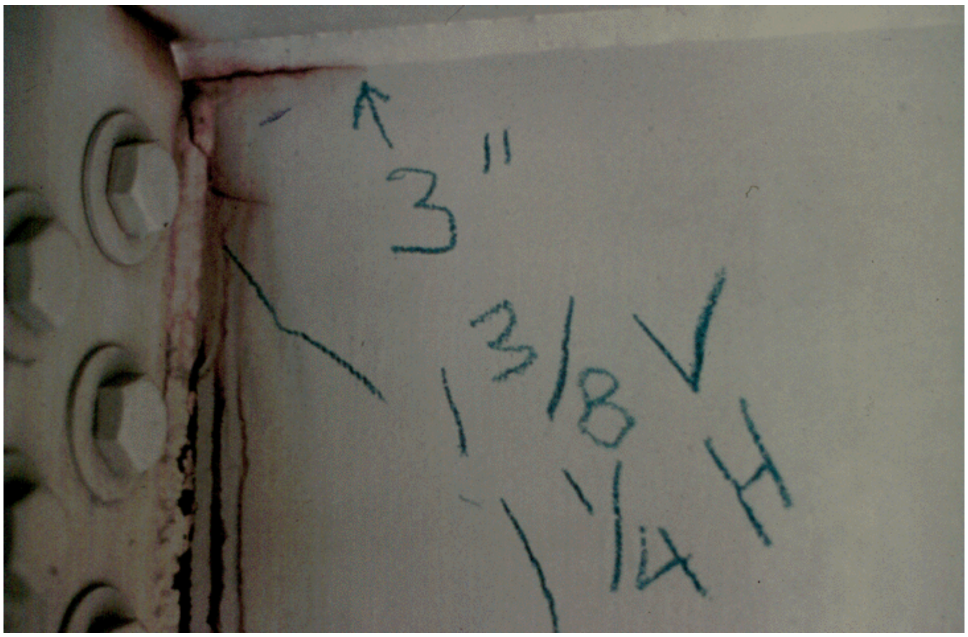


Figure 6.4.61 Cracks at Top of Floorbeam Connection to Girder

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

Pin and Hanger Assemblies

Pin and hanger assemblies are usually found in two-girder or multi-girder bridges constructed prior to the 1970s. Similar to the cantilevered-suspended span, pin and hanger connections simplified the design and analysis by introducing a hinge while moving the deck expansion joints away from substructure piers and bearing devices. Corrosion of the pin and hanger assembly may be accelerated since drainage is typically free to fall directly through the deck expansion joints onto the pin and hanger assembly. While normally designed for bearing and shear forces, the corrosion of the pin may prevent rotation and subsequently introduce torsional loading. Hangers normally designed to act in axial tension may experience in-plane bending when the pins are not free to rotate in the hanger opening. Pack rust expands between the hanger and girder web resulting in out-of-plane bending in the hanger. Refer to Topic 10.7 for more information regarding pin and hanger assemblies.

Back-Up Bars

Back-up bars are designed to prevent groove welds from blowing out the base metal during fabrication. In the past, tack welds have been used to attach the back-up bars and temporarily hold into place until after the groove weld has been placed. Common practice was to leave the tack welds in place. However, since they are connected, the stresses travel back and forth between the web, back-up bar, and flange. When a gap occurs in the back-up bar, the stress will abruptly change direction and enter the flange and/or web before returning to the back-up bar. This abrupt change causes stress risers at the tack weld and back-up bar gaps (see Figure 6.4.62).

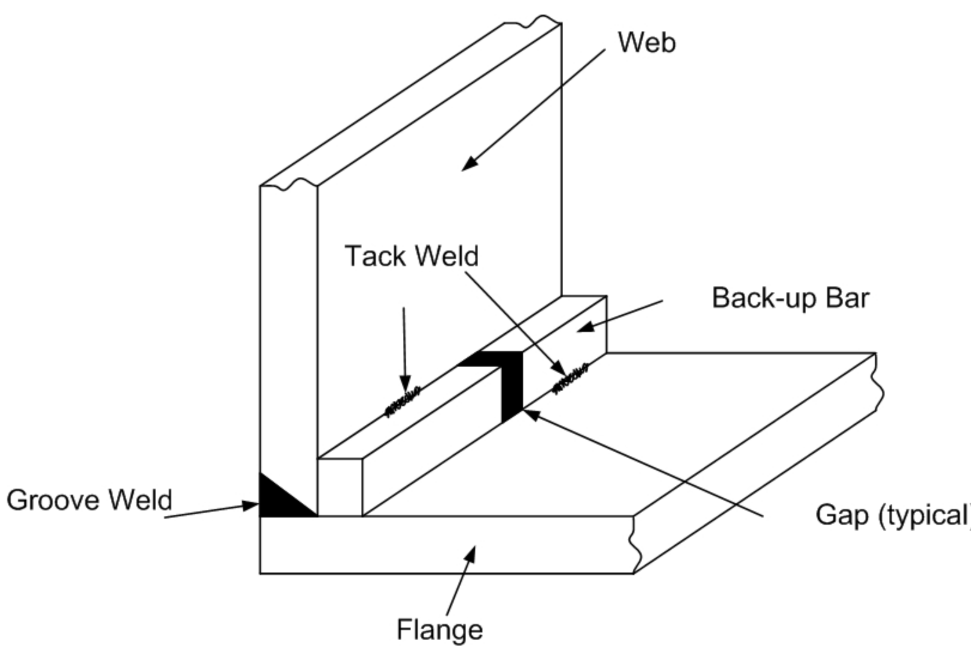


Figure 6.4.62 Back-Up Bars Tack Welded to Web and Flange Potentially Producing Abrupt Stress Reversal and Stress Risers

Mechanical Fasteners and Tack Welds

If the girder is riveted or bolted, check all rivets and bolts to determine that they are tight and in good condition (see Figure 6.4.63). Check for cracked or missing bolts, rivets and rivet heads. Check the base metal around the bolts and rivets, especially those located within the tension flange or tension member. Bolts have a fatigue classification of Category B and rivets are classified as D. Category D can be changed to Category B if the rivets are replaced with bolts and tightened to high strength bolt specifications.

Look for existing tack welds. These welds were typical in older structures and were used to temporarily hold members in place. This practice has since been deemed unacceptable, as tack welds may act as stress risers and are prone to fatigue cracking.

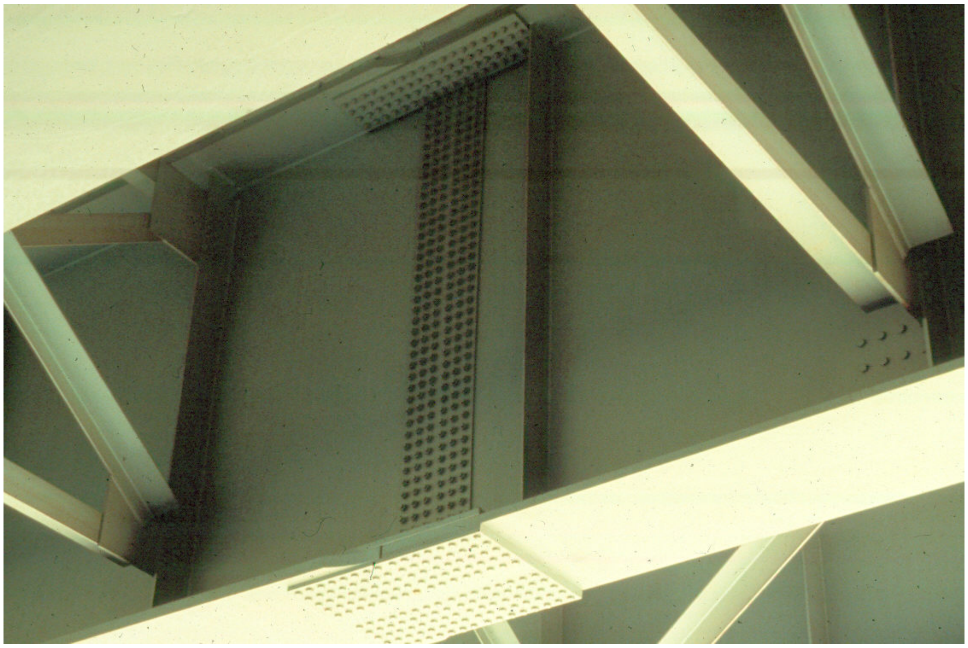


Figure 6.4.63 Bolted Field Splice

Miscellaneous Connections

Check any attachment welds located in the superstructure tension zones, such as traffic safety features, lighting brackets, utility attachments, catwalks and signs (see Figure 6.4.64).

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

Figure 6.4.64 Welded Attachment in Tension Zone of a Beam

Flange Terminations

It is also common to terminate the flange before the end of the member to facilitate fabrication (see Figure 6.4.65). When one or both flanges are removed, as in a blocked flange cut, the web plate has a lower cross section as compared to the entire member. This can increase the stress in the web plate where the flange is terminated.

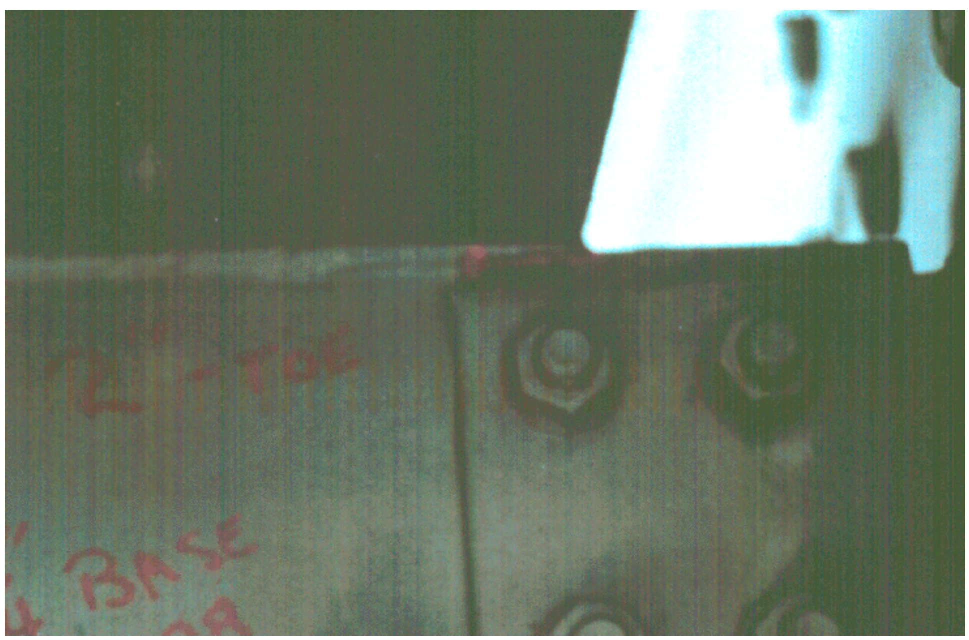


Figure 6.4.65 Flange Termination

Coped Flanges

Coping or cutting away of the flange and portion of the web, may be necessary to connect stringers, floorbeams, diaphragms and the main girders. Copes are often flame cut, resulting in residual tensile stresses along the cut edges, approaching the yield point (see Figure 6.4.66).

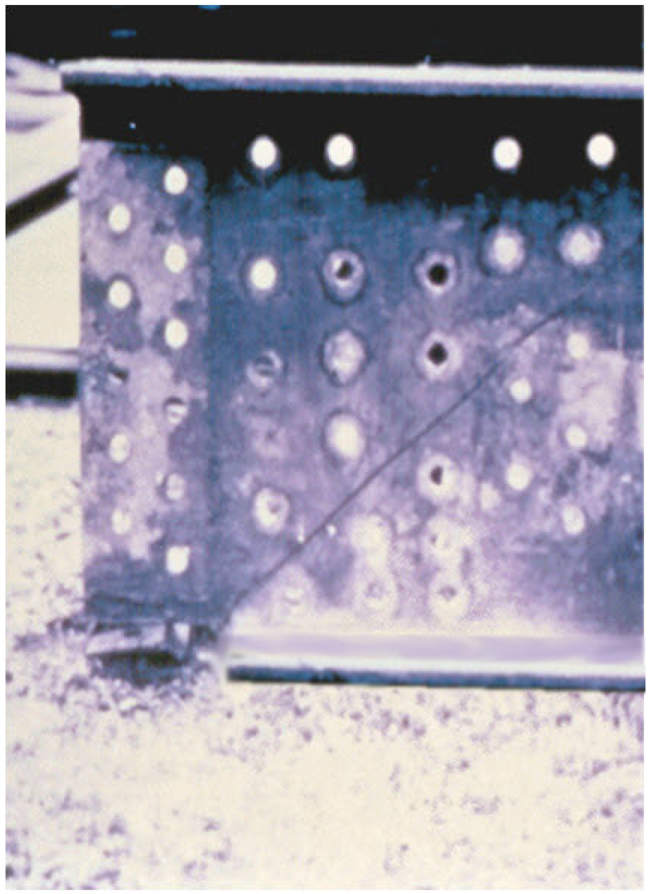


Figure 6.4.66 Fracture of a Coped Member

Blocked Flanges

Blocking a flange is done in a similar fashion and for the same reason as coping, however only half of the flange width is removed and the web plate is unaffected (see Figure 6.4.67).

Figure 6.4.67 Blocked Floorbeam Flange

Crack Orientation

Cracks Perpendicular to Primary Stress

Cracks perpendicular to primary stress are very serious because all stresses applied to the member work towards propagating the crack (see Figure 6.4.68). Report them immediately so that repairs can be performed.

Cracks Parallel to Primary Stress

Cracks parallel to primary members are less serious than transverse cracks. Cracks parallel to the main direction of stress, do not reduce the capacity load and have less tendency to propagate. These cracks are still important because they can turn perpendicular to the direction of stress at any time (see Figure 6.4.68).

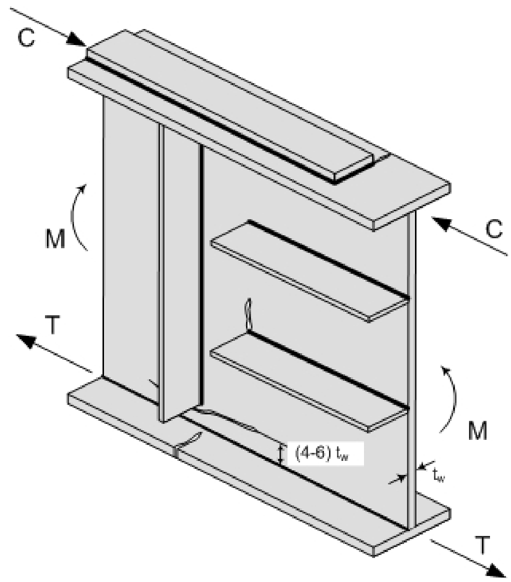


Figure 6.4.68 Cracks Perpendicular or Parallel to Applied Stress

Corrosion Areas

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

Nicks and Gouges

The bridge engineer responsible for the rating of the structure often evaluates any nicks or gouges because they cause stress concentrations and may result in fatigue cracking. If large nicks or gouges are found, evaluate these nicks or gouges in a manner similar to section loss occurring due to corrosion. Additionally, large nicks or gouges may be ground smooth in the direction of the stress to reduce stress concentrations.

6.4.9 Evaluation

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

NBI Rating Guidelines and Element Level Condition State Assessment

Refer to Topics 7.4, 10.1 through 10.9, 12.1 and 12.2 for specific rating guidelines for the various types of common steel bridge components.

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Topic 6.5 Stone Masonry

6.5.1 Introduction

Stone masonry is seldom used in new bridge construction today except as facing or ornamentation. However, many old stone bridges are still in use and require inspections (see Figure 6.5.1). Granite, limestone, and sandstone are the most common types of stone that were used and are still seen today in bridges. In addition, many smaller bridges and culverts were built of locally available stone. Stone masonry typically has a unit weight of approximately 170 pounds per cubic foot (pcf).

Figure 6.5.1 Stone Masonry Arch

6.5.2 Properties of Stone Masonry

The physical properties of stone masonry in bridge applications are of primary concern. Strength, hardness, workability, durability, and porosity properties of both the stone and the mortar play important roles in the usage of stone masonry.

Physical Properties

The major physical properties of stone masonry are:

- Hardness – the hardness of stone varies based on the stone type. Some types of sandstone are soft enough to scratch easily, while other stones may be harder than some grades of steel.
- Workability - measures the amount of effort needed to cut or shape the stone. Harder stones are not as workable as softer stones.
- Porosity – porosity in a stone indicates the amount of open or void space

within that stone. Stones have different degrees of porosity. A stone that is less porous can resist freeze/thaw action better than a stone with a higher degree of porosity. Water absorption is directly related to the degree of porosity.

Mechanical Properties

The major mechanical properties of stone masonry are:

- Strength – a stone generally has sufficient strength to be used as a load-bearing bridge member, even though the strength of an individual stone type may vary tremendously. As an example, granite's compressive strength can vary from 7,700 to 60,000 psi. For the typical bridge application, a stone with a compressive strength of 5,000 psi is acceptable. The mortar is almost always weaker than the stone.
- Durability – durability of a stone depends on how well it can resist exposure to the elements, rain, wind, dust, frost action, heat, fire, and air-borne chemicals. Some stone types are so durable that they are able to effectively resist the elements for two hundred years, while other stone types deteriorate after about ten years.

Mortar

Mortar is primarily composed of sand, cement, lime and water. The cement is generally Portland cement and provides strength and durability. Lime provides workability, water retentivity and elasticity. Sand is filler and contributes to economy and strength. The water, as in the case of concrete, can be almost any potable water. See Topic 6.2 for more information on mortar.

6.5.3 Stone Masonry Construction Methods

There are three general methods of stone masonry construction:

- Rubble masonry
- Squared-stone masonry
- Ashlar Masonry

Rubble Masonry

Rubble masonry consists of rough stones which are un-squared and used as they come from the quarry. It could be constructed to approximate regular rows or courses (coursed rubble) or could be un-coursed (random rubble). Random rubble was the least expensive type of stone masonry construction and was considered strong and durable for small spans if well constructed.

Squared-Stone Masonry

Squared-stone masonry consists of stones, which are squared and dressed roughly. It could be laid randomly or in courses.

Ashlar Masonry

Ashlar consists of stones, which are precisely squared and finely dressed. Like squared-stone masonry, it could be laid randomly or in courses.

6.5.4 Anticipated Modes of Stone Masonry and Mortar Deficiency

The primary types of deterioration in stone masonry are:

- Weathering – hard surfaces degenerate in to small granules, giving stones a smooth, rounded look; mortar disintegrates
- Spalling – small pieces of rock break out
- Splitting – seams or cracks open up in rocks, eventually breaking them into smaller pieces (see Figure 6.5.2)
- Fire – masonry is not flammable but can be damaged by high temperatures

Figure 6.5.2 Splitting in Stone Masonry

Some of the major causes of these forms of deterioration are:

- Chemicals – gases and solids, such as deicing agents, dissolved in water often attack stone and mortar; oxidation and hydration of some compounds found in rock can also cause damage
- Volume changes – seasonal expansion and contraction can cause fractures to develop, weakening the stone
- Frost and freezing – water freezing in the seams and pores can spall or split stone or mortar

- Abrasion – due primarily to wind or waterborne particles
- Plant growth – roots and stems growing in crevices and joints can exert a wedging force, and lichen and ivy can chemically attack stone surfaces
- Marine growth – chemical secretions from rock-boring mollusks deteriorate stone

Two major factors that affect the durability of stone masonry include:

- The proper curing of mortar
- Correct placement of stones during construction

6.5.5 Protective Systems

The different types of protective systems used for concrete can also be used for stone masonry. The two most common systems that are used are paints and water repellant membranes or sealers. See Topic 6.2 for a complete description of the different types of protective systems.

6.5.6 Inspection Methods for Stone Masonry and Mortar

The examination of stone masonry and mortar is similar to that of concrete. There are three basic methods used to inspect stone masonry and mortar. They include:

- Visual
- Physical
- Advanced inspection methods

Inspection techniques are generally the same as for concrete (see Topic 6.2 for the examination of concrete).

Visual Examination

Carefully inspect the joints for cracks, loose or missing mortar, vegetation, water seepage and other forms of mortar deterioration. Also, carefully inspect the stones for cracks, crushing, missing, bulging, and misalignment. Check masonry arches or masonry-faced concrete arches for mortar cracks, vegetation, water seepage through cracks, loose or missing stones or blocks, weathering, and spalled or split blocks and stones.

Physical Examination

Areas of stone masonry deterioration identified visually also be examined physically using an inspection hammer. This hands-on effort verifies the extent of the defect and its severity.

Hammer sounding is commonly used to detect areas of delamination and unsound stone masonry. A delaminated area has a distinctive hollow “clacking” sound when tapped with a hammer. A hammer hitting sound stone masonry result in a solid "pinging" type sound.

The location, length and width of cracks found during the visual inspection and sounding methods are given special attention. A crack comparator 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 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 crack width guidelines, see Topic 6.2.

Advanced Inspection Methods

If the extent of the stone masonry defect cannot be determined by the visual and/or physical inspection methods described above, advanced inspection methods may be used. Nondestructive methods, described in Topic 15.2.2, include:

- Acoustic Wave Sonic/Ultrasonic Velocity Measurements
- Flat Jack Testing
- Impact-Echo Testing
- Infrared Thermography
- Rebound and Penetration Methods
- Ultrasonic Testing

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Topic 6.6 Fiber Reinforced Polymer (FRP)

6.6.1 Introduction

Fiber Reinforced Polymer (FRP) is a modern bridge material that is becoming increasingly popular throughout the transportation community. First used in the United States in the early 1990s, modern FRP composite bridge applications include new bridge construction (primarily bridge deck members) as well as strengthening and rehabilitation.

Repair and Retrofit of Concrete Members Using FRP Composites

Some of the earliest implementations of FRP as a bridge material involved the repair of existing concrete members using external bonding techniques. FRP composites were applied to concrete pier caps, beams, and girders using laminate/layering methods (see Figure 6.6.1). Through extensive research and analysis, FRP laminate applications were found to increase the flexural strength of structural concrete members while exhibiting very few problems. In some cases, girders repaired using FRP laminate or wrapping techniques performed better than the originally designed member. Based on these findings, externally bonded FRP composite applications have since been confirmed to provide increased shear capacity, control of cracking and spalling, and increased corrosion resistance in harsh marine environments for concrete members.

Figure 6.6.1 Concrete Beam Repaired Using FRP

Seismic retrofitting of concrete structures has also been thoroughly researched following the 1989 Loma Prieta earthquake near Santa Cruz, California. This disaster sparked interest in the California Department of Transportation (Caltrans) for development of FRP composite wraps (see Figure 6.6.2) that would become a viable alternative to steel jacket systems. Similar to FRP composite wrapping techniques used for repair of beams and pier caps, the purpose of the seismic FRP composite wraps is to provide confinement of the concrete and increase ductility over non-wrapped traditional units. FRP composite wrapped columns may also exhibit additional axial capacity, an added benefit which could be used for column strengthening applications.

Thousands of concrete bridge piers and columns across several states have been successfully retrofitted with FRP composite wrap systems. These columns and piers have undergone substantial laboratory and field testing with positive results.

Figure 6.6.2 Seismic Retrofit of Concrete Columns Using FRP Composites

Repair and Retrofit of Steel Members Using FRP Composites

Still in the initial research stages, efforts have also been aimed at using FRP for the repair and retrofit of structural steel members. Research projects have been conducted using carbon fiber reinforced polymer (CFRP) post-tensioning rods and externally bonded CFRP plates to steel I-beams (see Figures 6.6.3 and 6.6.4).

Initial findings suggest that while CFRP strengthening systems may not reduce live load deflections (or increase member stiffness), these methods could return a damaged girder's strength to a pre-damaged level or increase the live load capacity of an undamaged steel girder.

Figure 6.6.3 CFRP Post-tensioned Steel Girder

Figure 6.6.4 Externally Bonded CFRP Plates to Steel Girder Bottom Flange

Repair and Retrofit of Timber Members Using FRP Composites

CFRP strands is becoming more popular for prestressing, especially for transverse post-tensioning of timber decks, but are currently limited in actual field applications. Aside from superior corrosion resistance, the low modulus of elasticity minimizes loss of prestress forces due to the creep of the wood over time. As with steel, the use of FRP composites is currently being researched to determine long term effects.

FRP Decks and Slabs in New Construction

Decks and slabs are the primary use of FRP composites for new bridge construction. FRP decks and slabs can be broken down into three basic categories according to configuration (which often relates to manufacturing process as explained in Topic 6.6.3). At the construction site, the individual panels (typically 8 to 10 feet wide and up to 30 feet in length) are bonded together with high performance adhesives. The system may also be made partially composite by cutting pockets into the deck to access welded shear studs on the top beam flanges and then grouting the pockets. It is important to note that FRP is not a good choice for new designs requiring composite action between the deck and superstructure unless expensive carbon fiber composites are used. However, non-composite action systems can benefit from a significant weight reduction, which lowers the dead load and allows for a greater live load capacity. See Topic 7.3 for more information on FRP decks and slabs.

FRP composites may also be used in concrete decks as a mixture of loose fibers and Portland cement. This combination is known as Fiber Reinforced Concrete (FRC). See Topic 6.6.3 for more information on FRC.

FRP Reinforcement in New Construction

An ongoing challenge in maintaining and preserving conventionally reinforced and prestressed concrete structures is controlling and minimizing the deterioration of the concrete. Concrete deterioration is most often caused internally by the corrosion of steel reinforcement. Given the superior corrosion resistance of FRP composites, the threat of reinforcement corrosion can be eliminated when incorporating glass fiber reinforced polymer (GFRP) or carbon fiber reinforced polymer (CFRP) composite reinforcing bars or plates (see Figure 6.6.5). Steel and timber can also benefit from FRP composite reinforcement. Post-tensioning bars or CFRP plates may be used to increase the live-load capacity of steel girders while timber beams and decks may be prestressed or post-tensioned to increase overall structure performance.

Figure 6.6.5 CFRP Plate and GFRP Reinforcing Bars

Despite significant research, understanding and improvement of FRP composite reinforcement since the 1990s, several challenges have yet to be resolved. One significant concern of FRP reinforcement (and FRP material in general) is failure in a brittle fashion due to the elastic material properties. FRP reinforcing bars may also lead to increased live load deflection and larger crack widths under load due to the lower modulus of elasticity. Properties of FRP are discussed in detail in Topic 6.6.2.

FRP Superstructure Members in New Construction

The majority of FRP decks are supported by steel, concrete, or timber superstructures. However, FRP girders and beams (pultruded sections) are continuing to be researched as a possible alternative to traditional superstructure materials (see Figures 6.6.6 and 6.6.7). FRP suspension and stay cables are also being considered due to a significant reduction in weight over their steel counterparts. Several experimental bridges have been constructed using FRP superstructure members and are generally performing well. These bridges are continuing to be closely monitored through field load tests and bridge inspections.

Figure 6.6.6 Steel I-Beam (back) and Pultruded FRP I-Beam (front)

Figure 6.6.7 Pultruded FRP Double-Web Beam

6.6.2 Properties of Fiber Reinforced Polymer (FRP)

The composition of a matrix resin, reinforcing fibers, and additives determines the applicability of FRP for bridges. Physical and mechanical properties such as weight, formability, strength, stiffness and elasticity, ductility, and corrosion resistance are vital to the continuing development of FRP as a bridge construction material.

Composition

The composition of FRP can be categorized into three major components:

- Matrix resin
- Reinforcement fibers
- Additives

Types of Matrix Resin

There are four popular types of matrix resin currently used for commercially available FRP:

- Orthophthalic polyester – most popular resin for commercially available FRP composites. This general-purpose low performance resin is inexpensive.
- Isophthalic polyester – offers better corrosion and structural performance than ortho-polyester while being less expensive than vinyl esters. This medium-performance resin is the most common used for bridge applications.
- Vinyl esters – increased corrosion resistance and structural performance than iso-polyesters, but at a higher cost. This resin is rarely used outside of demanding environmental conditions.
- Epoxies – physical properties are highly dependent on manufacturing processes but can offer maximum performance. Epoxies are the most expensive type of resin and are consequently not used for bridge applications.

Types and Forms of Reinforcement Fibers

Although many different reinforcement fibers have been developed and tested, few have entered the commercial market due to cost and availability:

- E-glass – lower performance reinforcement fiber that is relatively inexpensive when compared to carbon fiber
- High strength/strain carbon – high performance reinforcement fiber (approximately 50% greater strength than typical glass fiber). Carbon fiber also has 2-3 times the modulus of elasticity compared to glass fiber which reduces live load deflections. This reinforcing fiber is significantly more expensive than glass fiber.

Reinforcing fibers may also be arranged in 5 common forms:

- Continuous roving – bundle of individual strands that are gathered together to form a "roving" (see Figure 6.6.8). This form of fiber reinforcement and may be used in the pultrusion process and will offer highly uniaxial mechanical properties if aligned in a single direction.

Figure 6.6.8 Spools of Continuous Roving

- Discontinuous roving – individual strands that have been chopped into small pieces typically ½ inch to 2 inches in length (see Figure 6.6.9). This form of fiber reinforcement is used in fiber reinforced concrete (FRC) and other applications where lower mechanical properties are sufficient.

Figure 6.6.9 Discontinuous Roving

- Woven roving – glass or carbon fiber roving are woven into a coarse fabric that is commonly used in hand lay-up processes (see Figure 6.6.10). The weave can be made to provide more or less strength in a particular direction by adding or decreasing the number of fibers in that direction.

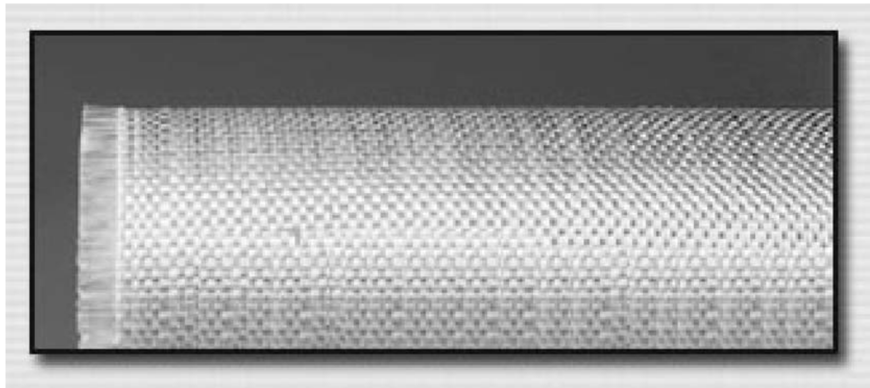

Figure 6.6.10 Woven Roving Fabric

- Mats – mats are produced by attaching continuous or discontinuous roving with a binder (see Figure 6.6.11). As with roving, continuous mats provide higher mechanical properties than discontinuous mats.

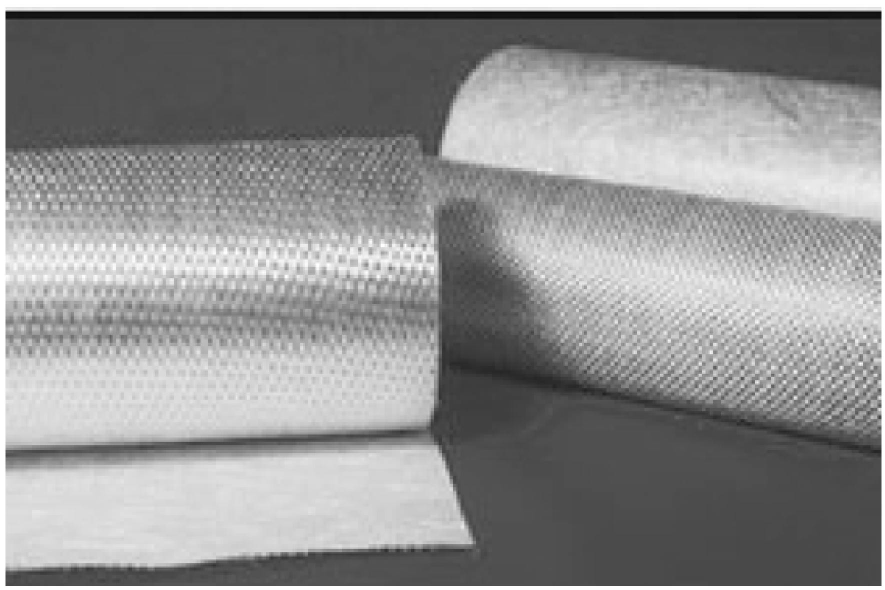

Figure 6.6.11 Discontinuous Roving Mat Fabric

- Non-crimp fabric – reinforcing fibers are stitched or knitted together to produce straight layers of sheet fabric in multiple directions (see Figure 6.6.12). The advantage to non-crimp fabric is the manufacturing of large quantities on single spools that have improved strength and stiffness over other methods. For this reason, non-crimp fabric is popular for the fabrication of deck panels, despite being more expensive than the other forms of fiber reinforcement.

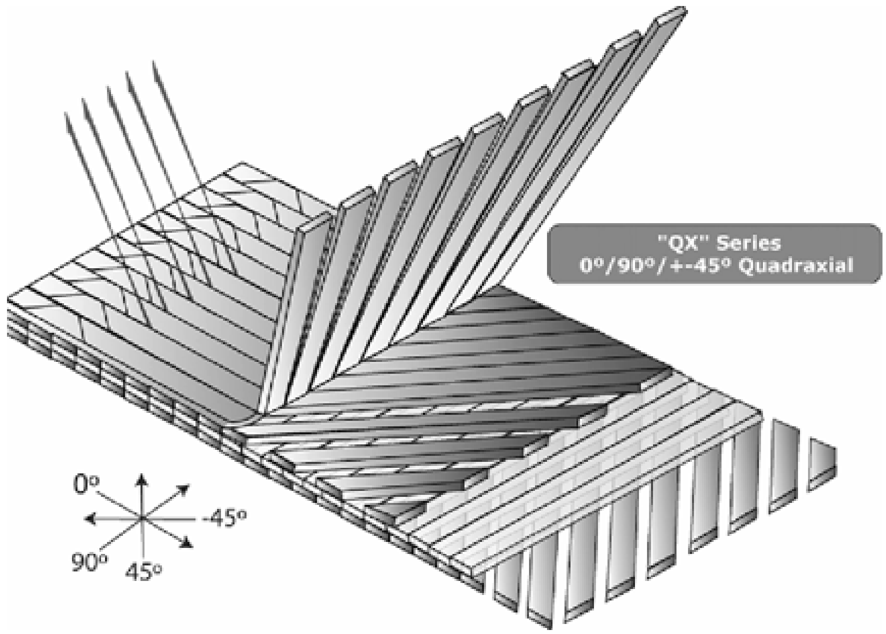

Figure 6.6.12 Non-Crimp Fabric

Types of Additives

Similar to concrete admixtures, other products are added to complete the FRP composite mixture. Depending on the specific application, these ingredients may include fillers, adhesives, light weight foam cores, or gelcoat.

Physical Properties

The major physical properties of FRP are:

- Lightweight – FRP is very lightweight which provides for quick and easy installations of components.
- Formability – can be fabricated into virtually any shape by using different methods
- Thermal expansion – thermal expansion is near zero for CFRP composites and similar to concrete for GFRP composites.
- Porosity – Surfaces exposed to weathering elements should be non-porous as degradation of the matrix and fibers may occur if allowed to penetrate through the surface.
- Fire resistance – FRP is considered to have poor natural fire resistance due to low temperature resistance. Fire resistance can be increased by incorporating fire retardant additives to the flammable resin or applying appropriate surface coatings.

Mechanical Properties

The major mechanical properties of FRP are:

- Strength – the strength of FRP is heavily dependent on the orientation and concentration of the reinforcement fibers and the type of matrix resin and fibers used such that FRP may have isotropic, orthotropic, or uniaxial strength properties. FRP exhibits serviceability in both tension and compression and is very lightweight, resulting in an excellent strength-to-weight ratio. Depending on matrix-resin combination, manufacturing process, and application, strength values may range from 20,000 psi (GFRP) to over 300,000 psi (CFRP).

For FRP deck panels, non-composite action between the deck and superstructure is recommended (unless high strength carbon fibers are used) since GFRP panels cannot resist the additional compression in regions of positive moment (discussed in detail in Topic 7.3).

- Stiffness – similar to the strength, the stiffness of FRP is also heavily dependent on the individual properties and interaction between the matrix resin and fiber reinforcement. Unlike CFRP composites, deflection will usually control the design for GFRP due to the inherently low stiffness of the glass fibers compared to carbon fibers.
- Elasticity – related to the stiffness, the modulus of elasticity of FRP is considerably low for GFRP composites (1,600,000 psi to 6,000,000 psi) but can be increased by incorporating higher strength carbon fibers (18,000,000 psi to 35,000,000 psi). Research has also shown the modulus of elasticity to decrease over time with exposure to environmental elements and cyclic loading, similar to time dependent prestress losses.
- Ductility – FRP composites are very brittle in nature, behaving nearly linear-elastic up to rupture. For this reason, overstress should be avoided by providing reserve capacity well below the point of rupture.
- Corrosion resistance – FRP composites have superior corrosion resistance and should not be impacted by contaminants such as road salts and chlorides.
- Ultraviolet (UV) radiation resistance – UV radiation has been shown to negatively affect polymer-based materials including FRP. Exposure to radiation may result in degradation and hardening of the matrix which is more deleterious in thin sections. Resin additives and surface coatings have been developed to increase the resistance to UV radiation.
- Creep – FRP composites will creep due to sustained loading, especially when exposed to higher temperatures. Creep has been determined to be a behavior of the resin matrix as opposed to the fiber reinforcement.
- Fatigue Resistance – Although fatigue characteristics of FRP composites are limited, research suggests that operating stresses should be kept well below 50% of the material strength.
- Impact Resistance – FRP is considered to have good impact resistance as the resin-fiber structure can absorb energy during collisions at the cost of causing internal damage.
- Durability – the durability of FRP composites in infrastructure environments is still widely unknown considering potential adverse affects from harsh field conditions and repetitive loading. Detailed analyses and studies are continuing to be conducted regarding this topic.

6.6.3 Fiber Reinforced Polymer Construction Methods

Construction methods differ for the two types of fiber reinforced composites used in bridges:

- Fiber Reinforced Polymer
- Fiber Reinforced Concrete

Fiber Reinforced Polymer

With the exception of repair and retrofitting applications, FRP composites are fabricated in a shop and transported to the construction site. This allows for an accelerated schedule with less time spent in the field. The lightweight nature of FRP composites also may eliminate the need for heavy-duty equipment, helping to offset expensive material costs.

The three common methods of manufacturing FRP composites are listed below:

- Hand lay-up
- Vacuum assisted resin-transfer molding (VARTM)
- Pultrusion

Hand Lay-Up

The hand lay-up method is still actively used across all commercial and industrial fields. Each lamination is carefully constructed by arranging the fiber reinforcement and then saturating the reinforcement with a resin matrix. After saturation, the resin is worked into the reinforcement fabric using rollers and paddles. After repeating this procedure for each lamination, the parts are left to cure for a few hours.

This method is very labor intensive and often does not produce uniform results due to the physical labor demanded. The advantage to the hand lay-up method is the ability to fabricate FRP composite parts at a relatively low cost. This advantage is especially true for unique or complex shapes, where more often than not, may only be produced with the hand lay-up process.

For repair, retrofit, and other field applications, the hand lay-up process is exclusively used with the steel or concrete members first thoroughly cleaned and then primed (for steel members) and coated with epoxy for bonding the FRP composite to the base material. For new bridge components fabricated in the shop, this method is sometimes used for complex or custom-sized deck panels.

Vacuum Assisted Resin-Transfer Molding

Vacuum assisted resin-transfer molding (VARTM) is used for large panels (such as decks) with a nearly solid cross-section. This procedure uses vacuum to infuse the fiber reinforcement with resin instead of manual labor. The advantages of VARTM are high fiber-resin ratios and remarkably quick fabrication times with the entire saturation procedure completed in just a few minutes. However, this procedure does not always work correctly and due to the high pressures, cannot be used with many filler materials as they would be crushed by the vacuum process.

VARTM must be performed in a controlled environment such as a fabrication shop.

Pultrusion

Pultrusion is ideal when FRP composite components require uniformity and consistency. Typically used for structural shapes such as boxes and I-beams, this method involves drawing a resin-fiber mixture through heated dies that cures the mixture immediately. Requiring almost no physical labor, pultrusion is very efficient for creating standard shapes and is cost efficient when producing large quantities. However, the main disadvantage to pultrusion is the ability to only produce long and narrow objects. FRP composite decks may be produced using pultruded elements such as box shapes, but must be bound together using an adhesive or bonding agent to achieve the desired width. Similar to VARTM, pultrusion must be performed using large machines in a fabrication shop.

Fiber Reinforced Concrete

FRC is constructed by mixing Portland cement and fiber (0.2 to 0.8 percent by volume) in a similar manner to conventional steel reinforced concrete (see Figure 6.6.13). The most common type of discontinuous fiber reinforcement is polypropylene, though organic timber fibers are currently being researched with promising results.

The purpose of the fiber is to minimize shrinkage cracking of fresh concrete and increase the impact strength of cured concrete. This type of concrete is used in bridge decks (refer to Topic 7.3 for more information).

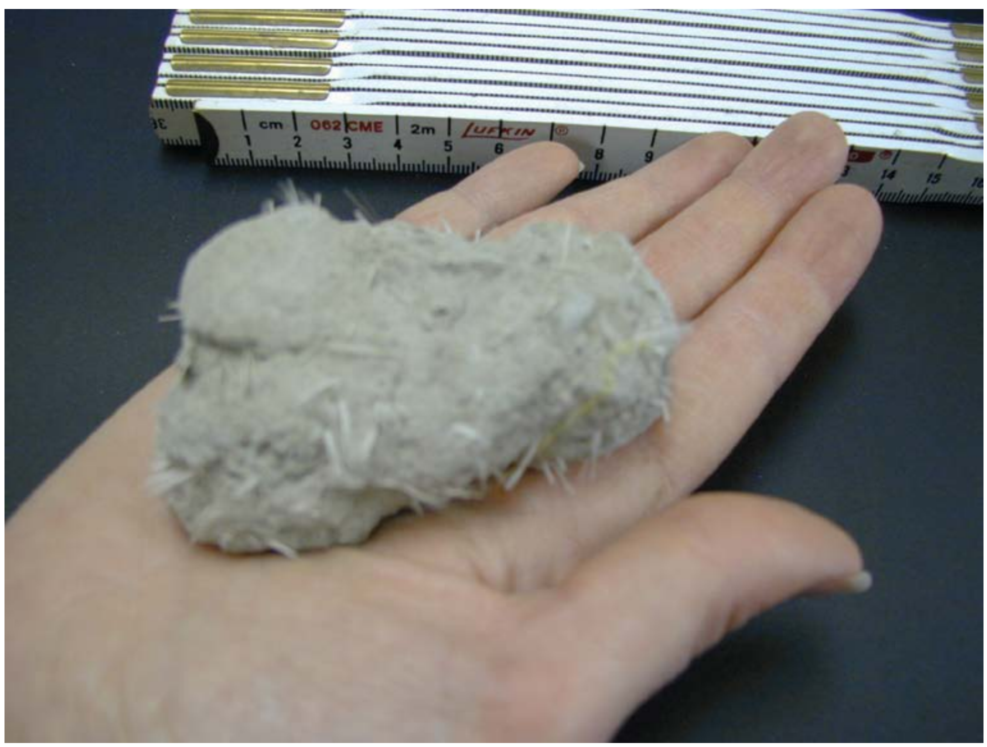

Figure 6.6.13 Fiber Reinforced Concrete (FRC)

6.6.4 Fiber Reinforced Polymer Deficiencies

In order to properly inspect FRP components, the inspector must be able to recognize possible types of deficiencies common to FRP composites. Some of the major forms of deficiencies in FRP composites include:

- Blistering
- Voids and Delaminations
- Discoloration
- Wrinkling
- Fiber exposure
- Scratches
- Cracking

Blistering

Blistering can be characterized as "surface bubbles" on the laminate surfaces or gelcoated surfaces due to trapped moisture in the laminate (see Figure 6.6.14). Although this phenomenon is somewhat common for thin-walled marine applications, FRP composite bridge members subjected to freeze-thaw cycles could experience this deficiency but would most likely not be affected structurally.

Figure 6.6.14 Blistering on a Laminated Surface

Voids and Delamination

Voids are debonded areas within the laminates. These regions will often be visible only after they have grown and resulted into a surface crack (see Figure 6.6.15). Delamination will often start at the initial site of a void, which can be detected with signal penetration equipment or by a tap test.

Figure 6.6.15 Voids Resulting in Surface Cracks

Discoloration

Discoloration of FRP components may be indicative of structural problems. Discoloration may result from:

- Chemical reactions including extensive UV radiation, heat or fire exposure.
- Crazing and whitening due to excessive strain of the material
- Subsurface voids resulting from improper wet-out or saturation procedures. This problem is more common for hand lay-up fabrication methods.
- Moisture infiltration of uncoated resin

Wrinkling

Wrinkling of the fabric is typically a result of excessive stretching during the wet-out process (see Figure 6.6.16). This defect is generally not a structural problem unless present at connectivity points or bonding regions.

Figure 6.6.16 Wrinkling of Fabric in Laminated Facesheet

Fiber Exposure

Fiber exposure is a structural deficiency that is typically a result from improper handling and erection methods (see Figure 6.6.17). Given the vulnerability of the fibers when exposed to moisture and contaminates, this deficiency could lead to significant damage if left untreated.

Figure 6.6.17 Fiber Exposure from Improper Handling and Erection Methods

Scratches

Although often incidental, scratches, if moderate to severe, may develop into cracks and pose a threat to the structural integrity of the surface and internal fibers. These deficiencies are often a product of improper handling, storage, erection, or tooling methods (see Figure 6.6.18).

Figure 6.6.18 Scratches on FRP Surface

Cracking

Cracks may result from impact with vehicles, debris, stones or may develop from another deficiency that has been left untreated. In some situations, areas with low concentrations of reinforcing fibers may exhibit false signs of impact cracks. Damage due to punching actions may also develop cracks and discoloration around the affected area (see Figure 6.6.19).

Cracks typically develop throughout the entire thickness of the laminate.

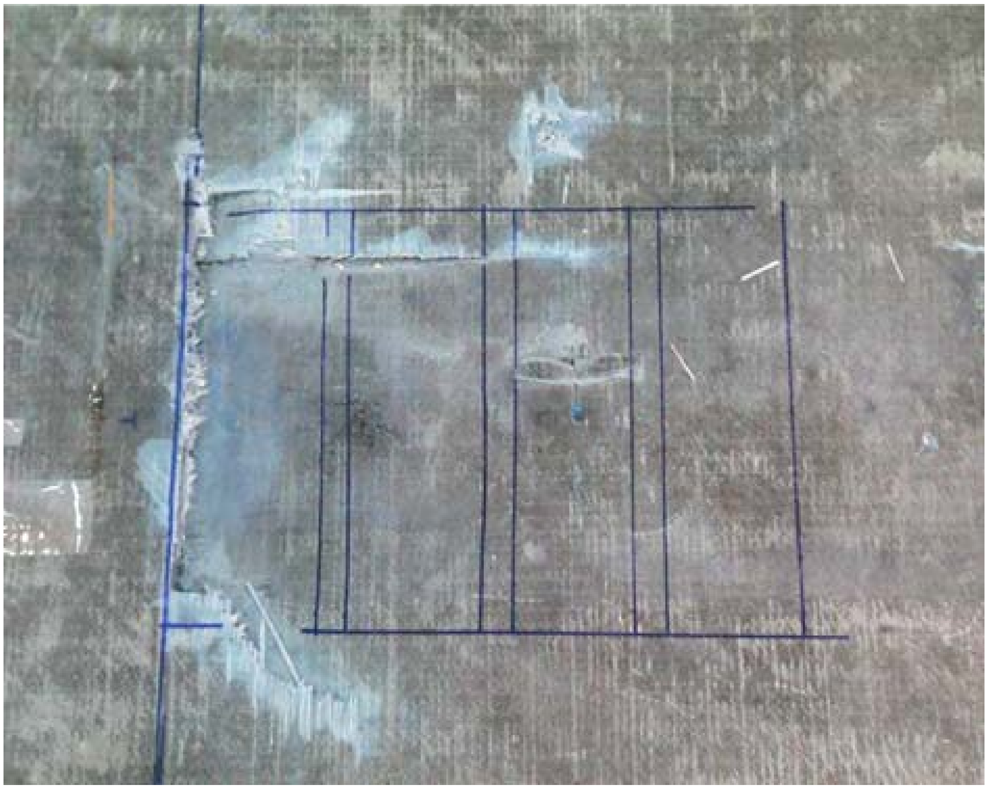

Figure 6.6.19 Cracks and Discoloration Around Punched Area

6.6.5 Inspection Methods for Fiber Reinforced Polymer

There are three basic methods used to inspect and evaluate FRP members. Depending on the type of inspection, the inspector may be required to use one or more of the methods. These methods include:

- Visual examination
- Physical examination
- Advanced inspection methods

Visual Examination

The visual examination of FRP composite members is the primary inspection method used by bridge inspectors for surface deficiencies. The following equipment is required when performing a visual assessment of FRP components:

- Flashlight
- Measuring tape
- Straight edge
- Markers
- Magnifying glass
- Inspection mirrors
- Feeler gages
- Geologist's pick

During an inspection, it may be helpful to incorporate a static or dynamic load (truck). This method is particularly useful when inspecting FRP decks (as described in Topic 7.3) to assist in detecting cracks and other deficiencies including vertical movement.

Physical Examination

Physical examinations of FRP are performed by sounding or tap testing. Analogous to concrete examinations using a chain drag, tap testing is a quick, inexpensive, and effective method for detecting areas of debonding or delamination in FRP.

This method of physical examination is typically performed by using a small hammer tap or large coin to measure the difference in frequency between sound and delaminated areas. Inspectors should listen for a clear sharp ringing sound to indicate well-laminated members and a dull thud to indicate delaminated members or hidden voids. It is also important to note that prior to performing tap testing, the inspector should review and be familiar with the geometry of the structure as changes in the structure's geometry can project different frequencies that may be otherwise incorrectly reported as a deficiency.

If the inspection is performed within a noisy environment, electronic units may be used to indicate suspect areas (see Figure 6.6.20). However, these units are typically not preferred over conventional methods due to the additional time required to perform an electronic tap test. The test is also ineffective for some deck sections such as pultruded deck sections or sections with varying thickness (see Topic 7.3 for more information).

Traditional and electronic tap testing does not require NDE certification and may be performed by a typical bridge inspector or engineer with very little training.

Figure 6.6.20 Electronic Tap Testing Device

Advanced Inspection Methods

If the extent of the FRP deficiency cannot be adequately determined by visual and/or physical examination methods described above, advanced inspection methods should be used. Examples of nondestructive evaluation methods are listed below:

- Thermal testing – thermal testing uses a heat source and imaging sensor to record the temperature gradient within the FRP composite material. This change in temperature identifies areas of delamination, impact, moisture, and voids (see Figure 6.6.21). Thermal testing requires moderate training to interpret the results, but does not require NDE certification. Despite the initial cost of a quality imaging system, thermal testing is considered to be one of the more favorable and practical advanced inspection methods for FRP.

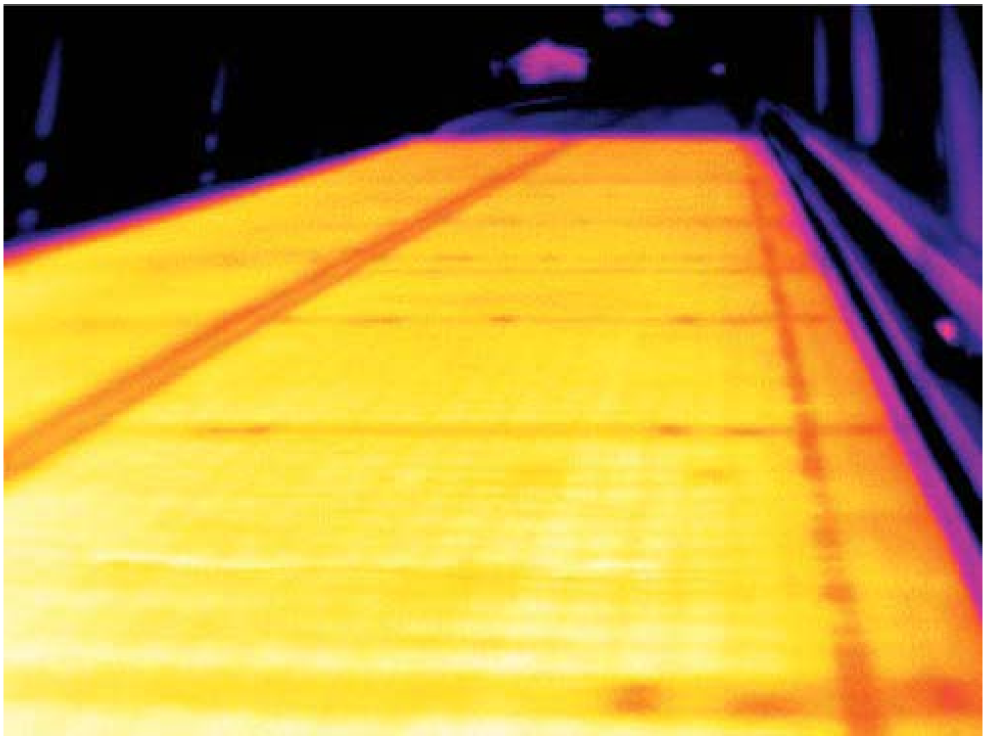

Figure 6.6.21 Thermographic Image of Bridge Deck

- Acoustic emission testing – acoustic emission testing is very useful for detecting areas containing deficiencies which can then be examined in more detail using other techniques. This method operates on stress waves being produced due to deformation, crack initiation, crack growth, breaks in reinforcing fibers, and delaminations (see Figure 6.6.22). Given the high level of experience and equipment required to perform acoustic testing, this type of NDE is normally performed by specialty technicians.

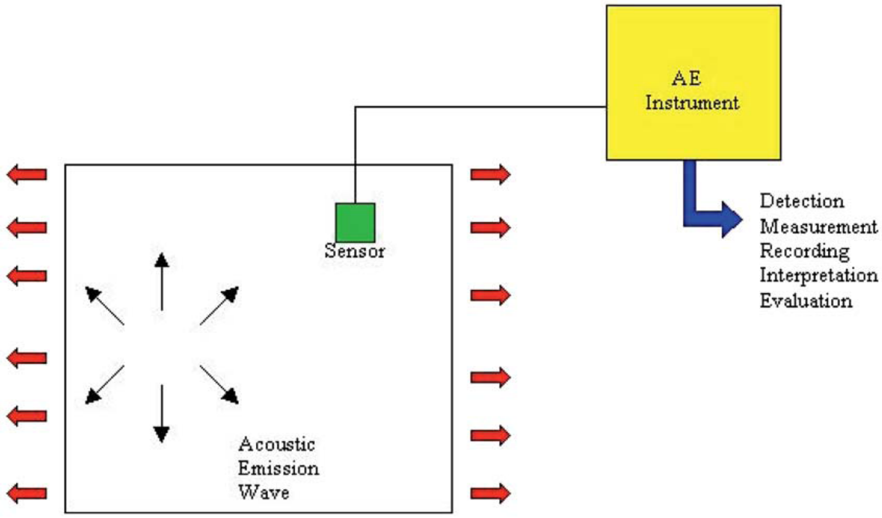

Figure 6.6.22 Acoustic Testing Technique

- Ultrasonic testing – ultrasonic testing sends high-frequency sound waves through the material. Defective material is detected from the deflected signal which can then be measured for magnitude. By knowing properties of the wave and material, the location of the deficiency can also be calculated. This NDE method is not effective for uneven surfaces and requires certification from the American Society of Nondestructive Testing (ASNT) to perform. Bridge inspectors already familiar and certified in ultrasonic testing for other materials can easily adapt for testing of FRP composite members.

- Laser-based ultrasound testing – as an alternative to ultrasonic testing, laser-based ultrasound testing uses one laser to generate sound waves and a second laser to detect the waves and subsequent deficiencies. Unfortunately this method is currently requires expensive portable equipment and is considered impractical for FRP inspections.
- Radiography – radiographic testing uses a source of radiation (X-ray or gamma ray) and radiographic film to record different levels of absorption as the rays pass through the specimen. This method can detect voids, resin variations, broken fibers, impact damage, cracks, and some delaminations. It is recommended that persons performing radiography be ASNT-certified. Radiography is dangerous due to radiation and often impractical since this method requires full access to both sides of the member.
- Reverse-geometry digital X-ray – Similar to radiography, this NDE method is safer than conventional radiography, does not require radiographic film, and can produce three-dimensional results unlike radiography which can only construct planar models of the deficiencies. However, reverse-geometry digital X-ray systems are very expensive and require very elaborate equipment and the associated knowledge to operate.
- Modal analysis – modal analysis considers the structural dynamics, frequency, and mode shapes of the system. This method also requires pre-existing knowledge of the system to make a baseline reference or an elaborate approximation of the structure's as-built condition. Modal analyses require highly trained personnel and expensive equipment.
- Load testing – load testing is performed using external sensors such as strain gages, accelerometers, and displacement sensors to evaluation the condition of the structure. As with modal analysis, load testing requires knowledge of structure's original condition as well as well-trained personnel to interpret the data. In addition, load testing requires significant time in the field to position the truck and the appropriate collective information.

6.6.6 Inspection Locations for Fiber Reinforced Polymer

Special attention should be given to FRP composite members at the following locations:

- Splice joints – inspect for delaminations, cracks, and other deficiencies
- Butt joints – inspect for delaminations, cracks, and other deficiencies
- High stress areas near connections – examine for cracking and discoloration around the bolts and clips
- Underneath deck near beams or supports – look for discoloration and

signs of drainage leakage

- Connections – all connections should be inspected for tightness, especially clip-type connections
- Deck-girder interfaces – measure for gaps between the deck and girders or supporting members
- Areas of maximum moment – look for distress in beams and decks, especially in the compression faces of decks utilizing composite action between the beams and deck
- Bearing areas – inspect for crushing of the FRP members including punching action in deck sections
- Shear areas – areas prone to high shear stresses should be checked for cracks and delaminations

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Chapter 7

Inspection and Evaluation of Bridge Decks and Areas Adjacent to Bridge Decks

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Chapter 7
Inspection and Evaluation of Bridge Decks and Areas Adjacent to Bridge Decks

Topic 7.1 Timber Decks

7.1.1 Introduction

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.

Like any investment, a timber bridge must be inspected and maintained on a regular basis to maximize the investment. The fact is that a poor design, poor construction, and poor management practices can be major factors in the degradation of a timber structure. Over the life of a timber bridge, deficiency can be minimized, by identifying and recording information on the condition and performance of the structure. With such information, timely maintenance operations can be undertaken to correct situations that could otherwise lead to extensive repair and even replacement. Bridge inspectors have the difficult task of accurately assessing the condition of an existing timber member, due to the fact that most decay occurs on the inside of a timber member. Timber inspection is a learned process that requires some knowledge and understanding of wood pathology, wood technology, and timber engineering.

7.1.2 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 considered non-composite because of the inefficient shear transfer through the attachment devices between the deck and superstructure. 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 boards laid transversely across the bridge (see Figure 7.1.1). The planks are individually attached to the superstructure using spikes or bolt clamps, depending on the superstructure material. It is common for plank decks to have 2-inch depth timbers nailed longitudinally on top of the planks to distribute load and retain the bituminous wearing surface.

Figure 7.1.1 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 7.1.2). On timber beams, each lamination is toenailed to the beam. On steel beams, clamp bolts are used as required. In either case, laminates span across the beams and are perpendicular to the roadway centerline.

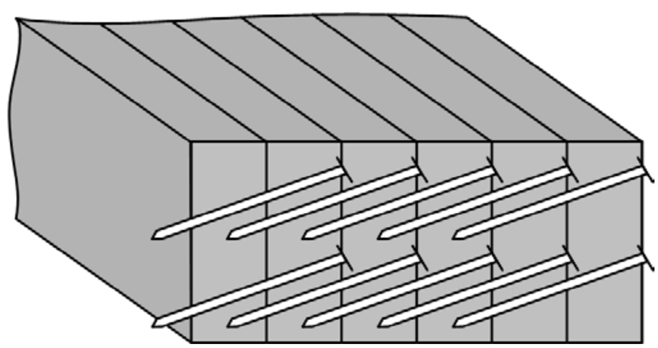

Figure 7.1.2 Section of a Nailed Laminated Deck

Glued-laminated Deck Panels

Glue-laminated (Glulam) deck panels are an engineered wood product in which pieces of sawn lumber are attached together with waterproof glue adhesives. Glulam deck panels come in sizes usually 4 feet wide. The panels can be laid transverse to the traffic depending on superstructure orientation. 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 7.1.3).

The nailing method is generally not preferred due to the possibility of the nails being pried loose by the vehicle traffic vibrations and deflections.

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 superstructure members, 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 beams or stringers and then using bolts to attach the clip angles to the deck.

Nailers are planks that run along the top of steel superstructure flanges. This technique involves the bolting of the nailers to the flanges and nailing the timber planks to the nailers. This prevents the costly bolting of all planks to the steel superstructure.

Figure 7.1.3 Glued-laminated Deck Panels Attached to Superstructure

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 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 7.1.4).

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 7.1.4 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 fabricated by gluing together thin sheets of rotary-peeled wood veneer with a waterproof adhesive. Parallel strand lumber is fabricated 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 (see Figure 7.1.5).

See Topic 6.1 for various T-beam and box shape configurations used for Structure Composite Lumber Decks.

Figure 7.1.5 Structural Composite Lumber Deck Using Box Sections

7.1.3 Wearing Surfaces

The wearing surface of a timber deck is constructed of timber, bituminous materials, or concrete. 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, or parallel with traffic, only in the wheel paths where the vehicles ride (see Figure 7.1.6).

Figure 7.1.6 Timber Wearing Surface on a Timber Plank Deck

Bituminous

Bituminous or asphalt wearing surfaces generally utilize a coarse aggregate mix. 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 because 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.

7.1.4 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 has to be properly prepared. See Topic 6.1.6 for detailed information on protective systems.

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 will 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, pentachlorophenol, copper naphthenate and oxine copper. Coal-tar creosote is no longer used to health concerns; pentachlorophenol is used as an above-ground decay inhibitor; copper napthenate can be used for above-ground, ground contact and only freshwater applications since it is not suited for salt water applications; and oxine copper is used for above-ground applications once dissolved in a heavy oil.

Chromated copper arsenate (CCA), ammonical copper zinc arsenate (ACZA), alkalkine copper quaternary (ACQ) and copper azole are common water-based preservatives. CCA was the most popular preservative used from the late 1970s until 2004; ACQ and Copper azole have both been recently developed while ACZA is no longer available in the United States.

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.

7.1.5 Overview of Common Deficiencies

A prepared bridge inspector will know what to look for prior to the inspection. The following is a list of common deficiencies that may be encountered when inspecting timber bridge decks. Refer to Topic 6.1 for a detailed description of these common deficiencies:

- Inherent defects: checks, splits, shakes, knots
- Fungi
- Insects
- Marine borers
- Chemical attack
- Delaminations
- Loose connections
- Surface depressions
- Fire
- Impact or collision
- Wear, abrasion and mechanical wear
- Overstress
- Weathering or warping
- Protective coating failure

7.1.6 Inspection Methods and Locations

Methods

Visual

The inspection of timber decks for deficiency and decay is primarily a visual activity. All exposed surfaces of the timber decks will receive a close visual inspection.

Physical

However, physical examinations will also be used for suspect areas. The most common physical inspection techniques for timber include sounding, probing, boring or drilling, core sampling, and electrical testing. An inspection hammer will 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 7.1.7). 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 2 inches thick, suspect planks will be drilled to determine the extent of decay. If decks are drilled, a protectant will be applied and the hole will be plugged with a wooden dowel.

Figure 7.1.7 Inspector Probing Timber with a Pick at Reflective Cracks in the Asphalt Wearing Surface

Advanced Inspection Methods

Several advanced methods are available for timber inspection. Nondestructive methods, described in Topic 15.1.2, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 15.1.3, include:

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

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 will be inspected. In all instances, it is helpful to have the previous inspection report so that the progression of any deficiency 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 7.1.8)
- **Bearing and shear areas** where the timber deck contacts the supporting superstructure – inspect for crushing, decay, and fastener deficiencies (see Figure 7.1.9)
- **Tension areas** between the support points – investigate for flexure damage, such as splitting, sagging, and cracks
- **Areas exposed to drainage** – check for decay, particularly in areas exposed to drainage (see Figure 7.1.10)
- **Outside edges of deck** – inspect for decay
- **Connections –** note any looseness that may have developed from inadequate nailing or bolting, or where the spikes have worked loose. Observation under passing traffic will reveal looseness or excessive deflection in the members
- **Nailed laminated areas –** 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 damage of the deck. Check for loose, corroded or damaged nails
- **Prestressing anchorages** – check for tightness, corrosion, crushing, and decay (see Figure 7.1.11)
- **Fire damage –** check for any section loss or member damage caused by fire

Figure 7.1.8 Wear and Weathering on a Timber Deck

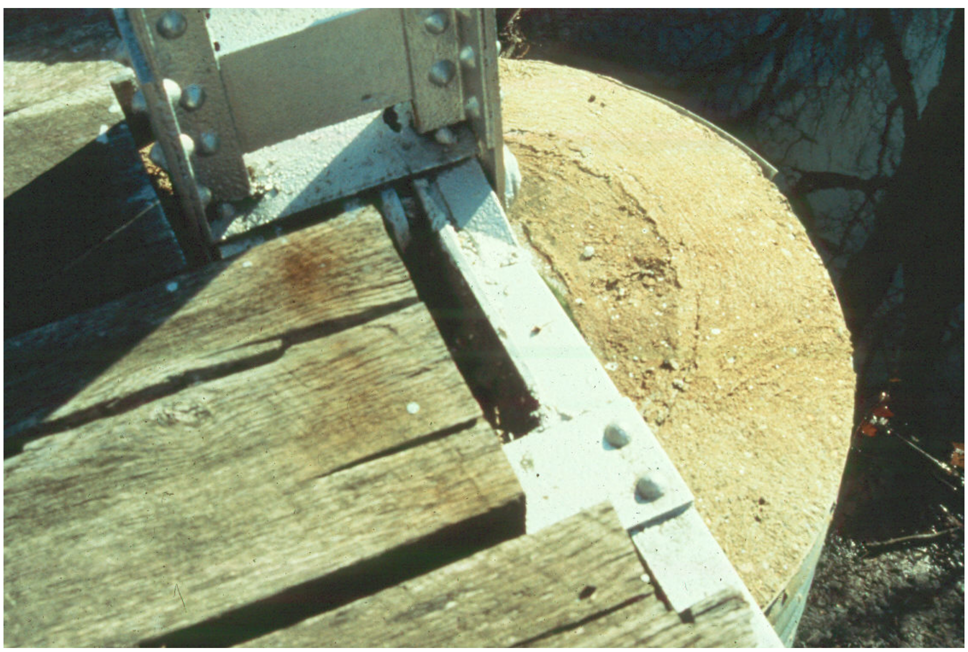

Figure 7.1.9 Bearing and Shear Area on a Timber Deck

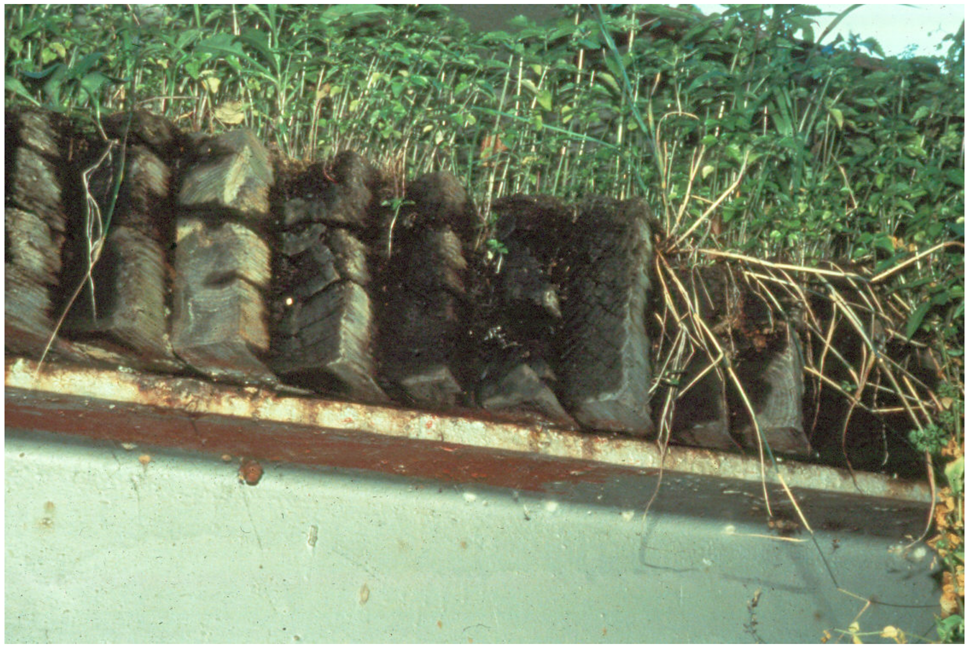

Figure 7.1.10 Edge of Deck Exposed to Drainage, Resulting in Plant Growth

Figure 7.1.11 Broken Prestressing Anchors

7.1.7 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the deck. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 58) for additional details about the NBI component condition rating guidelines.

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

Element Level Condition State Assessment

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

NBE No.	**Description**
31	Timber Deck
54	Timber Slab

BME No.	**Description**
510	Wearing Surfaces
520	Deck/Slab Protection Systems

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

For the purposes of this manual, a deck is supported by a superstructure and a slab is supported by substructure units.

The following Defect Flags are applicable in the evaluation of timber decks:

Deflect Flag No.	**Description**
366	Deck Traffic Impact

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

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Chapter 7

Inspection and Evaluation of Bridge Decks and Areas Adjacent to Bridge Decks

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Topic 7.2 Concrete Decks

7.2.1 Introduction

The most common bridge deck material is concrete. The physical properties of concrete permit placing in various shapes and sizes, providing the bridge designer and the bridge builder with a variety of construction methods. This topic presents various aspects of concrete bridge decks and related bridge inspection issues.

7.2.2 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:

- Conventionally reinforced cast-in-place (CIP)
- Precast conventionally reinforced
- Precast prestressed
- Prestressed deck panels with CIP topping

Conventionally Reinforced Cast-in-Place

Concrete decks that are placed at the bridge site are referred to as "cast-in-place" (CIP) decks. Forms are used to contain conventional 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 conventional reinforcement in the proper location during construction. 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 taken away from the deck after the concrete has cured.

Stay-in-place (SIP) forms are corrugated metal sheets permanently installed between the supporting superstructure members. After the concrete has cured, these forms, as the name indicates, remain in place as permanent, nonworking members of the bridge (see Figure 7.2.1).

Figure 7.2.1 CIP Concrete Deck with Stay-in-Place Forms

Precast Conventionally Reinforced

Precast deck panels are conventionally reinforced concrete panels that are cast and cured somewhere other than on the bridge site. Proper deck elevations are 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 connect the deck panels to the superstructure. 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 decks 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 deck (pretensioned) or after the deck is cured (post-tensioned). The tendons are held in position until the deck has sufficiently cured. This creates compressive forces in the deck, which reduce the amount of tension cracking in the cured concrete.

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 7.2.2). A cast-in-place layer, which may be reinforced, is then placed which engages both the supporting superstructure members and the precast deck units. The CIP layer provides a jointless top surface for the deck which results in a smoother ride for motorist. After the cast-in-place layer has cured, composite action is achieved with the shear connectors and superstructure.

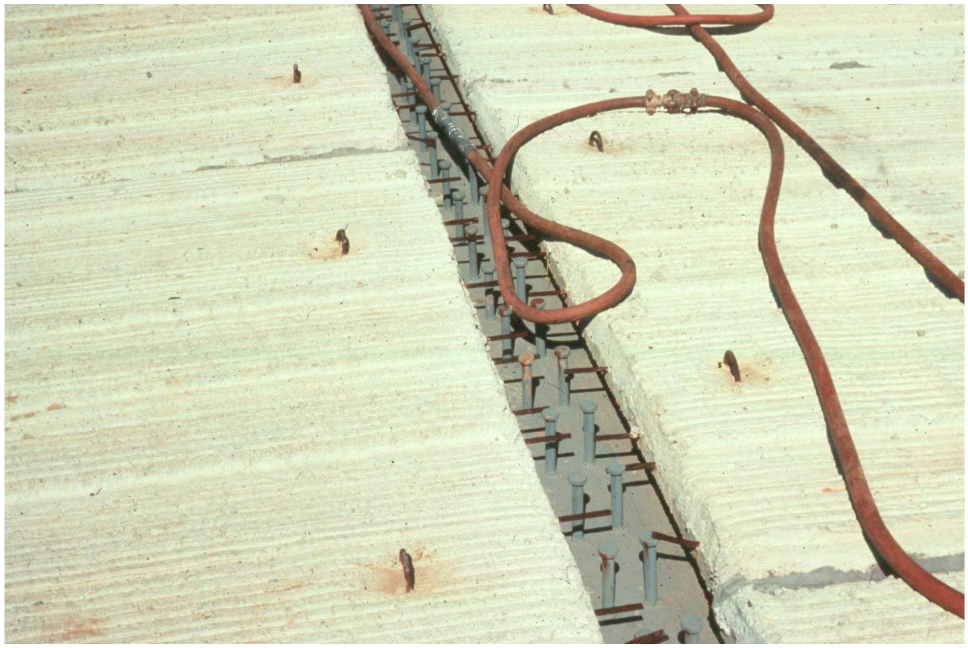

Figure 7.2.2 Precast Deck Panels with Lifting Lugs Evident and Top Beam Flange Exposed (Prior to CIP Topping)

Composite Action

A concrete deck is generally required when composite action is desired in the superstructure (refer to Topic 5.2.32). 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 superstructure 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 7.2.3 and 7.2.4). A precast deck can also develop composite action through grout pockets, which engage shear connectors (see Figure 7.2.2). Some examples of shear connectors are studs, spirals, channels, or stirrups. Shear connectors are generally welded to the top flange of steel superstructure members. In prestressed concrete beams, shear connectors are extended portions of stirrups which protrude beyond the top of the beam. 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 Action

A non-composite concrete deck is not mechanically attached to the superstructure and does not contribute to the capacity of the superstructure.

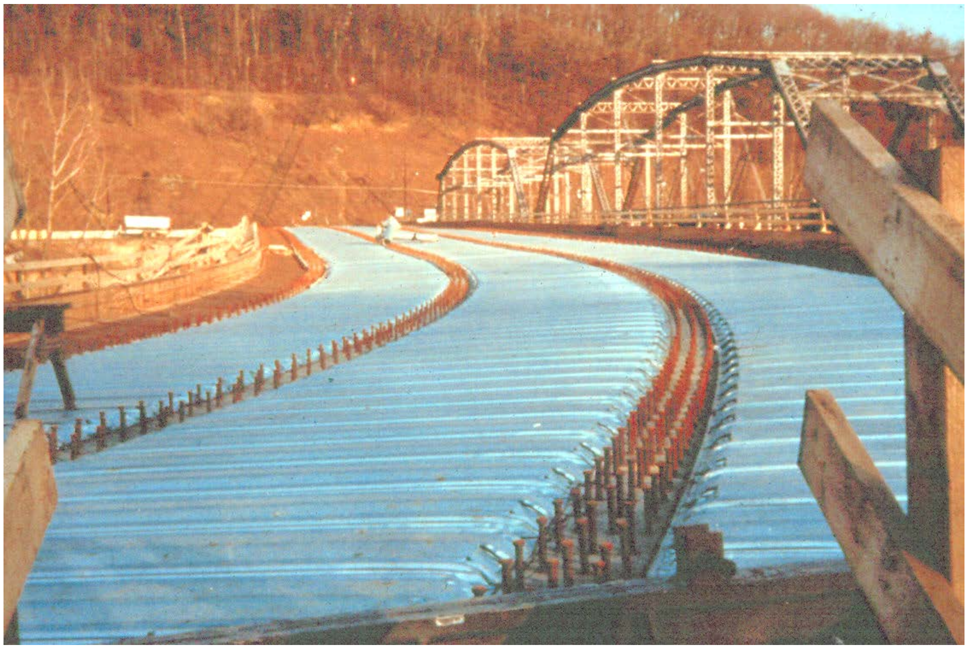

Figure 7.2.3 Shear Connectors Welded to the Top Flange of a Steel Girder for Composite Deck

Figure 7.2.4 Prestressed Concrete Beams with Shear Connectors Protruding for Composite Deck

Steel Reinforcement

Because concrete has relatively little tensile strength, conventional steel reinforcement is used to resist the tensile stresses in the deck. When conventional reinforcement was first used for bridge decks, it was either round or square steel rods with a smooth finish and had a tendency to debond with the surrounding concrete when a tension force was applied. Today, the most common conventional 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 reinforcement 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. 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. 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 conventional 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 steel rebars against corrosion. Less common methods of protection include galvanizing and use of stainless steel. Refer to Topic 6.2.4 for detailed explanations on various reinforcement types.

Primary reinforcement carries the tensile stress in a concrete deck and is located on both the top and bottom of the deck. Decks are designed with thickness that shear reinforcement is not normally required. Older, thinner decks utilized bent tensile reinforcement to act as shear reinforcement in areas close to superstructure support. These bent bars are sometimes referred to as ‘crank’ bars. 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 in the composite superstructure.

It is important 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 7.2.8). 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 top and bottom concrete surface than secondary reinforcement.

Figure 7.2.5 Spall 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 top cover is generally 2 to 2-1/2 inches minimum for cast-in-place decks without a wearing surface, and 1 inch minimum for precast decks with a separate wearing surface. Refer to bridge plans, standards or actual field measurements to determine exact location of reinforcement bars.

7.2.3 Wearing Surfaces

Wearing surfaces are placed on top of the deck and 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 bituminous 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, typically adding an extra 1/2 to 1 inch of thickness to the deck. When the wearing surface has deteriorated to the extent that rebar protection 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 or 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 1¼ inches thick, but now a 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 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 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.

Lightweight concrete (LWC) overlays use concrete with lightweight aggregates and a higher entrained air content. This produces an overlay of approximately 80 to 100 pcf compared to 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, fiberglass, 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 propagating through the overlay. This type of overlay is gaining acceptance but is still in the research stage.

Bituminous

The most common overlay material for concrete decks is bituminous concrete (commonly referred to as ‘asphalt’). Bituminous overlays generally range from 1 ½ inch up to 3 inches thick, depending on the severity of the repair and the load capacity of the superstructure. When bituminous 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 bituminous layer. Not all attempts at providing a waterproof membrane are successful.

Epoxy Polymers

Epoxy polymer overlays on concrete decks help prevent the infusion of the chloride ions and can help provide skid resistance and protected system for 15 to 30 years, depending on the volume of traffic.

7.2.4

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 conventional steel and prestressed 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 bituminous overlay.

Epoxy Coated Reinforcement Bars

Conventional steel reinforcement corrosion causes detrimental effects on concrete decks. An epoxy coating is often used on all conventional steel deck reinforcement to prevent 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 conventional steel reinforcement is by galvanizing the steel. Galvanizing slows down the corrosion process and lengthens the life of the conventional reinforced concrete deck. Galvanizing is achieved by coating the bare conventional 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. Solid stainless steel reinforcement bars can be used due to its corrosion resistance being greater than conventional reinforcement with an estimated service life of 100 years. Stainless steel coating can be used on conventional reinforcement to which will protect the reinforcement from water and air and quickly reform if the surface is scratched.

Fiberglass Reinforced Polymer (FRP) bars

Fiberglass reinforced polymer (FRP) bars for concrete reinforcement has an advantage over conventional reinforcement due its resistance to corrosion. They are also lightweight, weighing about one-quarter the weight of an equivalent size steel bar. Corrosion resistance and weight are offset by lower allowable tensile strengths.

Cathodic Protection of Reinforcement Bars

Cathodic protection is sometimes used on decks with black bare steel reinforcement (not epoxy coated). Conventional steel reinforcement corrosion can also be slowed down by cathodic protection. Corrosion of conventional 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 (see Figure 7.2.6).

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 (see figure 7.2.6). Thus, the artificial anode mesh or coating is also spared from corrosion.

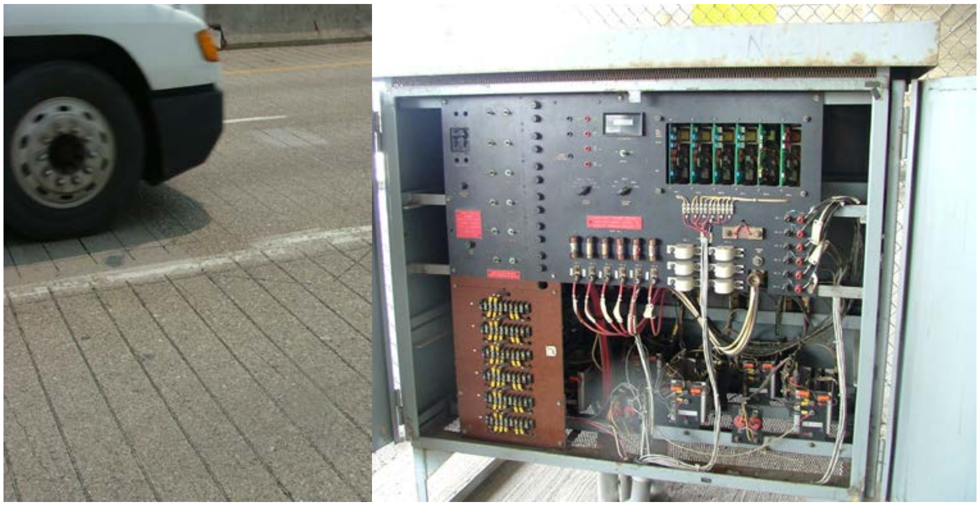

Figure 7.2.6 Cathodic Protection: Deck Wires Connected to Direct Current Rectifier

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.
- 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. This membrane type is applied through 'spraying or painting' the material to the deck.

A layer of bituminous base and wearing course is then applied over the membrane for both these methods. These systems are used to retard reflective cracking and provide waterproofing.

7.2.5 Overview of Common Deficiencies

Common concrete deck deficiencies are listed below. Refer to Topic 6.2 for a detailed description of these deficiencies:

- Cracking (flexure, shear, temperature, shrinkage, mass concrete)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Surface breakdown
- Pore pressure
- Efflorescence
- Alkali Silica Reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Reinforcing steel corrosion
- Prestressed concrete deterioration

7.2.6 Inspection Methods and Locations

Methods

Visual

The inspection of concrete decks for surface cracks, spalls, and other deficiencies is primarily a visual activity. All surfaces of the concrete deck will receive a close visual inspection.

Physical

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.

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. It will be dragged across a deck and make a note of the resonating sounds. A chain drag can usually cover about a 3-feet wide section of deck at a time (see Figure 7.2.7). Evaluate suspect areas with a hammer to determine the exact dimensions of the delaminated area.

Figure 7.2.7 Sounding for Delaminated Areas of Concrete

Many of the problems associated with concrete bridge decks are caused by corrosion of the reinforcement. When the deficiency 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 Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

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

Other methods, described in Topic 15.2.3, include:

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

If it is deemed necessary, core samples can be taken from the deck and sent to a laboratory to determine the extent of any chloride contamination.

Locations

Both the top and bottom surfaces of concrete decks will be inspected for deficiencies listed in Topic 7.2.5. In all instances, it is helpful to have the previous inspection report available so that the progression of any deficiency can be noted. This provides a more meaningful inspection by helping the inspector determine the rate of deficiency.

For concrete deck inspections, special attention will 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. Verify that repairs are acting as intended.
- **Areas exposed to drainage** – investigate for ponding water, scaling, delamination, and spalls.
- **Bearing and shear areas** where the concrete deck is supported – check for cracks, spalls and crushing near supports.
- **Shear key joints** between precast deck panels – inspect for leaking joints, cracks, and other signs of independent panel action.
- **Anchorage zones** of precast deck tie rods – check for deteriorating grout pockets or loose lock-off devices. If a previous inspection report is available, this will be used to see if the progression of any deficiency can be noted.
- **Top of the deck** over the supports – examine for flexure cracks which would be perpendicular to the primary tension reinforcement.
- **Bottom of the deck** between the supports – check for flexure cracks which would be perpendicular to the primary tension reinforcement (see Figure 7.2.8).
- **Bituminous overlays** – if present, they will be inspected. Cracks, delaminations, and spalls are to be noted. Often water penetrates overlays and then penetrates into the structural deck. Bituminous overlays prevent visual inspection of the top surface of the deck. The wearing surface does not affect the evaluation of the structural 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 7.2.9).
- **Cathodic protection** – during the bridge inspection, check that all visible electrical connections and wiring from the rectifier to the concrete structure are intact. Check the rectifiers after an electrical storm. Nearby lightning has been known to 'trip the circuits' and to inactivate the system. 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.
- **Areas previously repaired –** investigate for deterioration of any patches that were previous noted. Determine if the repairs are in place, and they are functioning properly.
- **Areas of closure pours** – investigate for signs of any delamination or spalling around the area of a closure pour
- **Adjacent to joints -** investigate for signs of delamination or spalling in general area around the joint.
- **Fire damage** – check for any damage caused by fire

Figure 7.2.8 Underside View of Longitudinal Deck Crack

Figure 7.2.9 Deteriorated Stay-in-Place Form

7.2.7 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the deck. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 58) for additional details about the NBI component condition rating guidelines.

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

Element Level Condition State Assessment

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

NBE No.	Description
12	Reinforced Concrete Deck
38	Reinforced Concrete Slab

BME No.	Description
510	Wearing Surfaces
520	Deck/Slab Protection
521	Concrete Protective Coating

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

For the purpose of this manual, a deck is supported by a superstructure, and a slab is supported by substructure units.

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

Defect Flag No.	Description
358	Concrete Cracking
359	Concrete Efflorescence
366	Deck Traffic Impact

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

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Chapter 7

Inspection and Evaluation of Bridge Decks and Areas Adjacent to Bridge Decks

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Topic 7.3 Fiber Reinforced Polymer (FRP) Decks

7.3.1 Introduction

Fiber Reinforced Polymer (FRP) is a modern bridge material that is becoming increasingly popular throughout the transportation community. First used in the United States in the early 1990s, FRP has been explored both in the repair and retrofit of existing structures as well as new bridge construction.

Figure 7.3.1 Fiber Reinforced Polymer (FRP) Deck

7.3.2 Design Characteristics

Modern FRP composite decks are typically made of pultruded sections (e.g., honeycomb shaped, trapezoidal, or double-web I-beams). Slabs are often made using a vacuum assisted process.

There are three types of FRP composite decks:

- Honeycomb sandwich
- Solid core sandwich
- Hollow core sandwich

Honeycomb Sandwich

Honeycomb sandwich construction will provide considerable flexibility in the depth of the deck. However, the hand lay-up process will require a large amount of attention to quality control when bonding the top and bottom facesheets to the core (see Figure 7.3.2).

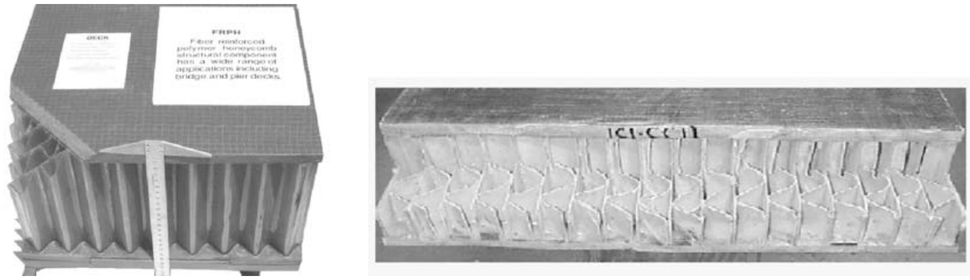

Figure 7.3.2 Honeycomb sandwich configuration (Photograph from NCHRP Report 564 – Field Inspection of In-Service FRP Bridge Decks)

Solid Core Sandwich

Solid core sandwich decks contain foam or other fillers at the core. This type of decks is manufactured by using a process called Vacuum-Assisted Resin-Transfer Molding (VARTM). (see Figure 7.3.3)

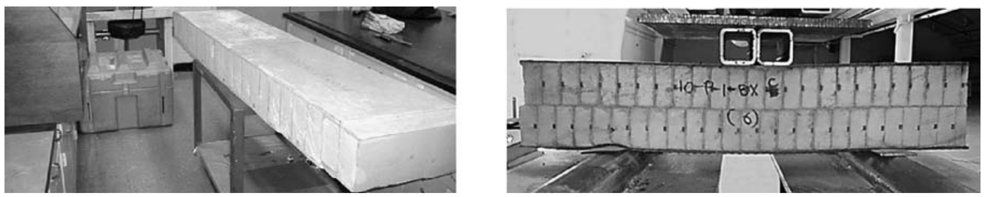

Figure 7.3.3 Solid core sandwich configuration (Photograph from NCHRP Report 564 – Field Inspection of In-Service FRP Bridge Decks)

Hollow Core Sandwich

Hollow core sandwich decks consist of deck sections that contain pultruded shapes that are fabricated together. (see Figure 7.3.4)

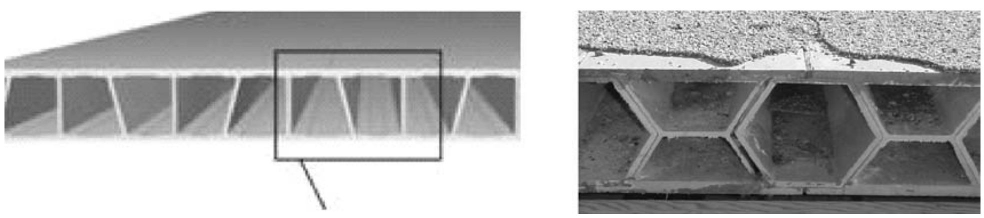

Figure 7.3.4 Hollow core sandwich configuration (Photograph from NCHRP Report 564 – Field Inspection of In-Service FRP Bridge Decks)

7.3.3 Wearing Surfaces

FRP decks require an overlay due to the low skid resistance of the materials. For both deck and slabs, Thin polymer-concrete overlays are often used for the wearing surface. Bituminous overlays have also been used.

Bituminous

The most common overlay material for FRP decks is bituminous concrete commonly referred to as asphalt. Asphalt overlays generally range from 1 ½ inch up to 3 inches thick. When asphalt is placed on FRP, a waterproof membrane may be applied first to protect the FRP from the adverse effects of water borne deicing chemicals, which pass through the permeable bituminous concrete layer. Not all attempts at providing a waterproof membrane are completely successful.

Epoxy Polymers

Epoxy polymer overlays have been used to protect FRP decks. They help prevent the infusion of the chloride ions and can help provide skid resistance for 15 to 30 years, depending on the volume of traffic.

7.3.4 Overview of Common Deficiencies

Common FRP deck deficiencies are listed below. Refer to Topic 6.6 for a detailed description of these deficiencies:

- Blistering
- Voids and Delaminations
- Discoloration
- Wrinkling
- Fiber exposure
- Scratches
- Cracking

7.3.5 Inspection Methods and Locations

Methods

Visual

The visual inspection of FRP decks for surface deficiencies is the primary inspection method. Even though it may be easy to detect blistering and debonding, it is often helpful to incorporate a static or dynamic load (e.g. a truck) to assist in detecting a crack or any vertical deck movement while performing a visual inspection. (see Figure 7.3.5)

Figure 7.3.5 Use of Truck for Visual Inspection of FRP Deck

Physical

Tap testing is the most common method for visual inspections for fiber reinforced polymers. This method traditionally uses large coins or hammer taps to detect changes in frequency associated with areas of delamination or debonding.

If a physical inspection is performed in a noisy area, an electronic tapping device may be used (see Figure 7.3.6). However, the traditional tap test is preferred over the electronic method due to less time required to perform the traditional test and the ineffectiveness of an electronic tap test for certain deck sections, such as sections with varying thicknesses.

Figure 7.3.6 Electronic Tap Testing Device

Advanced Inspection Methods

Several advanced methods are available for FRP inspection. Nondestructive methods, described in Topic 6.6.5, include:

- Thermal testing
- Acoustic emission testing
- Ultrasonic testing
- Laser based ultrasound testing
- Radiography
- Reverse-geometry digital X-ray
- Modal analysis
- Load testing

Locations

Both the top and bottom surfaces of FRP decks should be inspected for any blistering, delaminations, discoloration, wrinkling, fiber exposure, scratches or cracking. In all instances, it is helpful to have the previous inspection report available so that the progression of any deficiency can be noted. This provides a more meaningful inspection. Refer to Topic 6.6 for a detailed description of FRP deficiencies.

For FRP deck inspections, special attention should be given to the following locations:

- **Deck panel splice joints** – check for reflective cracking or oozing of joint material which may indicate movement or improper fitment between panels (see Figure 7.3.7)
- **Deck panel butt joints** – where joints are left exposed on the deck underside, measure the gap between panels
- **Vicinity of joints** – investigate for signs of delamination or spalling in general area around the joint. Tap tests should be performed to detect possible delamination
- **Areas exposed to traffic** – examine for surface texture and wheel ruts due to wear
- **Areas exposed to drainage** – investigate for ponding water and delamination
- **Top of deck** – at the expansion joints, check for signs of buckling, misalignment, differential vertical or horizontal movement
- **Underside of deck** – near support beams or abutments, inspect for discoloration, signs of flow, cracks, or other signs of distress (see Figure 7.3.8)
- **Haunch areas** – inspect for separation between deck and haunch or supporting superstructure component and measure distance. Also note any cracking of haunch grout material
- **Deck support areas** – perform tap tests near supports to check for delamination
- **Connections** – check all clip-type connections (see Figure 7.3.9) for tightness, soundness, scratches, abrasion, signs of movement, or any cracks in FRP from bearing against bolt for clip connection

Figure 7.3.7 Deck expansion joint

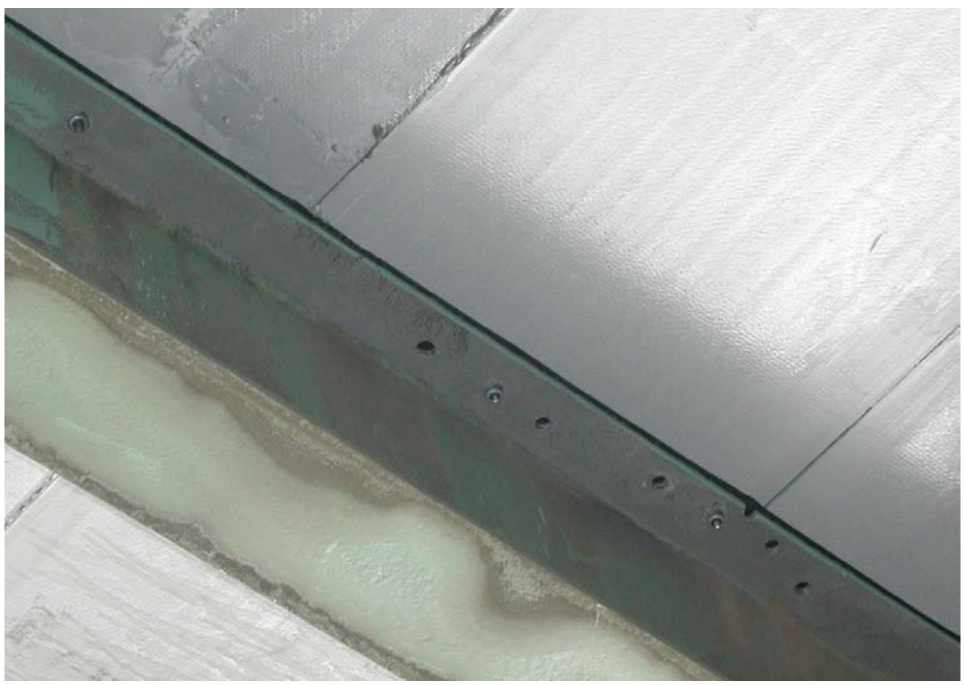

Figure 7.3.8 FRP Deck underside near superstructure beam

Figure 7.3.9 Clip-type connection Between FRP Deck and Steel Superstructure

7.3.6 Evaluation

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

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the deck. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 58) for additional details about the NBI component condition rating guidelines. Since the FHWA *Coding Guide* was published in 1995 when FRP was not prevalent, NCHRP has provided a supplement to the NBI component condition rating guidelines (see Figure 7.3.10).

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

Element Level Condition State Assessment

In an element level condition state assessment of a fiber reinforced polymer deck, there are currently no AASHTO National Bridge Elements (NBEs) for FRP decks.

Possible AASHTO Bridge Management Elements (BME) are:

BME No.	Description
510	Wearing Surface
520	Deck/Slab Protective Systems

The unit quantity for these elements is square feet. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

For the purpose of this manual, a deck is supported by a superstructure, and a slab is supported by substructure units.

The following Defect Flags are applicable for fiber reinforced polymer decks:

Defect Flag No.	**Description**
366	Deck Traffic Impact

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

Rating Code	**Condition**	**Description**
9	Excellent	Excellent condition, typically new construction.
8	Very Good	No significant problems noted.
7	Good	Minor surface damage in the form of hairline cracks in resin and scratches with no delamination evident on the deck surfaces or underneath.
6	Satisfactory	Minor damages in the form of shallow cracks in resin, scratches, blistering, abrasion and small delaminations over less than 2% of surface area total. Fibers are not exposed, ruptured, or buckled at the surface damage locations. Delamination smaller in every dimension than 4 in. and away from structural details or located such that structural function will not be impaired.
5	Fair	Damage in the form of shallow cracks in resin, scratches, blistering, abrasion, and small delamination extends over 2% to 10% of surface area total. Fibers exposed but not ruptured, buckled, or debonded at the surface damage locations. Delamination smaller in every dimension than 8 in. and located away from structural details or located not to have structural effects. Deck will function as designed.
4	Poor	Surface damage in the form of cracks in resin, scratches, blistering, abrasion, and delamination extends over 10% to 25% of area total. Fibers in the cracks exposed but not debonded, buckled, or ruptured at the surface damage locations. Delamination smaller in every dimension than 8 in. but near structural details or located to have structural effects. Deck will function as designed, but functionality may be impaired without repairs.
3	Serious	Surface damage in the form of deep cracks in resin, scratches, blistering, abrasion, and delamination extends over more than 25% of area total. Fibers are visibly exposed and debonded, but not ruptured or buckled at the surface damage locations. Delamination smaller in every dimension than 14 in. Structural analysis may be necessary to determine whether the deck can continue to function without restricted loading.
2	Critical	Fibers are exposed, debonded, and ruptured, or buckled at the surface damage locations. Delamination larger in any dimension than 24 in. Unless closely monitored or posted for reduced loads, closing the bridge may be necessary until corrective action is taken.
1	Imminent Failure	Major deterioration or damage present; large delaminations, cracks or voids, punctures, major fiber rupture, or buckling through cracks perpendicular to the FRP panel span, sag, or dislocation visible; large, and inconsistent deflections under traffic observed. Bridge is closed to traffic but corrective action may put back in service.
0	Failed	Out of service - beyond corrective action / deck must be replaced.

Figure 7.3.10 Condition rating of FRP deck structure (Source: NCHRP Report 564: *Field Inspection of In-Service FRP Bridge Decks: Inspection Manual*: Table 7.1.2-1)

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Chapter 7

Inspection and Evaluation of Bridge Decks and Areas Adjacent to Bridge Decks

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Topic 7.4 Steel Decks

7.4.1 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, as presented in Topic 7.4.2.

7.4.2 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. For open grid decks, the trade-off of this weight savings is that water is permitted to pass through the deck, which deteriorates the superstructure, bearings and substructure. Steel grid decks can be filled or partially with concrete to prevent the water from passing through. The four basic types of steel decks are:

- Orthotropic decks
- Buckle plate decks
- Corrugated steel decks
- 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 their supports. The deck acts integrally with the steel superstructure. An orthotropic deck becomes the top flange of the entire floor system (see Figure 7.4.1).

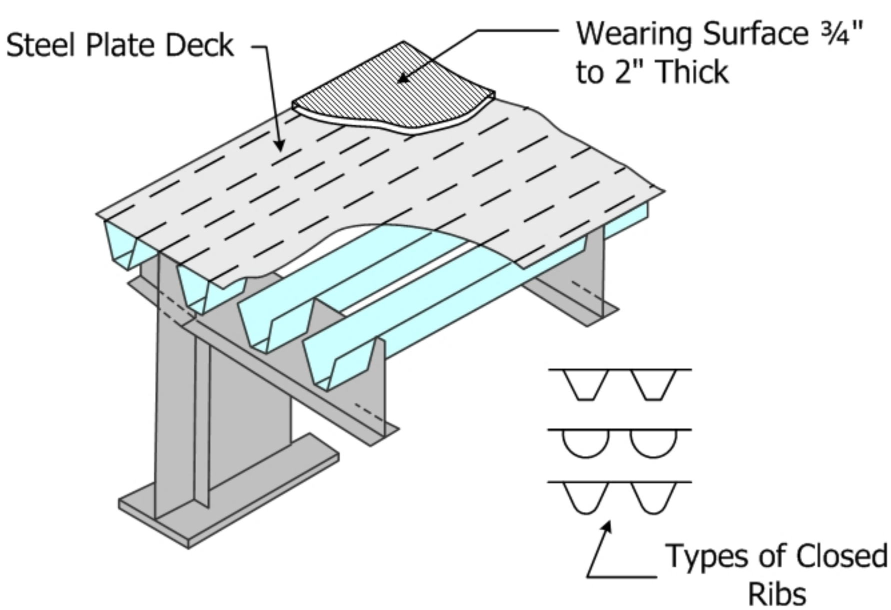

Figure 7.4.1 Orthotropic Bridge Deck

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 7.4.2). The plates are concave or "dished" with drain holes in the center. The sides are typically riveted to the superstructure. Buckle plate decks serve as part of the structural deck and as the deck form. They are not being used in current design but many buckle plate decks are still in service.

Figure 7.4.2 Underside View of Buckle Plate Deck

Corrugated Steel Decks

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 bituminous wearing surface (asphalt) (see Figure 7.4.3). The bituminous wearing surface thickness varies from the centerline of the deck to the edge of the roadway, to achieve proper cross slope. The corrugated flooring spans between the supporting superstructure. Corrugations are smaller than stay-in-place (SIP) forms, but the steel is thicker, ranging from 0.1 inch to 0.18 inch. The steel planks are welded in place to steel superstructure. In the case of timber superstructures, the corrugated flooring is attached by lag bolts. The corrugations are filled with bituminous pavement, and then a wearing surface is applied. There are no reinforcement bars utilized in this deck type.

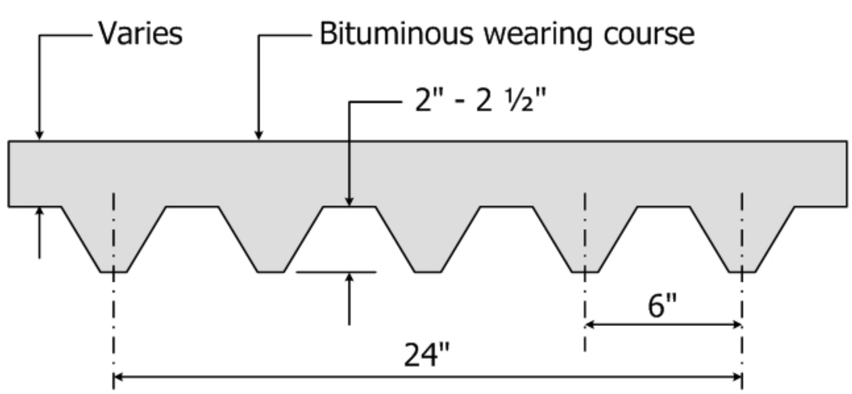

Figure 7.4.3 Corrugated Steel Floor

Grid Decks

Grid decks are the most common type of steel deck because of their light weight and high strength. They are commonly welded, riveted or fitted units, which may be open, filled or partially 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. Grid decks are often found on rehabilitated bridges. Their lower weight reduces the dead load on a rehabilitated bridge, and their installation method can reduce the time that the bridge will be closed for repairs.

The four types of grid decks include:

- Welded grid decks
- Riveted grate decks
- Concrete-filled 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 7.4.4).

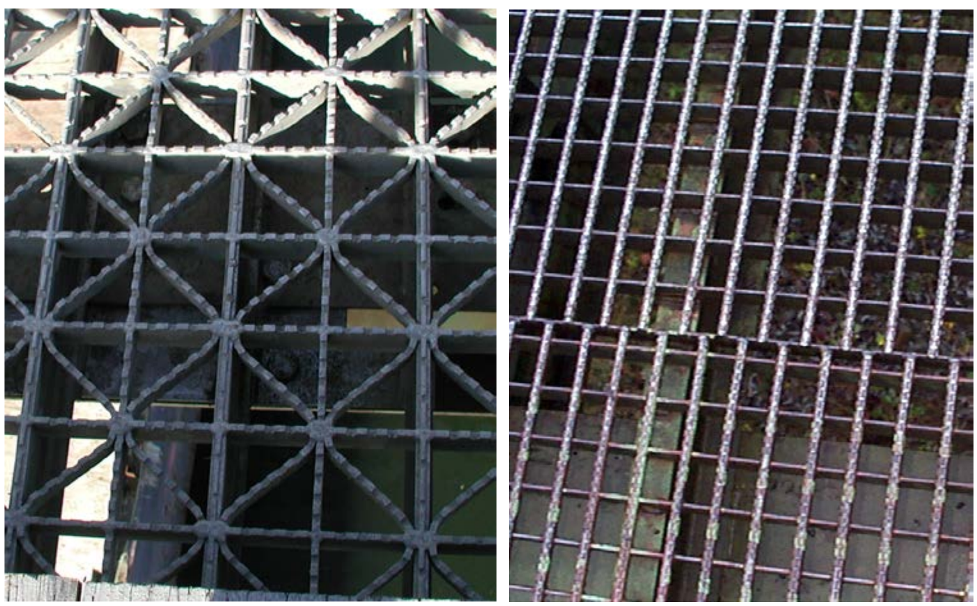

Figure 7.4.4 Various Patterns of Welded Steel Grid Decks

The bearing bars support the grating. Bearing bars are laid on top of the beams or stringers perpendicularly and are then field-welded or bolted to the superstructure. These bars are also referred to as the primary or main bars (see Figure 7.4.7).

The distribution bars are grating bars that are laid perpendicular to 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 (see Figure 7.4.7).

The supplementary bars are grating bars parallel to the bearing bars. They are also shop- or field-welded to the cross bars. Not all grating systems have supplementary bars. These supplementary bars are also referred to as tertiary bars.

Riveted Grid Decks

A riveted grid deck consists of bearing bars, crimp bars, and intermediate bars. Bearing bars run perpendicular to the superstructure and are attached to the beams or stringers by either welds or bolts. They are similar to the bearing bars in welded grates (see Figure 7.4.5).

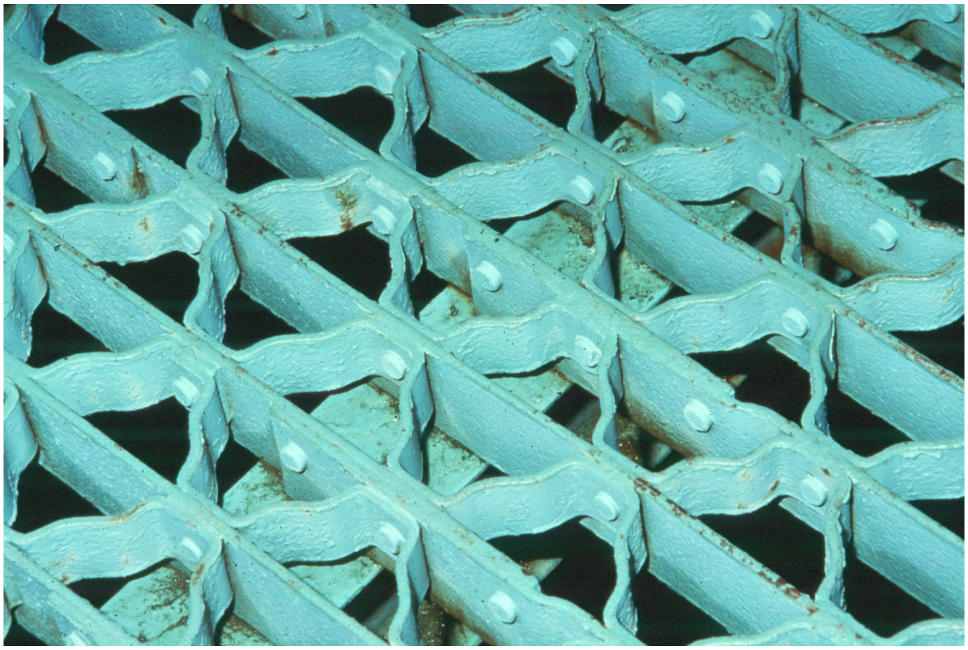

Figure 7.4.5 Riveted Grid Deck

Crimp bars are riveted to the bearing bars to form the grating.

Intermediate bars are 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. Intermediate bars may not be present on all riveted grate decks.

Welds and rivets used to construct steel grid decks have long been be a source of cracking. In recent years, steel grid decks have been fabricated to eliminate the use of welds or rivets. The bearing bars are fabricated with slotted holes. Transverse distribution bars are inserted into the slots rotated into position and locked into place without the use of any welds or rivets (see Figure 7.4.6).

Figure 7.4.6 Steel Grid Deck with Slotted Holes (to eliminate welding and riveting)

Concrete-Filled Decks

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 or fully filled. The addition of the concrete is not normally considered when determining the total capacity of the concrete-filled deck.

Fully-filled decks are grid decks that have been completely filled with concrete (see Figure 7.4.7). These decks provide the maximum protection of the underlying bridge members. Form pans are welded at the bottom of the grid to hold the concrete.

Partially-filled decks are grid decks which the top portion is filled with concrete. This provides a reduction in the dead load from the fully-filled deck and the protection of a concrete-filled system.

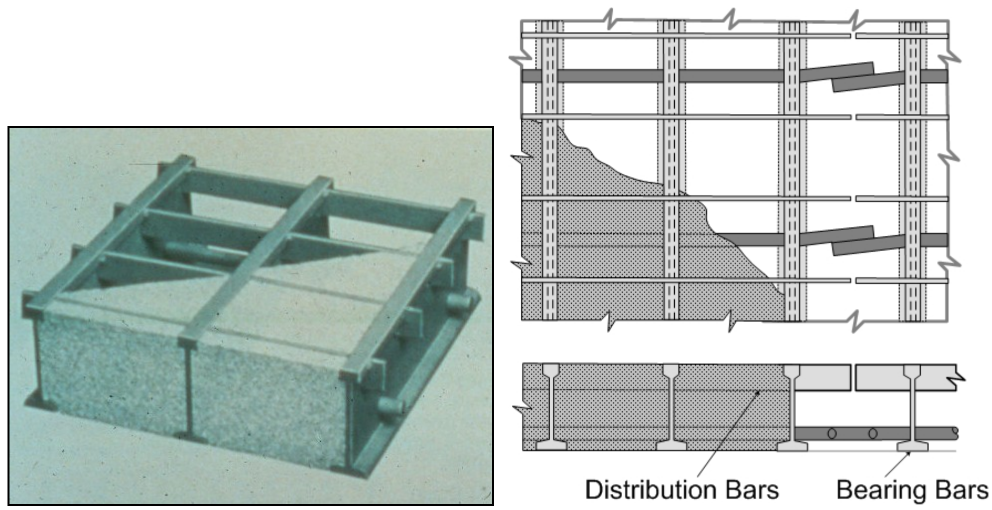

Figure 7.4.7 Concrete-Filled Grid Deck

Figure 7.4.8 Filled and Un-filled Steel Grid Deck

Exodermic Decks

Exodermic decks are a newer type of bridge deck. Reinforced concrete is composite with the steel grid (see Figure 7.4.9). Composite action is achieved by studs that extend into the reinforced concrete deck 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.

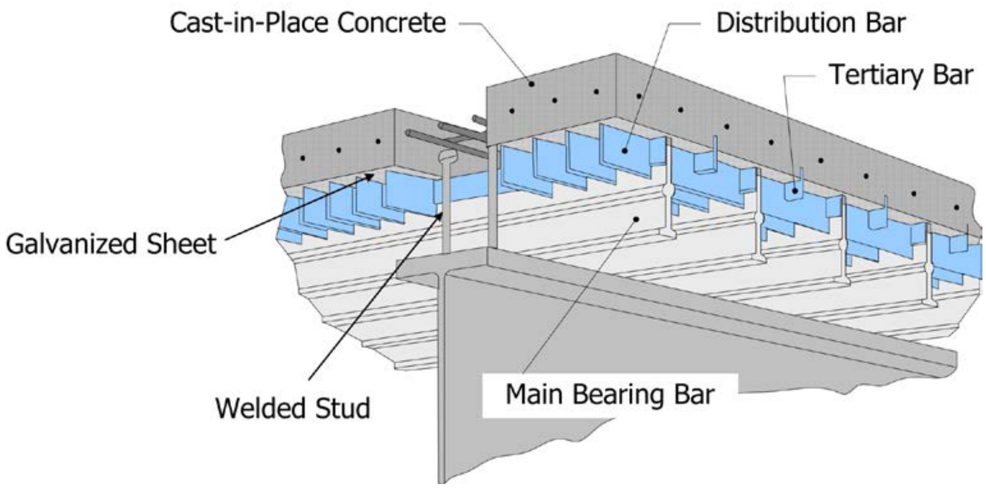

Figure 7.4.9 Schematic of Exodermic Composite Profile

7.4.3 Wearing Surfaces

Wearing surfaces protect the steel deck, provide an even riding surface, and may reduce the water on the deck, bearings and superstructure. Wearing surfaces for steel decks can consist of:

- Serrated steel
- Concrete
- Bituminous
- Gravel

Studs can be welded to steel decks for skid resistance.

Serrated Steel

Open grid decks usually have serrated edges on the grating (see Figure 7.4.4). These serrations allow the standing water to pass more easily through the deck and reduces the chance of hydroplaning.

Concrete

Concrete above the top of the grids, acts as the wearing surface for filled grid decks. This concrete wearing surface and the concrete used to fill the grids are generally placed at the same time. Different types of concrete wearing surfaces are listed and described in Topic 7.2.3. In the case of an exodermic bridge deck, the wearing surface is part of a reinforced deck.

Bituminous

Steel plate decks, such as orthotropic decks, typically have a layer of bituminous or asphalt as the wearing surface. Bituminous overlays generally range from 1 ½ inches up to 3 inches thick. Corrugated steel plank decks also have bituminous wearing surfaces.

An epoxy bituminous polymer concrete also is used for orthotropic bridge deck wearing surfaces. Unlike conventional bituminous mixes, epoxy bituminous polymer concrete will not melt after it has cured because of the thermoset polymer in the mix. This polymer is different than thermoplastic polymer used in conventional bituminous mixes. Epoxy bituminous polymer concrete is used when high strength and elastic composition are important.

Gravel

Corrugated metal decks may utilize a gravel wearing surface applied to the top of the deck. For these type of decks, drains may be located at midspan to minimize water accumulation in the corrugations.

7.4.4 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

Galvanizing can be used to protect steel decks. The galvanized coating retards the corrosion process and lengthens the life of the steel deck. This occurs by coating the bare steel with zinc. The two dissimilar 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 member 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 needs to be mixed properly, and the surface has to be prepared correctly.

Metalizing

Metalizing is a protective coating for steel. Specifically, it is a thermal spray method, by either flame or arc, for applying aluminum and zinc coatings on steel. Metalizing coatings applied to steel are generally a zinc-aluminum coating and can be applied in the shop or field. The coating provides protection to the steel similar to galvanizing. Metalized coatings often have a top coat (sealer) to extend service life.

Overlay

Another protective system for steel decks is the overlay material itself. The overlay covers the steel deck to create a barrier from corrosive agents. Overlays slow down the deficiency process for steel decks.

Epoxy Coating

Epoxy coating steel grates is another means of protecting the steel decking. This protective coating is rare since the deck becomes very slippery when wet. However, there are a limited number of steel decks with epoxy coating still in service.

7.4.5 Overview of Common Deficiencies

Some of the common steel deck deficiencies are listed below. Refer to Topic 6.3 to review steel deficiencies in detail.

- Bent, damaged, or missing members
- Corrosion
- Fatigue cracks
- Other stress-related cracks

7.4.6 Inspection Methods and Locations

Methods

Visual

The inspection of steel decks for surface corrosion, section loss, buckling, and cracking is primarily a visual activity. Most surfaces of the steel deck can be visually inspected. See Topic 6.3 for a more detailed explanation of visual inspection methods for steel bridge members.

Physical

Once the deficiencies are identified visually, physical methods can be used to verify the extent of the deficiency. Use an inspection hammer or wire brush to remove loose corrosion. This partial loss of cross section due to corrosion is known as section loss. Section loss can be measured using a straight edge and a tape measure. However, a more exact method of measurement, such as calipers or a D-meter, can be used to measure the remaining section of steel. More accurate section loss measurements can be recorded after removal of all corrosion products (rust scale).

Non-corroded bridge members can be measured to verify dimensions recorded in the plans or inspection report are accurate. If incorrect member sizes are used, the load rating analysis for safe load capacity of the bridge is not accurate.

Broken or cracked welds and rivets can be found by listening to the bridge deck. Listen for any unusual or clanking noises as vehicles drive across the steel deck.

Advanced Inspection Methods

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

- Acoustic emissions testing
- Computed tomography
- Corrosion sensors
- Smart coatings
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Robotic inspection
- Ultrasonic testing
- Eddy current

Other methods, described in Topic 15.3.3, include:

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

Locations

The primary locations for steel deck inspection include:

- **Bearing and shear areas** – check the primary bearing bars for buckled bars, cracked welds, broken fasteners, or missing bars which connect the steel deck to the supporting floor system.
- **Areas exposed to traffic** – examine the top surface for wheel ruts or wear. Verify that the deteriorated deck will not damage tires.
- **Tension areas** – on steel grid decks, check positive and negative moment regions of the primary bearing bars. Look for deficiencies such as broken, bent, fatigue cracks or other stress related cracks, or missing bars.
- **Areas exposed to drainage** – check areas where drainage can lead to corrosion. Look at areas along the curb lines that collect dirt and debris.
- **Corrugated deck** – check between the support points for section loss due to corrosion. Vertical movement of the deck under live load may indicate weld failure.
- **Orthotropic decks** – check orthotropic steel plate decks for debonding of the overlay, rust-through or cracks in the steel plate, and for the development of fatigue cracks in the web elements or connecting welds. Check the connection between the orthotropic plate deck and supporting members.
- **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. Check the condition of the concrete.
- **Areas previously repaired** – document the location and condition of any repair plates and their connections to the deck.

Figure 7.4.10 Broken Members of an Open Steel Grid Deck

7.4.7 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the deck. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 58) for additional details about the NBI component condition rating guidelines.

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

Element Level Condition State Assessment

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

NBE No.	Description
28	Steel Deck – Open Grid
29	Steel Deck – Concrete-Filled Grid
30	Steel Deck – Corrugated/ Orthotropic

BME No.	**Description**
510	Wearing Surfaces
515	Steel Protective Coating
520	Deck/Slab Protection Systems

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

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

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
363	Steel Section Loss
366	Deck Traffic Impact

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

Table of Contents

Chapter 7

Inspection and Evaluation of Bridge Decks and Areas Adjacent to Bridge Decks

Topic 7.5 Deck Joints, Drainage Systems, Lighting and Signs

7.5.1 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, contraction and rotation of the deck and superstructure. Deck joints are usually located at the each abutment, above piers in multiple span bridges, or at the ends of a drop-in span. In most bridges, the deck joints 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 will be able to withstand all possible weather extremes in a given area. It does all of this without compromising the ride quality of vehicles crossing the bridge.

Drainage Systems

The function of a drainage system is to remove water and all hazards associated with it from the structure. The function is also to protect the superstructure, bearings and substructure. The drainage system also requires as little maintenance as possible and is located so that it does not cause safety hazards.

Lighting and Signs

Lighting serves various functions on bridge structures. 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.

7.5.2 Components of Deck Joints, Drainage Systems, Lighting and Signs

Deck Joints

Do not confuse deck joints 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 six categories of deck joints are:

- Strip seal expansion joint
- Pourable joint seal
- Compression joint seal
- Assembly joint with seal (Modular)
- Open expansion joint
- Assembly joint without seal

Strip Seal Expansion Joint

A strip seal consists of two slotted steel anchorages cast into the deck or backwall. A neoprene seal fits into the grooves to span the joint extrusion. This joint can accommodate a maximum movement of approximately 4 inches (see Figure 7.5.1).

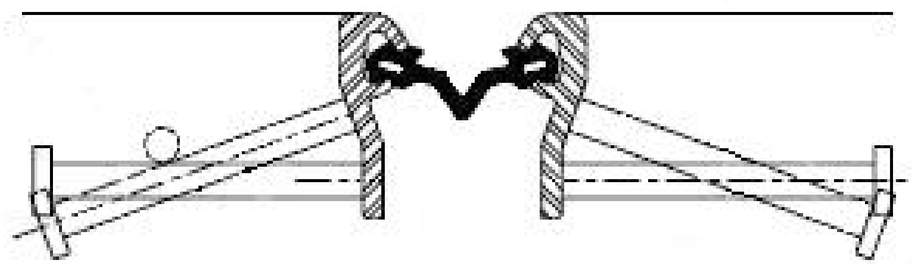

Figure 7.5.1 Strip Seal (Drawing Courtesy of the D.S. Brown Co.)

Pourable Joint Seal

A pourable joint seal is made up of three materials: backing material, preformed joint filler and poured sealant (see Figure 7.5.2). The top of this material is 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 and/or a bond breaker. Since the pourable joint seal can only accommodate a movement of about 1/4 inch, it is usually found on short span structures (see Figure 7.5.3).

Neoprene foam can be used as an alternative to the preformed expansion filler, allowing a movement of greater than ¼”. Both types are used mostly for short span prestressed concrete bridges.

Figure 7.5.2 Pourable Joint Seal

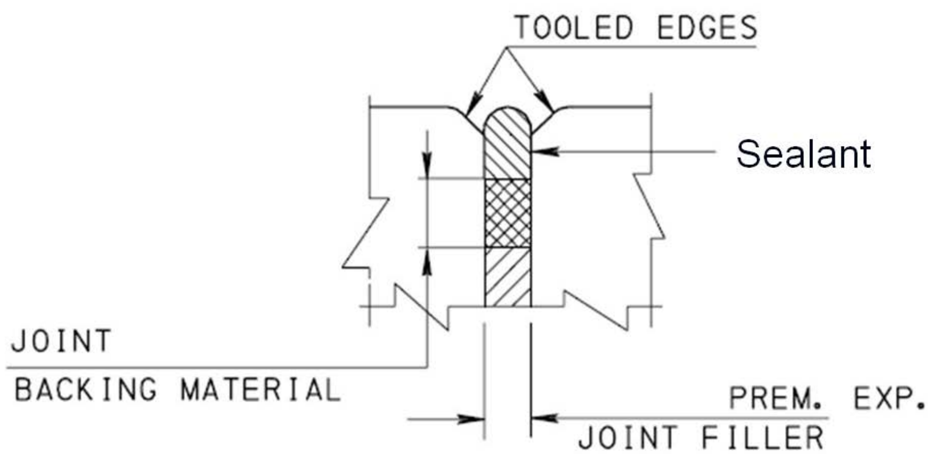

Figure 7.5.3 Cross Section of a Pourable Joint Seal

Compression Joint Seal

A compression joint seal consists of neoprene formed in a rectangular shape with a honeycomb cross section (see Figure 7.5.4 and 7.5.5). The honeycomb design allows the compression joint seal to fully recover after being distorted during bridge expansion and contraction. It is called a compression joint seal because it functions in a partially compressed state at all times. Compression joint 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 joint seal can accommodate a maximum movement of approximately 2 inches.

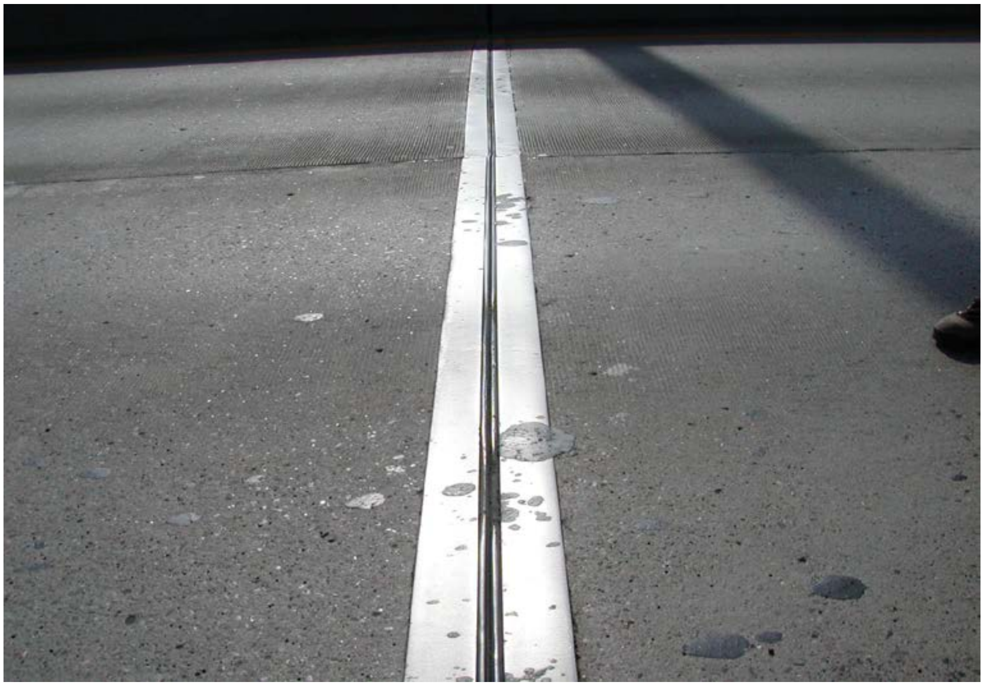

Figure 7.5.4 Compression Joint Seal with Steel Angle Armoring

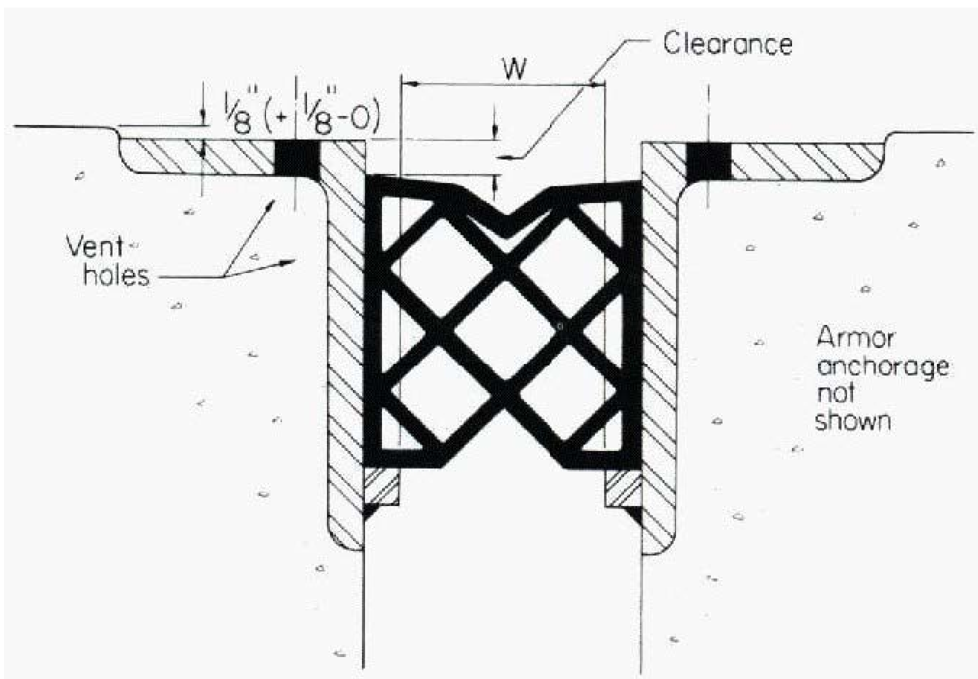

Figure 7.5.5 Cross Section of a Compression Joint Seal with Steel Angle Armoring

Cellular Seal

The cellular seal is similar to the compression joint seal, and its armoring is almost identical. However, they differ in the type of material used to seal the joint. Unlike the compression joint seal, the cellular seal is made of a closed-cell foam that allows the joint to move in different directions without losing the seal (see Figure 7.5.6). 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.

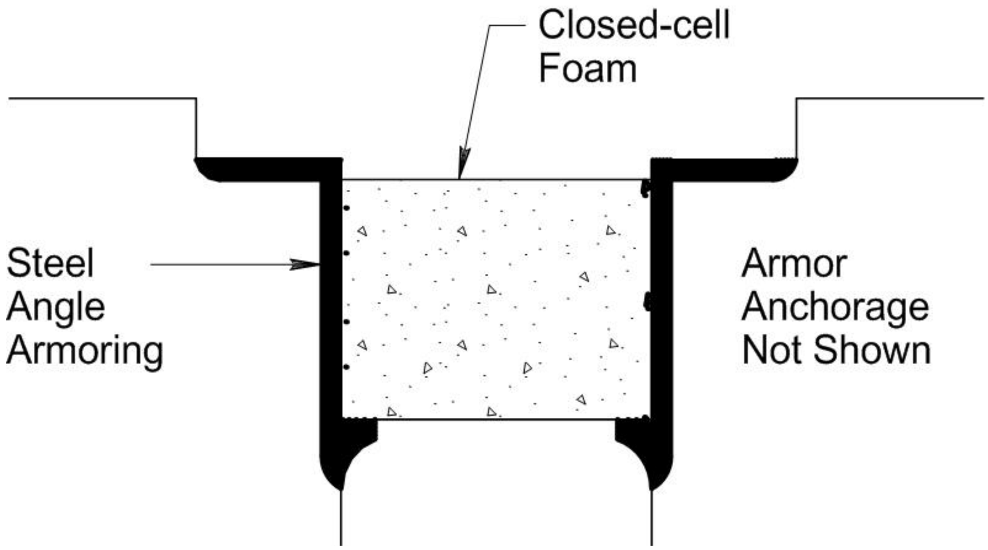

Figure 7.5.6 Cross Section of a Cellular Seal

Assembly Joint with Seal (Modular)

A modular 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 7.5.7 and 7.5.8). The normal range of operation for movement is between 4 and 24 inches. It can, however, be fabricated to accommodate movements up to 48 inches.

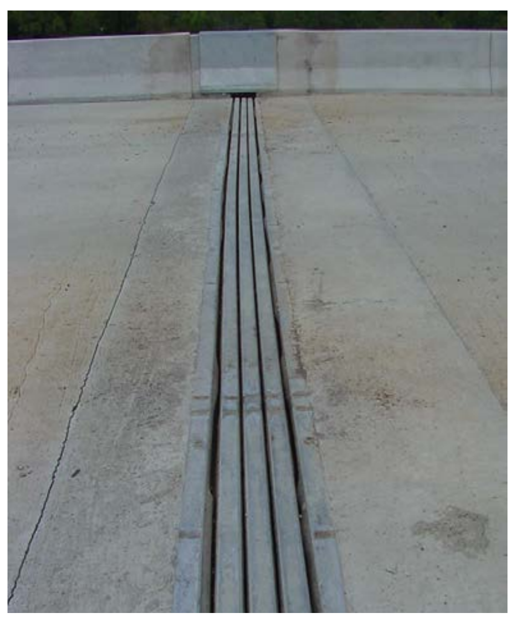

Figure 7.5.7 Modular Seal

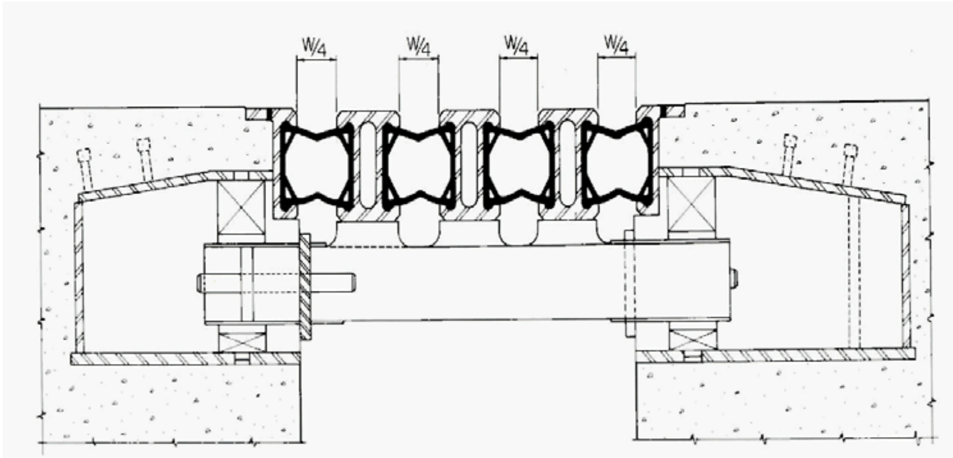

Figure 7.5.8 Schematic Cross Section of a Modular Seal

Assembly joints may also include plank seals, sheet seals and asphaltic expansion joints.

Plank 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 up to 4 inches (see Figure 7.5.9). Plank seals are no longer commonly used.

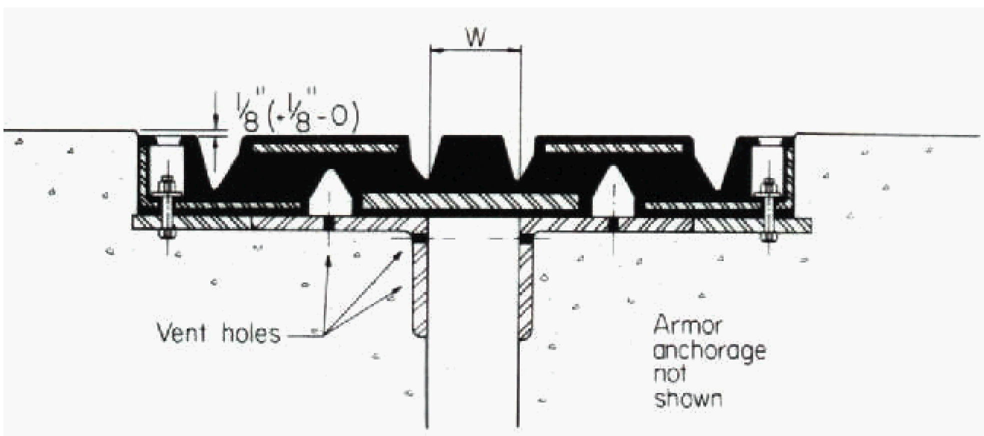

Figure 7.5.9 Plank Seal

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 4 inches (see Figure 7.5.10).

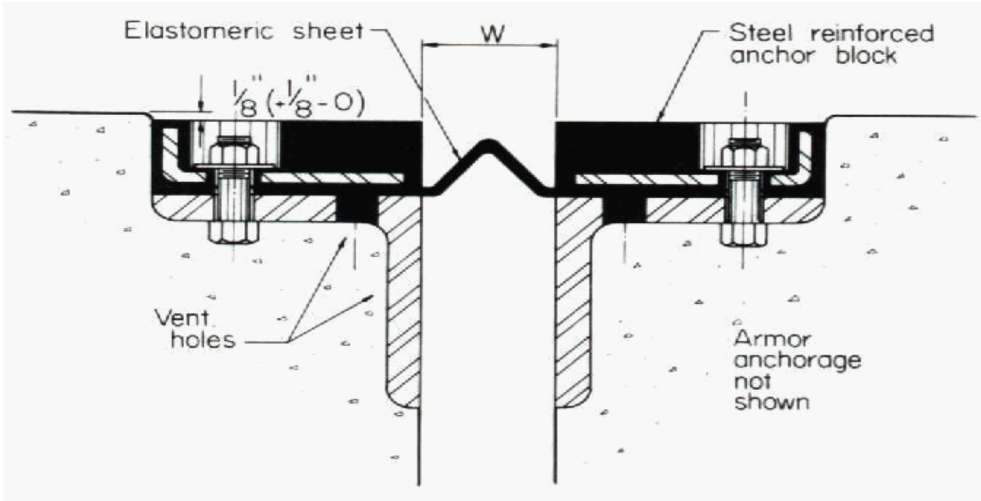

Figure 7.5.10 Sheet Seal

Asphaltic Expansion Joint

An asphaltic expansion joint is typically used on short bridges that are to be overlaid with asphalt. The joint can accommodate an expansion of 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 7.5.11).

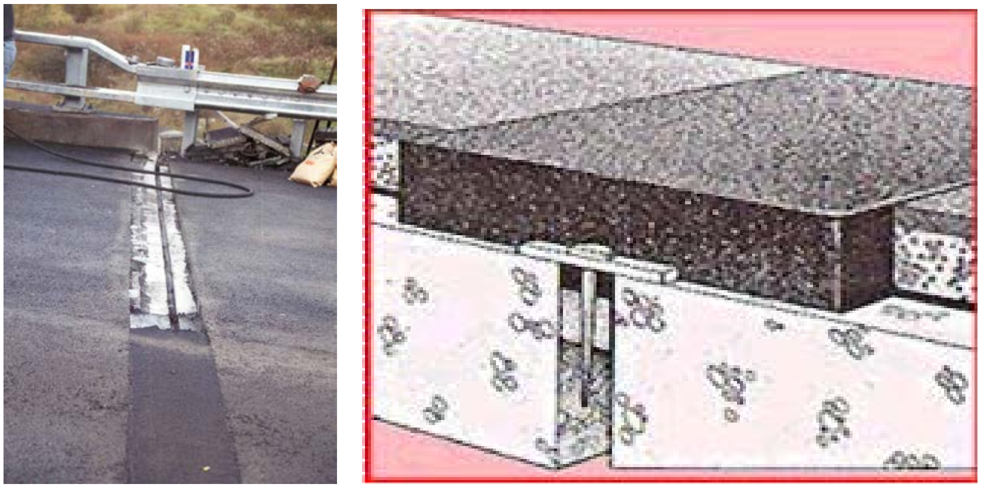

Figure 7.5.11 Asphaltic Expansion Joint

Open Expansion Joint

Open expansion 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 open expansion joint is usually unprotected, but the deck and backwall can be armored with steel angles. Open expansion joints are common on short span bridges with concrete decks (see Figures 7.5.12 and 7.5.13).

Figure 7.5.12 Open Expansion Joint

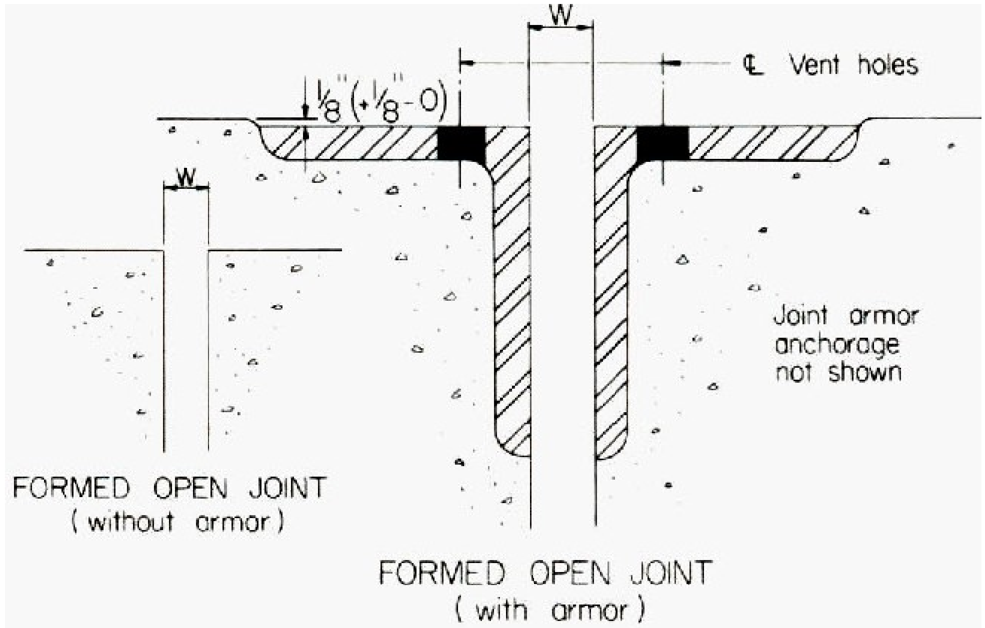

Figure 7.5.13 Cross Section of a Open Expansion Joint

Assembly Joint without Seal

Assembly joints without a seal can include finger plate joints and sliding plate joints.

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 requiring greater expansion. 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 4 to over 24 inches (see Figures 7.5.14 through 7.5.16).

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 7.5.14 Finger Plate Joint

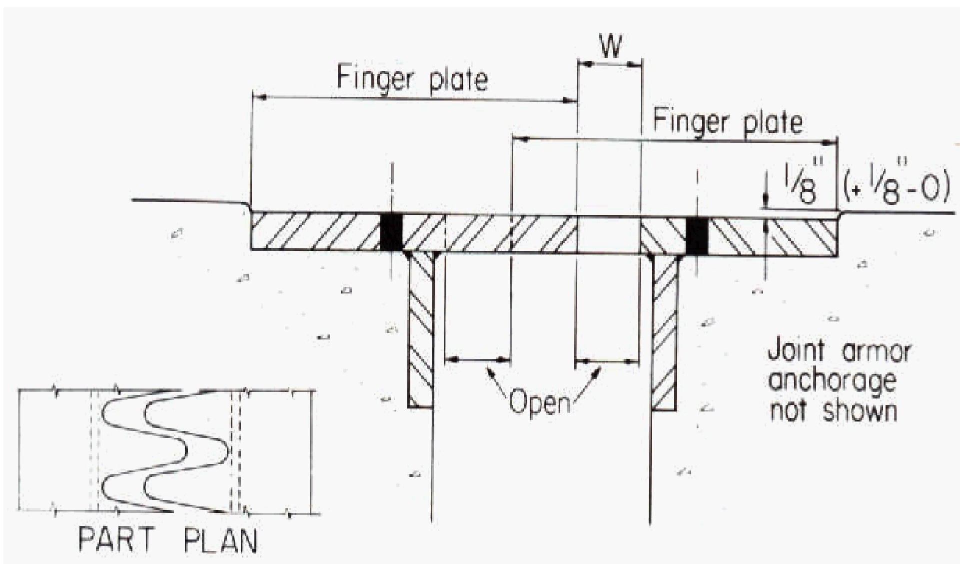

Figure 7.5.15 Cross Section of a Cantilever Finger Plate Joint

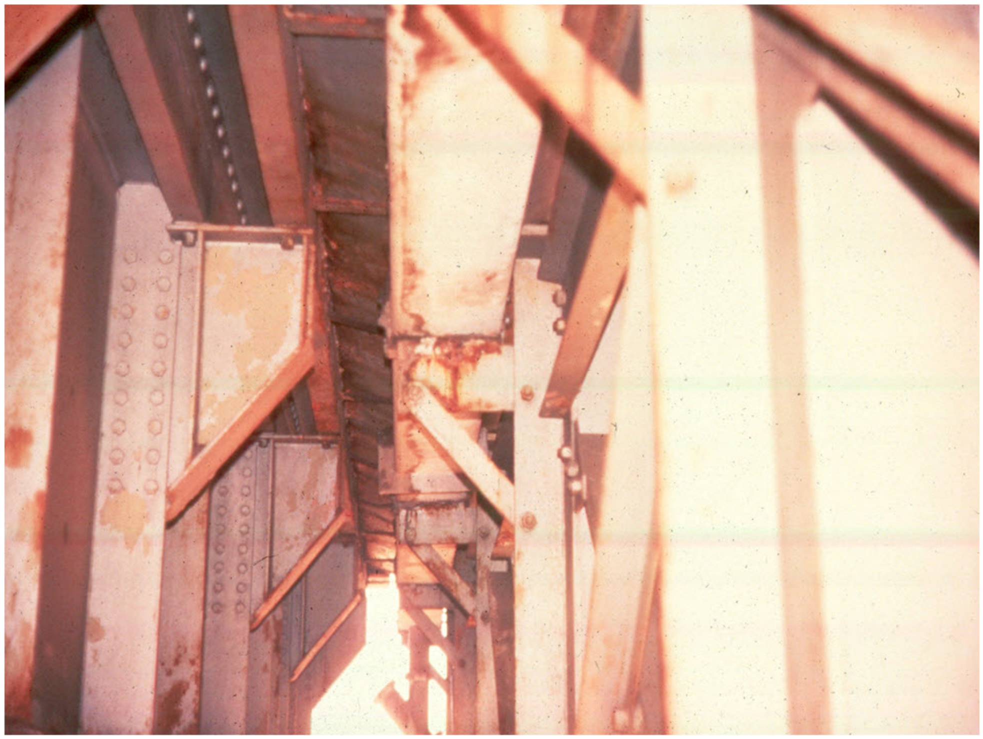

Figure 7.5.16 Supported Finger Plate Joint

Sliding Plate Joint

A sliding plate joint is composed of two plates and is not watertight. The top plate slides across the bottom plate. 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 7.5.17 and 7.5.18). The sliding plate joint can accommodate a maximum movement of approximately 4 inches.

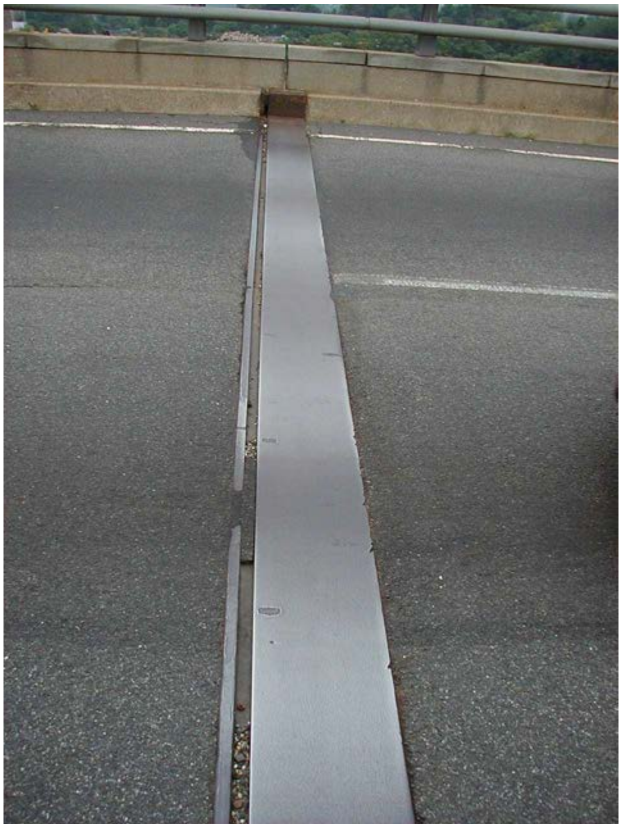

Figure 7.5.17 Sliding Plate Joint

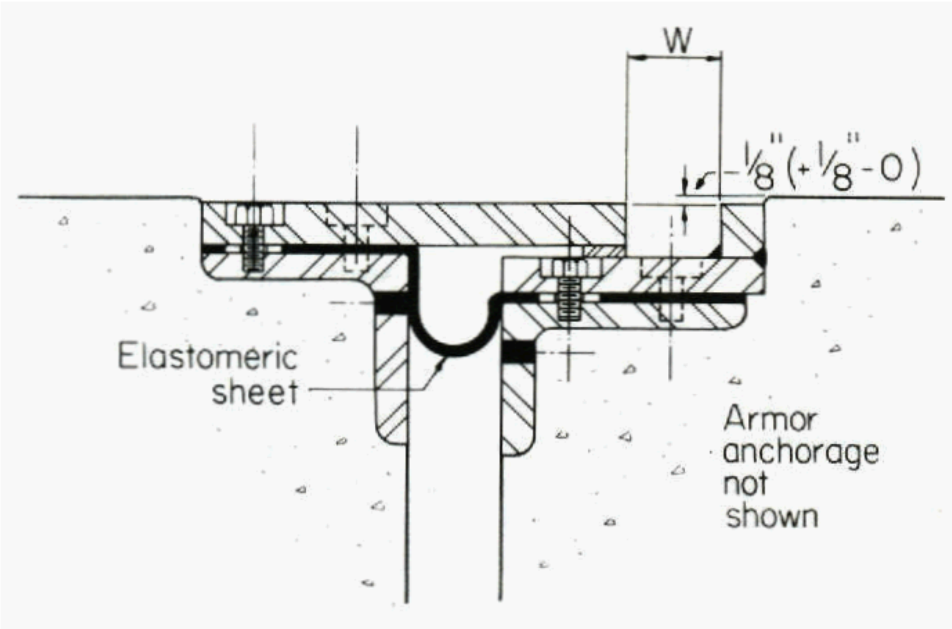

Figure 7.5.18 Cross Section of a Sliding Plate Joint

Drainage Systems

Drainage systems are created to move water away from specific locations on or near a bridge. This is to prevent potential hazards or damage to the bridge and to protect the superstructure, bearings and substructure.

There can be up to three different drainage systems on a given bridge:

- Deck drainage (includes inlet and outlet systems)
- Joint drainage
- Substructure drainage

In order to perform an inspection of a deck drainage system, it is necessary to become familiar with its various elements:

- **Grade and cross slope -** directs the runoff to the inlets and eliminates or reduces ponding. Runoff is the water and any contents from the surface of the bridge deck.
- **Inlets** – receptacle to receive water
- **Outlet pipes -** outlet pipe leads water away from the drain
- **Downspout pipes** – directs deck drainage away from outlet pipes to nearby storm sewer
- **Cleanout plugs -** removable plug in the piping system that allows access for cleaning.
- **Drainage troughs -** maybe located under open joints to divert runoff away from underlying superstructure, bearings and substructure members

Deck Drainage

Inlet System

Inlet systems incorporate scuppers or deck drains (see Figure 7.5.19). Scuppers 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 2 inches on center. A bicycle safety grate has steel rods placed perpendicular to the grating bars, spaced at approximately 4 inches on center. The 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.

Deck drains could either be open holes or embedded tubes that are either made of plastic or metal and functions similarly to scuppers**.**

In addition to scuppers and deck drains, inlet systems may also include openings in a filled grid deck or slots in the base of a parapet.

Figure 7.5.19 Bridge Deck Scupper (left) and Deck Drain (right)

Outlet System

The outlet system may incorporate either outlet pipes or downspouts. If present, the outlet pipe leads water away from the inlet system (see Figure 7.5.20). When the bridge is not over a roadway, the outlet pipe normally extends just below the superstructure so that drainage water is not windblown onto the superstructure. When a bridge is over a roadway or a feature, the outlet pipe normally connects to other pipes to prevent runoff from directly falling on the roadway beneath.

When a bridge is located over a roadway, a downspout pipe is used to direct the drainage 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 7.5.21).

Figure 7.5.20 Outlet Pipe

Figure 7.5.21 Downspout Pipe

Joint Drainage

Joint drainage systems use either a separate gutter or trough (see Figure 7.5.22) to collect water that passes through an unsealed joint such as either a finger plate joint or sliding plate joint. Once the water is collected here, the water is then transported away from the bridge elements.

Debris from the deck runoff may cause the trough to clog frequently and require frequent cleaning to enable them to function as designed. These systems may be constructed from copper, steel or elastomeric sheeting.

Figure 7.5.22 Drainage Trough

Substructure Drainage

Substructure drainage consists of weep holes and underdrains. Weep holes are small drainage holes found in abutment stems and retaining walls which allows water to drain from behind the abutment (see Figure 7.5.23). This type of drainage reduces the earth pressure behind the substructure.

Underdrains are perforated pipes which are routed along the back face of the abutment or retaining wall and are channeled to a nearby waterway or storm water drainage systems.

Figure 7.5.23 Weep Holes

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 luminary attached to a bracket arm. Both the luminary 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 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 will 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:

- Warning signs
- Traffic regulatory signs
- Guide signs

Warning Signs

Warning signs alert drivers to existing or potentially hazardous conditions.

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 will be placed on these abutment walls and pier edges. The approaching pavement will be appropriately marked to warn motorists of the hazard.

Traffic Regulatory Signs

Regulatory signs instruct drivers to do or not do something. Traffic regulatory 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.

Speed Limit

Speed limit signs are important since they indicate any speed restriction that may exist on the bridge.

Weight Limit

Weight limit signs are very important since they indicate the maximum vehicle load that can safely use the bridge.

Guide Signs

Guide signs come in a variety of shapes and colors and have information to help drivers arrive safely at their destination.

7.5.3 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
- Deck joint troughs clogged or partially clogged
- 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 presented unacceptably or incorrectly
- Signs defaced or covered with graffiti
- Corrosion or section loss on lighting or sign supports
- Loose or missing anchorages at supports
- Lighting outages

7.5.4 Inspection Locations and Methods for Deck Joints, Drainage Systems, Lighting and Signs

Deck Joints

The deck joints allow for the expansion and contraction of the bridge deck and superstructure. Inspectors report and document any site conditions that prevent the deck joints from functioning properly.

Using the NBIS guidelines, there is not a separate item on the Structure Inventory and Appraisal (SI&A) sheet to code the serviceability of deck joints. Deck joint conditions are not considered in the rating of the deck. However, it is important for the inspector to note their condition since leaking deck joints lead to the deterioration of superstructure, bearings and substructure elements beneath the joints.

The Element Level Inspection system, however, does rate deck joints. For a detailed description of deck joint condition states, see the AASHTO *Guide Manual for Bridge Element Inspection* and the evaluation section of this topic.

Inspect the deck joints for:

- Dirt and debris accumulation
- Proper alignment (horizontal/vertical)
- 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 7.5.24 and 7.5.25).

Figure 7.5.24 Debris Lodged in a Sliding Plate Joint

Figure 7.5.25 Dirt in a Compression Seal Joint

Proper Alignment

To ensure a smooth ride and to prevent snow plow damage, deck joints are placed to provide smooth transition between spans or between the end spans and the abutments. Any vertical or horizontal displacement between the two sides of the joint is documented by the inspector. On straight bridges, the joint opening is designed to be parallel across the deck.

In a finger plate joint, the individual fingers will mesh together properly, and they will be in the same plane as the deck surface. Document any vertical or horizontal misalignment (see Figure 7.5.25).

Figure 7.5.26 Improper Vertical Alignment at a Finger Plate Joint

Thermal expansion and contraction of a bridge is possible through properly functioning deck joints. It is important that the relative joint openings are consistent with the current temperature. It is also important to record the deck joint opening to determine if the opening is consistent with the temperature at the time of the inspection. Temperature above the average causes the bridge to expand (lengthen) resulting in a decreased or smaller deck joint opening. Temperature below average causes the bridge to contract (shorten) resulting in an increased deck joint opening. Measurements will be taken at each curb line and the centerline of the roadway. The superstructure temperature can be taken by a regular thermometer or by placing a surface temperature thermometer against the superstructure member itself. The superstructure temperature is generally about 3 to 5 degrees Fahrenheit below 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 7.5.26). It can also cause damage to armored plates. Any of these conditions will be noted by the inspector. Also look for evidence of leakage through sealed joints.

Figure 7.5.27 Failed Compression Seal

Indiscriminate Overlays

When new pavement or wearing surface 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 7.5.27).

Figure 7.5.28 Asphalt Wearing Surface over an Expansion Joint

Deck Joint Supports

Joint supports are required when large deck joints are utilized. These supports connect the deck joint devices to the superstructure. Inspect these joint supports carefully for proper function and for corrosion and section loss (see Figure 7.5.28).

Figure 7.5.29 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.

Deck Areas Adjacent to Deck Joints

Many deck joints are connected to the deck utilizing some type of anchorage (see Figures 7.5.15 and 7.5.17). Examine deck areas adjacent to deck joints for material deterioration such as section loss, spalls, delaminations, and vehicular/snow plow damage. Deterioration of the deck in these areas may be an indication of problems with the anchorage.

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 NBIS 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.

Inspect the following drainage system elements:

- Grade and cross slope
- Inlets
- Outlet pipes
- Downspout pipes
- Cleanout plugs
- Drainage troughs

Grade and Cross Slope

The deck cross slope and profile should not prevent runoff from entering the deck drains and inlets. Check to determine adequate cross slope or profile is provided so that water runs off the bridge deck at a sufficient rate. Ponding is an indication of insufficient cross slope or profile.

Inlets

Careful examination of the drainage elements is to be performed at each bridge inspection since runoff conditions can change. For the runoff to be carried away from the structure, inlets are designed at a sufficient size and spacing to allow water to pass through. Document any deteriorated, broken or missing grates on inlets, which can be considered a safety issue. Inlets should be clear of debris to allow the runoff to enter. Clogged inlets lead to accelerated deck deterioration and the undesirable condition of standing water in the traffic lanes (see Figure 7.5.29). Standing water on the deck is a safety hazard.

Figure 7.5.30 Clogged Scupper

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 will be long enough so that runoff is not discharged onto the structure.

Downspout Pipes and Cleanout Plugs

Downspout pipes are a series of pipes (see Figure 7.5.30). Examine downspout pipes for split or disconnected pipes that may allow runoff to accelerate deterioration of the structure. Check the connections between the downspout 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.

Cleanout plugs are removable caps that allow access so the outlet pipes can be cleaned and kept clear of debris (see arrows in Figure 7.5.30). Having access to the cleanout plugs is important. If there is evidence of clogged outlet pipes, make recommendations to remove the cleanout plugs and clear the debris.

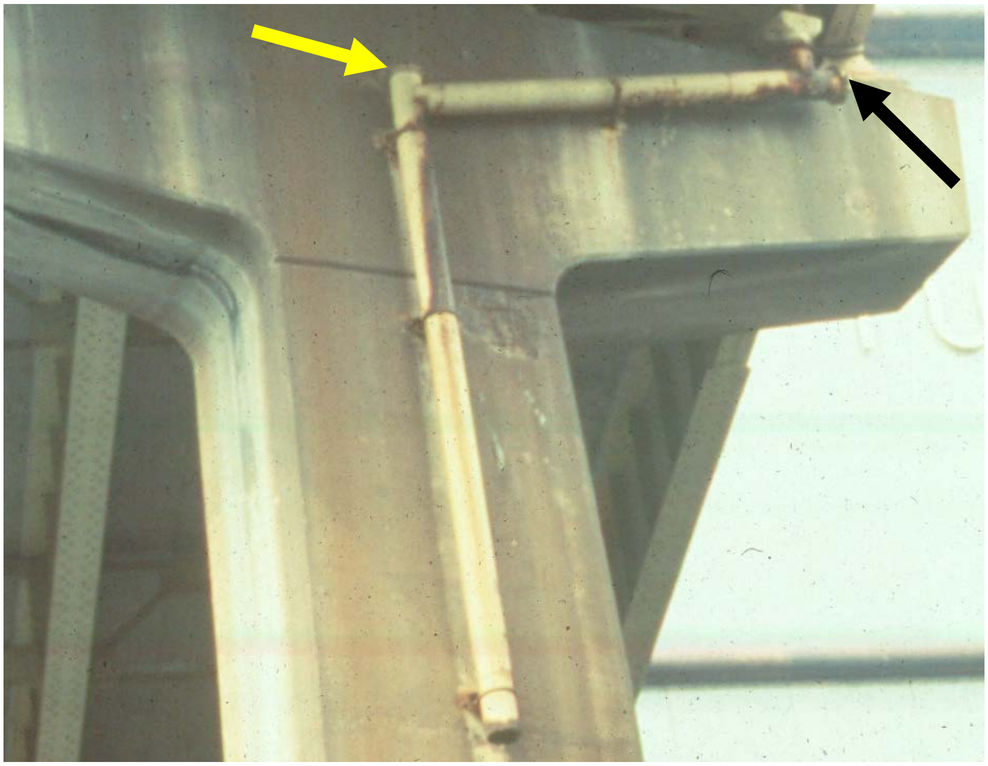

Figure 7.5.31 Outlet Pipe with Cleanout Plugs

Drainage Troughs

Carefully examine drainage troughs, if present, located under unsealed joints. A buildup of debris can accelerate the deterioration of the trough or its supports and allow water to drain onto structural members (see Figure 7.5.30). If possible, use a shovel to clean as much debris as practical; report the remaining condition for appropriate maintenance work. Once cleaned, note any holes found in the trough. Record any evidence that indicates the trough is overflowing.

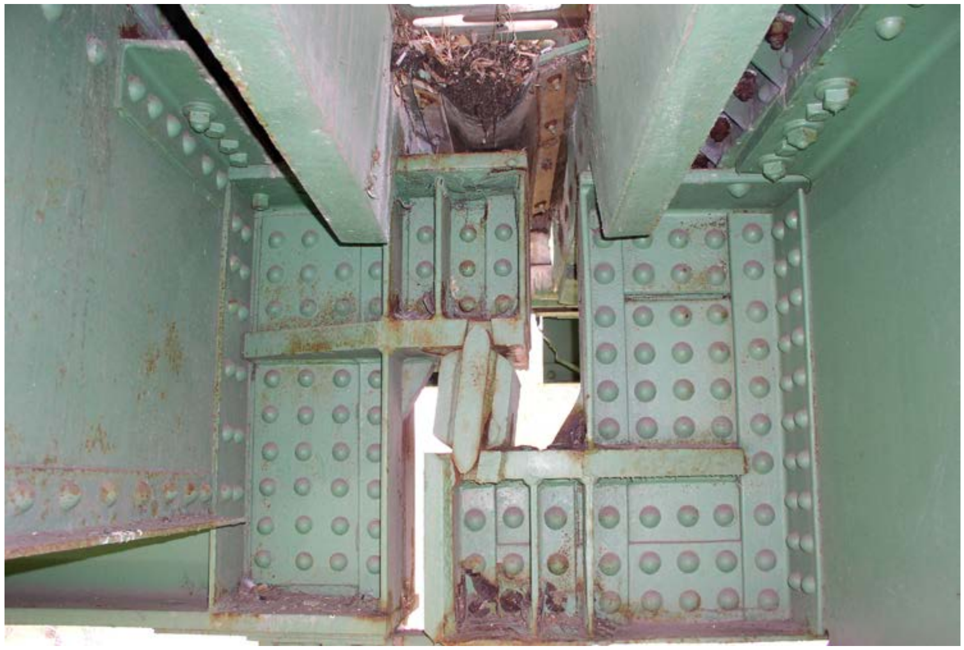

Figure 7.5.32 Drainage Trough with Debris Accumulation

Lighting

All lights are to be clearly visible. Verify that all lights are functioning and that they are not obstructed from view. Check for fatigue cracking, corrosion, and collision damage to light supports. Verify that appropriate lighting is provided. Exercise caution against electrical shock. Contact the maintenance department to de-energize the lighting.

Signs

Signs are to be located sufficiently in advance of the structure to permit the driver adequate time to react. All signs are to be clearly legible. Verify that signs have not been defaced and are not obstructed from view. Inspect for fatigue cracking, corrosion, and collision damage to sign supports. Verify that appropriate signing is provided.

Adhesive Anchors

Adhesive anchors have several applications used in bridge construction, but two of the most prominent include fence or light support attachments and sign mounting (see Figure 7.5.33).

It may be necessary to review the design or as-built or rehabilitation drawings to determine how the anchor bolts are attached to the bridge. Based upon the application, the anchor itself may not be visible which will make a visual inspection difficult. There are clues that would provide some evidence as to the condition and effectiveness of the anchor.

Depending on the direction of the loads, the anchor bolts may experience one or more of the following: axial tension, axial compression, tension or compression due to moment or shear. Although the yielding of the anchor bolts is a failure mode, the inspector looks for anchor embedment problems, or anchor pullout, that results from adhesive failure. Fence or light pole anchors will more often than not be subjected due to moment and not axial tension. Axial tension anchorages are not very common unless the attachment is "hung" from the bridge.

Figure 7.5.33 Sign and Light Structures Attached to a Bridge

Be sure to pay particular attention to any anchor pullout that may exist. This could be caused by excessive creep or failure of the adhesive. Look for inconsistent spacing between the anchor plate and concrete surface (see Figure 7.5.34). This could occur from axial tension load or tension due to moment.

Figure 7.5.34 Sign Attachment Exhibiting Anchor Pullout

Large signs attached to the backside of a concrete barrier is another possible application where adhesive anchors may be used in today's bridges (see Figure 7.5.35). It is important to not only document the anchor location and orientation, but to determine, as close as possible, how the anchor functions. Note any gaps between the mounting hardware and the concrete surface where the anchor is embedded. If gaps exist, measure the gaps and document them with notes, photographs, and sketches.

Figure 7.5.35 Sign Mount with Loose Adhesive Anchorage

7.5.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

Deck joints, drainage systems, lighting, and signs do not impact the deck rating, but their condition can be described on the inspection form. Record deficiencies in deck joints, drainage systems, lighting, and signs on the inspection and maintenance sheets.

Element Level Condition State Assessment

In an element level condition state assessment of deck joints, there are no AASHTO National Bridge Elements (NBEs).

Possible AASHTO Bridge Management Elements (BMEs) are:

BME No.	Description
300	Strip seal expansion joint
301	Pourable joint seal
302	Compression joint seal
303	Assembly joint with seal (modular)
304	Open expansion joint
305	Assembly joint without seal

The unit quantity for deck joints is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

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 following Deflect Flags are applicable in the evaluation of steel decks:

Defect Flag No.	Description
356	Steel Cracking/Fatigue
357	Pack Rust
358	Concrete Cracking
359	Concrete Efflorescence
363	Steel Section Loss

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

Drainage systems, lighting, and signs have no National Bridge or Bridge Management separate element numbers. The condition of the drainage systems, lighting, and signs, however, will be noted on the inspection form.

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Chapter 7
Inspection and Evaluation of Bridge Decks and Areas Adjacent to Bridge Decks

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Topic 7.6 Safety Features

7.6.1 Introduction

Highway design includes 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), safety features such as highway or bridge barrier systems have been provided to screen motorists from the hazards present (see Figure 7.6.1). Such barriers sometimes constitute fixed object hazards themselves, though hopefully of less severity than the hazard they screen.

Figure 7.6.1 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 railings
- Transitions
- Approach guardrail
- Approach guardrail ends

These four basic components are designed to satisfy agency standards, which specify acceptable heights, materials, strengths, and geometric features.

Bridge Railings

The function of bridge railing is to contain and smoothly redirect errant vehicles on the bridge (see Figures 7.6.2 and 7.6.3). Many bridge rails could conceivably do this, but the safety of the driver and redirection of the vehicle must be taken into account.

Transitions

A transition occurs between the approach guardrail system and bridge railing (see Figures 7.6.2 and 7.6.3). Its purpose is to provide both a structurally secure connection to the rigid bridge railing and also a zone of gradual stiffening and strengthening of the more flexible approach guardrail system. Stiffening is essential to prevent "pocketing" or "snagging" of a colliding vehicle just before the rigid bridge railing end.

If, on impact, a redirective device undergoes relatively large lateral displacements within a relatively short longitudinal distance, pocketing is said to have occurred. Depending on the degree, pocketing can cause large and unacceptable vehicular decelerations. When a portion of the test vehicle, such as a wheel, engages a vertical element in the redirective device, such as a post, snagging is said to have occurred. The degree of snagging depends on the degree of engagement. Snagging may cause large and unacceptable vehicular decelerations.

Approach Guardrail

The approach guardrail system is intended to screen motorists from the hazardous feature beneath the bridge as they are approaching the bridge (see Figures 7.6.2 and 7.6.3). This approach guardrail screening is often extended in advance of the bridge so as to also keep motorists from any additional hazardous roadside features on the approach to the bridge.

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.

Approach Guardrail Ends

The approach guardrail end is the special traffic friendly anchorage of the approach guardrail system (see Figures 7.6.2 and 7.6.3). It is located at the end at which vehicles are approaching the bridge. Ground anchorage is essential for adequate performance of the guardrail system. A special approach guardrail end is necessary in order to minimize its threat to motorists as another fixed object hazard within the roadside recovery area.

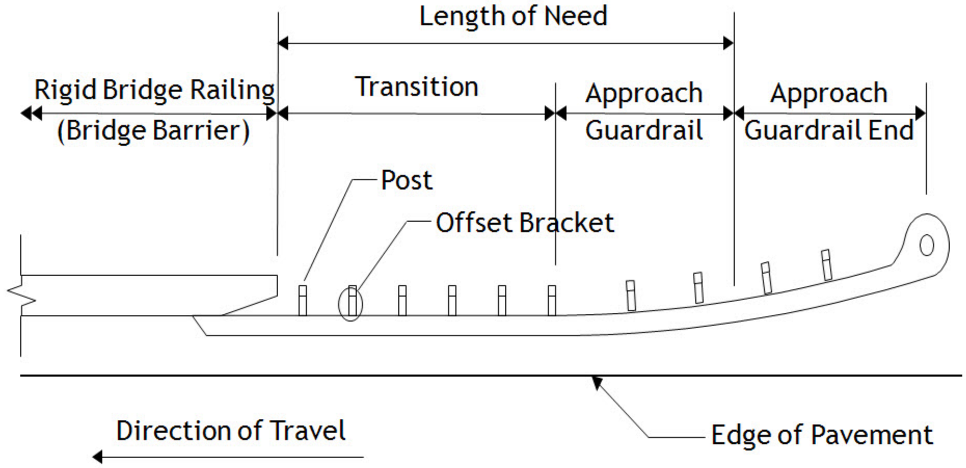

Figure 7.6.2 Traffic Safety Features

Figure 7.6.3 Bridge Railing, Transition, Approach Guardrail and Approach Guardrail End

7.6.2 Evaluation

Each of the various elements of traffic safety features are 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 function as intended. The elements for bridge railings and guardrail systems, including transitions and approach guardrail ends, must pass the minimum standard criteria established by AASHTO and FHWA and NCHRP minimum standards for structures on the NHS.

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 safety feature 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 successfully crash tested and approved 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.

History of Crash Testing

Full scale crash testing began in 1962. "Highway Research Correlation Circular 482" listed methods including specified vehicle mass, impact speed and approach angle.

National Cooperative Highway Research Program (NCHRP) Project 22-2 in 1973 addressed questions not covered in "Circular 482". The final report is "NCHRP Report 183" which gave more complete set of testing methods. Several parts of the document were known to be based on inadequate information. Methods gained wide acceptance after their publication in 1974, but the need for periodic updates was recognized. In 1976, Transportation Research Board (TRB) committee A2A04 accepted responsibility for reviewing procedure efficiency. The minor changes were addressed and "Transportation Research Circular 191" was published in 1978.

NCHRP Project 22-2(4) initiated in 1979 was intended to address the major changes required in "NCHRP Report 183". The objective was to review, revise and expand the scope of "Circular 191" to reflect current technology. Final report was published as NCHRP Report 230 "Recommended Procedures for the Safety Performance Evaluation of Highway Safety Appurtenances" in 1980. This report served as the primary reference for full scale crash testing of highway safety appurtenances.

In 1987, AASHTO recognized the need to update Report 230. This was due to changes in vehicle fleet, emergence of many new designs, matching safety performance to levels of roadway utilization, new policies requiring use of safety belts, and advances in computer simulation and other evaluation methods. NCHRP Project 22-7 was initiated to update Report 230.

Efforts began in 1989 with a series of white papers. A panel met to discuss the issues, debate and develop a consensus on methods to be included in the update. The draft document was distributed for review, and the panel met two more times to discuss comments and to develop a final document. This document is NCHRP Report 350.

In 1997, NCHRP Project 22-14, "Improvement of the Procedures for the Safety-Performance Evaluation of Roadside Features", was initiated to determine the relevance and efficiency of procedures outlined in NCHRP Report 350. Upon completion in 2001, it was determined that NCHRP Report 350 should include updates to the following high priority topics:

- Test vehicles and specifications
- Impact conditions
- Critical impact point
- Efficacy of flair space model
- Soil type/condition
- Test documentation
- Working width measurement

In 2002, updates to NCHRP Report 350 were initiated through NCHRP Project 22-14(2). Upon completion in 2008, the revised crash testing methods were published as the 2009 *AASHTO Manual for Assessing Safety Hardware* (MASH). Key differences between MASH and Report 350 include the following:

- Presentation as a dual-unit document
- Changes in test matrices including impact angles, impact speeds, head-on tests with mid-size vehicles, and mandatory TMA tests that were previously optional
- Changes in test installations including performance-based specifications for soil, rail element splices, cable tensioning, and more-detailed documentation and requirements
- Changes in test vehicles including target vehicle weight and vehicle minimum center of gravity
- Changes in evaluation criteria including windshield damage, maximum roll and pitch angles, and required documentation on vehicle rebound for crash cushion tests
- Changes in test documentation and performance evaluation

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 methods 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* were issued as an alternate to the long-standing *AASHTO Standard Specifications for Highway Bridges*. The 2010 *AASHTO LRFD Bridge Design Specifications* have six test levels that correspond to the six levels in Report 350.

In 2009, NCHRP Report 350 was superseded by *AASHTO* Manual for Assessing Safety Hardware (MASH). The updates contained in MASH represent major revisions to Report 350 including changes to testing vehicles, impact conditions, criteria used for evaluation, and the addition of newly approved traffic safety features. The implementation of MASH on the NHS includes the following:

- The AASHTO Technical Committee on Roadside Safety is responsible for developing and maintaining the evaluation criteria as adopted by AASHTO. FHWA is responsible for review and acceptance of highway safety hardware
- All highway safety hardware accepted prior to adoption of MASH using criteria contained in NCHRP Report 350 may remain in place and may continue to be manufactured and installed
- Highway safety hardware accepted using NCHRP Report 350 criteria is not required to be retested using MASH criteria
- If highway safety hardware that has been accepted by FHWA using NCHRP Report 350 criteria fails testing using MASH criteria, AASHTO and FHWA will jointly review the test results and determine a proper course of action

Current FHWA Policy

Bridge railings to be installed on National Highway System (NHS) projects must meet the acceptance criteria contained in NCHRP Report 350 (Figure 7.6.4) or AASHTO MASH (Figure 7.6.5). 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 44 mph, a TL-2 bridge railing is considered acceptable.

Test Level	Impact Speed	Vehicle Type
TL-1	30 mph	1,800 lb car; 4,500 lb pickup
TL-2	45 mph	1,800 lb car; 4,500 lb pickup
TL-3	60 mph	1,800 lb car; 4,500 lb pickup
TL-4	60 mph 50 mph	1,800 lb car; 4,500 lb pickup 18,000 lb single unit truck
TL-5	60 mph 50 mph	1,800 lb car; 4,500 lb pickup 80,000 lb tractor trailer
TL-6	60 mph 50 mph	1,800 lb car; 4,500 lb pickup 80,000 lb tanker trailer

Figure 7.6.4 2010 *AASHTO LRFD Bridge Specifications* Test Level Index (based on the NCHRP Report 350 Test Level Index)

Test Level	Impact Speed	Vehicle Type
TL-1	31 mph	2,420 lb car; 5,000 lb pickup
TL-2	44 mph	2,420 lb car; 5,000 lb pickup
TL-3	62 mph	2,420 lb car; 5,000 lb pickup
TL-4	62 mph 56 mph	2,420 lb car; 5,000 lb pickup 22,000 lb single unit truck
TL-5	62 mph 50 mph	2,420 lb car; 5,000 lb pickup 79,300 lb tractor trailer
TL-6	62 mph 50 mph	2,420 lb car; 5,000 lb pickup 79,300 lb tanker trailer

Figure 7.6.5 2009 *AASHTO Manual for Assessment of Safety Hardware* (MASH) Test Level Index

Railings that have been found acceptable under the crash testing and acceptance criteria of NCHRP Report 350 or the *AASHTO LRFD Bridge Design Specifications* will be considered as meeting the requirements of AASHTO

MASH. This comparison of equivalencies has been tabulated by the FHWA in their November 20, 2009 memorandum on the implementation of AASHTO MASH.

The FHWA continues to encourage support for development of railing test level selection methods. New crash-tested railings continue to be approved and added, and their identity and features can be obtained from the FHWA Roadside Hardware Policy and Guidance website:

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/

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

All of the bridge and longitudinal roadside barrier systems, transitions, and approach guardrail ends which have been found to meet the various crash test requirements of NCHRP Reports 350 and/or AASHTO MASH are identified on the FHWA Roadside Hardware Policy and Guidance website, which is located at:

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/

This 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 for NCHRP 350 criteria can also be found on the website.

Longitudinal barriers specifically used as bridge barriers which meet current NCHRP Report 350 crash test performance criteria are found at:

www.fhwa.dot.gov/bridge/bridgerail/

The "2005 Bridge Rail Guide" can be found at this web site. This document contains photographs, drawings, test level, contact information and cost for the acceptable bridge rails per NCHRP Report 350 criteria.

Additional information can also be found in the current AASHTO "Roadside Design Guide" and in the current AASHTO MASH.

Available Training Courses

FHWA-NHI 380032A Roadside Safety Guide

This three-day course discusses the use of the *Roadside Design Guide* including applying the clear zone concept, identifying the need for a traffic barrier, recognizing unsafe roadside design features and elements.

FHWA-NHI 380079 AASHTO Roadside Design Guide – Web-based

This web-based course provides an overview of the *Roadside Design Guide* including applying the clear zone concept, identifying the need for a traffic barrier, recognizing unsafe roadside design features and elements.

FHWA-NHI 380034 Design, Construction, and Maintenance of Highway Safety Appurtenances and Features

This one-day course allows participants to identify advantages and disadvantages of different types of longitudinal barriers and crash cushions, identify NCHRP 350 tested safety appurtenances, and recognize substandard or potentially hazardous highway appurtenances or features.

FHWA-NHI 380034A Design, Construction, and Maintenance of Highway Safety Appurtenances and Features

This two-day course allows participants to identify advantages and disadvantages of different types of longitudinal barriers and crash cushions, identify NCHRP 350 tested safety appurtenances, and recognize substandard or potentially hazardous highway appurtenances or features.

FHWA-NHI 380034B Design, Construction, and Maintenance of Highway Safety Appurtenances and Features

This three-day course allows participants to identify advantages and disadvantages of different types of longitudinal barriers and crash cushions, identify NCHRP 350 tested safety appurtenances, and recognize substandard or potentially hazardous highway appurtenances or features.

The courses listed above can be found by using the following website link: http://www.nhi.fhwa.dot.gov/.

7.6.3 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. Obtain a listing of currently conforming versus non-conforming bridge safety features 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.

Document the condition of the safety features in the inspection report even though the condition is not considered in the appraisal coding. After determining whether the safety features at the site are acceptable, assign an appraisal code. The FHWA *Coding Guide* contains four entries for safety features: one each for the bridge railing, transition, approach guardrail, and approach guardrail ends. Some states have modified and set different coding standards.

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) or N (Not applicable or a safety feature is not required*) for each element of Item 36 (page 19), FHWA *Coding Guide*:

36A Bridge railings
36B Transitions
36C Approach guardrail
36D Approach guardrail ends

* For structures on the NHS, national standards are set by federal regulation. For those not on the NHS, it shall be the responsibility of the highway agency (state, county, local or federal) to set standards.

While there is only one safety features coding for each element, there are at least two bridge railings and up to four approach guardrail treatments. Therefore, code the worst situation for each element even though they may occur at different locations on the bridge.

The following descriptions of Appraisal Items 36A – 36D are for bridge sites on the National Highway System (NHS). Local bridge owners may set different criteria to evaluate Items 36A – 36D.

36A Bridge Railings

Factors that affect the appraisal ratings of NHS bridge railings, Item 36A, include height, material, strength and geometric features (see Figure 7.6.6). The railing must be able to smoothly redirect the impacting vehicle. Evaluate the bridge railings using the current *AASHTO LRFD Bridge Design Specifications* for specific geometric criteria and static loading. The railings must be crash tested as per FHWA policy (see Figure 7.6.7). If the railings meet these criteria, they are considered acceptable. Other railings that have been crash tested but do not meet current requirements are considered unacceptable.

Figure 7.6.6 Acceptable Bridge Rail

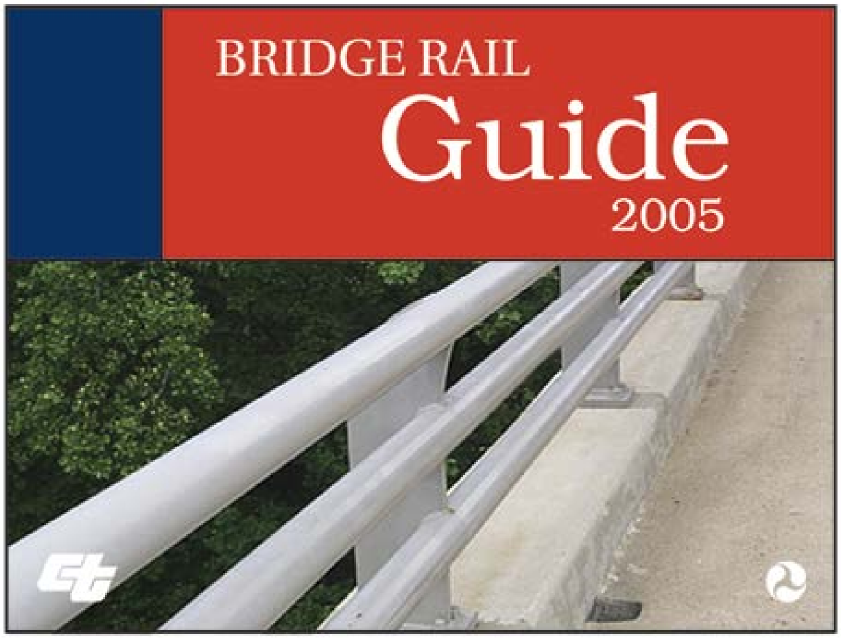

Figure 7.6.7 Bridge Rail Guide

36B Transitions

Appraisal Item 36B, transitions, requires the transition from the approach guardrail to the bridge railing be firmly attached to the bridge rail and gradually stiffened as it approaches the bridge rail (see Figure 7.6.8). 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

Figure 7.6.8 Acceptable Transition

The ends of curbs or safety walks are currently designed to gradually taper out or be shielded. Vehicle snagging is reduced 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 all current acceptable features. 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 railing 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 on the NHS. No transition is provided at all when the bridge railing and approach guardrail are not structurally connected.

36C Approach Guardrail

Because the need for a barrier generally does not stop at the end of the bridge, the approach guardrail, Item 36C, is evaluated for adequacy. Evaluate the structural adequacy and design compatibility of the approach rail and transition. The approach guardrail must be of adequate length and strength to shield motorists from the hazards at the bridge site. The guardrail is designed to safely redirect the impacting vehicle without snagging or pocketing. Acceptable design suggestions may be found in the *AASHTO Roadside Design Guide*, subsequent AASHTO guidelines, or the previously referenced FHWA website.

The strong post (steel or wood) W-beam guardrails with wood or approved plastic blocks (see Figure 7.6.9) are examples of systems meeting the requirements of Test Level 3, as are the strong post thrie-beam systems. The same W-beam barriers used with a steel block are included for Test Level 2. See Topic 7.6.2 for detailed explanation of the various test levels.

Figure 7.6.9 Approach Guardrail System and Approved Plastic Offset Block

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 for bridges on the NHS.

36D Approach Guardrail Ends

Evaluate approach guardrail ends, Item 36D, for adequacy. A variety of approach guardrail ends have been approved for use by the FHWA. The specific installation is dependent on various roadway features and testimony methods as administered by the National Cooperative Highway Research Program (NCHRP). Current listings of crash tested approach guardrail ends and documentation of their performance can be found at:

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/term_cush.cfm

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 7.6.10). 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 7.6.10 W-Shaped Guardrail End Flared and Buried into an Embankment

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. The guardrail must be flared beyond the clear zone which is the area beyond the traveled way available for vehicle recovery. This area may consist of shoulder, recoverable or non-recoverable slope, and/or clear run-out area. The required width depends on traffic volume, speed, and roadside geometry.

Burying the guardrail end has been used with and without flaring. If the guardrail end is turned down for burying without flaring, it has frequently produced rollover accidents and is not currently considered an acceptable approach guardrail end for high speed/high volume roadways.

One of several breakaway treatments can be used. The approach 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 approach guardrail end.

The last method for railing approach guardrail end 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 approach guardrail end locations where frequent impacts might be expected. This information can be found at:

http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/term_cush.cfm

In certain cases, such as at the trailing end of a one-way bridge, guardrail is not required at all since it will not prevent motorist from impacting what is under the bridge.

A type of approach guardrail end, which has sometimes been called a boxing glove, is not a current acceptable approach guardrail end unless properly flared away from the traveled way. If the guardrail ends are left unprotected, this is also unacceptable (see Figure 7.6.11).

Figure 7.6.11 Unacceptable Blunt Ends

7.6.4 Safety Feature Inspection

The inspection of bridge safety features involves evaluation of the condition of the bridge railing, the transition, the approach guardrail, and approach guardrail ends 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.

For structures which are over roadways, the adequacy and condition of traffic safety features for both the upper and lower roadways are to be evaluated during the inspection, but only the adequacy of the safety features for the roadway carried by the bridge is coded for Items 36A-36D.

Inspection

Criteria considered during the inspection of the bridge railing are the height, material, strength, geometric features, and the likelihood of acceptable crash test performance. See Topic 7.6.3 for the appraisal coding of Items 36A – 36D. Keep in mind that only the appraisal coding and the design of the traffic safety feature is addressed in Items 36 A – 36D. Record deficiencies due to the condition separately in the inspection notes.

Many state agencies have developed their own acceptance guidelines for bridge railings. Familiarize yourself with agency guidelines and standards for your state.

Bridge Railing

Comparison of existing bridge railing systems with approved crash-tested designs will establish their acceptability and crash worthiness.

Metal bridge railings should be firmly attached to the deck or superstructure and should be functional. Check especially for corrosion and collision damage, which might render these railings ineffective (see Figure 7.6.12). Check for loose or missing connections.

Concrete bridge railing is generally cast-in-place and engages reinforcing bars to develop structural anchorage in the deck or slab. Verify that the concrete is sound and that reinforcing bars are not exposed. Inspect for impact damage or rotation, and note areas of damage or movement.

Check for evidence of anchorage failure in precast parapets. Perform a physical examination by sounding exposed anchor bolts with a hammer. Check for separations between the base of the precast units and deck, or evidence of active water leakage between parapet and deck. Some states are removing all precast parapets because water is seeping in along the curb line and corroding reinforcement. This reinforcement can not be visually inspected.

Inspect post and beam railing systems for collision damage and deterioration of the various elements. Check post bases for loss of anchorage. The exposed side of the railing must be smooth and continuous.

For a through truss or arch configuration, separate traffic from structural members, especially fracture critical members, with an adequate railing system to prevent major structural damage to the bridge and protect vehicles.

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.

Figure 7.6.12 Deficiency Steel Post Bridge Railing

Approach Guardrail

For approach guardrails, verify that agency guidelines or 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. Note any areas where the railing may “pocket” during collision, causing an abrupt deceleration or erratic rebound.

Document any significant collision damage, which is evident (see Figure 7.6.13). Report posts which are displaced horizontally. Note any deficiency of guardrail elements, which could weaken the system. Check for cracks, rust or breakage of elements. Check wood posts for rot or insect damage, especially at the ground line. The connection between rails and posts should be secure and tight. Note any loose or missing bolts.

Check the approach rail for proper alignment. Note any area of settlement or frost heave. Posts embedded in the ground should not be able to be moved by hand. Check the slope beyond the posts for settlement or erosion which may reduce embedment of the posts (see Figure 7.6.14).

Unless specifically designed for impact, timber approach guardrail does not meet minimum criteria for strength on NHS roadways.

Figure 7.6.13 Approach Guardrail Collision Damage

Figure 7.6.14 Erosion Reducing Post Embedment

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. For nested installations, be sure that the approach rail is properly nested with the lap splice away from the direction of traffic (see Figure 7.6.15). Also check railing, post and offset bracket condition. Check the condition of the transition and look for material deficiencies similar to bridge railings and approach guardrails.

Timber should not be used for the rails in transitions on the National Highway System (NHS).

Figure 7.6.15 Proper Nesting of Guardrail at Transition

Approach Guardrail End

Note the type, condition, and suitability of any approach guardrail end. Acceptable crash-tested approach guardrail ends are identified in the *AASHTO Roadside Design Guide* or with current FHWA issuances. Check impact attenuation devices adjacent to bridge elements for evidence of damage due to collision and that the energy absorbing elements have not ruptured (see Figure 7.6.16). Ensure that any cables and anchorages are secure and undamaged. Check for material deficiencies that may affect the condition of the approach guardrail end.

Approach guardrail ends may not be required on the trailing end of a one-way bridge.

Figure 7.6.16 Impact Attenuator

Inspection for Non-NHS Bridges

The requirements for inspection of traffic safety features presented in this topic are applicable to bridges on the National highway System (NHS). For bridges which are not located on the NHS, it is up to each governing agency to set their own policies.

There are still various requirements that should be met as a minimum for these installations. The bridge rail must be crashworthy. The approach guardrail must be adequately connected to the bridge rail. Post spacing from the approach guiderail to the transition should be reduced to limit deflection. It is recommended to have nested rail at the transition, but it is not absolutely necessary. Approach guardrail ends should be crash worthy with no blunt ends. Existing turned down ends and breakaway cable terminal (BCT) approach guardrail ends are acceptable if governing policy is so stated. Crash worthy approach guardrail ends would be better, but may not be cost effective on low volume, low speed roads.

Figure 7.6.17 Timber Traffic Safety Features, Rocky Mountain National Park

7.6.5 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 approach guardrail end 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. They should meet the requirements for Item 36A, bridge railing. Inspect for collision damage and attachment to any additional safety features. Check for deterioration and spalling on concrete median barriers, and examine for corrosion and loose connectors on steel railings and posts.

7.6.6 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of bridge railings. The two major rating guideline systems currently in use are the FHWA *Coding Guide* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

Bridge railings do not impact the deck rating, but their condition can be described on the inspection form. Record deficiencies in bridge railings on the inspection and maintenance sheets.

Element Level Condition State Assessment

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

NBE No.	Description
330	Metal Bridge Railing
331	Reinforced Concrete Bridge Railing
332	Timber Bridge Railing
333	Other Bridge Railing
334	Masonry Bridge Railing

BME No.	Description
515	Steel Protective Coating
521	Concrete Protective Coating

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

The following Defect Flags may be applicable in the evaluation of bridge railings:

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
358	Concrete Cracking
359	Concrete Efflorescence
363	Steel Section Loss

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

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Chapter 8

Inspection and Evaluation of Timber Superstructures

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Chapter 8
Inspection and Evaluation of Timber Superstructures

Topic 8.1 Solid Sawn Timber Bridges

8.1.1 Introduction

Timber bridges are gaining resurgence in popularity in some parts 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 8.1.1).

Figure 8.1.1 Elevation View of a Solid Sawn Multi-Beam Bridge

8.1.2 Design Characteristics

Multi-beam Bridges

Solid sawn multi-beam bridges consist of multiple solid sawn beams spanning between substructure units (see Figure 8.1.2). The deck is supported by the beams and is typically comprised of transversely laid timber planks, and longitudinally laid planks called runners. 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 approximately 6 inches by 12 inches to 8 inches by 16 inches, and the beams are usually spaced about 24 inches on center.

Figure 8.1.2 Underside View of a Solid Sawn Multi-Beam Bridge

This bridge type is generally used in older, shorter span bridges, spanning up to 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 8.2).

Covered Bridges

Covered bridges are generally found along rural roads and get their name from the walls and roof which protect the bridge superstructure (see Figures 8.1.3 and 8.1.4). Covered bridges 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 or light weight vehicles. 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.

Figure 8.1.3 Elevation View of Covered Bridge

Figure 8.1.4 Inside View of Covered Bridge Showing King Post Truss Design

Trusses

The majority of covered bridges are essentially truss bridges (see Figure 8.1.5). Solid sawn timber members make up the trusses of these historic structures. The covers on the bridges prevent decay of the truss and are responsible for their longevity. Typical truss types for covered bridges include the king post, queen post, Town, Warren, and Howe (see Figure 8.1.6). The floor system consists of timber deck planks, stringers, and floorbeams. The span lengths of covered bridges generally range from 50 to 100 feet, although many are well over 100 feet and some span over 200 feet.

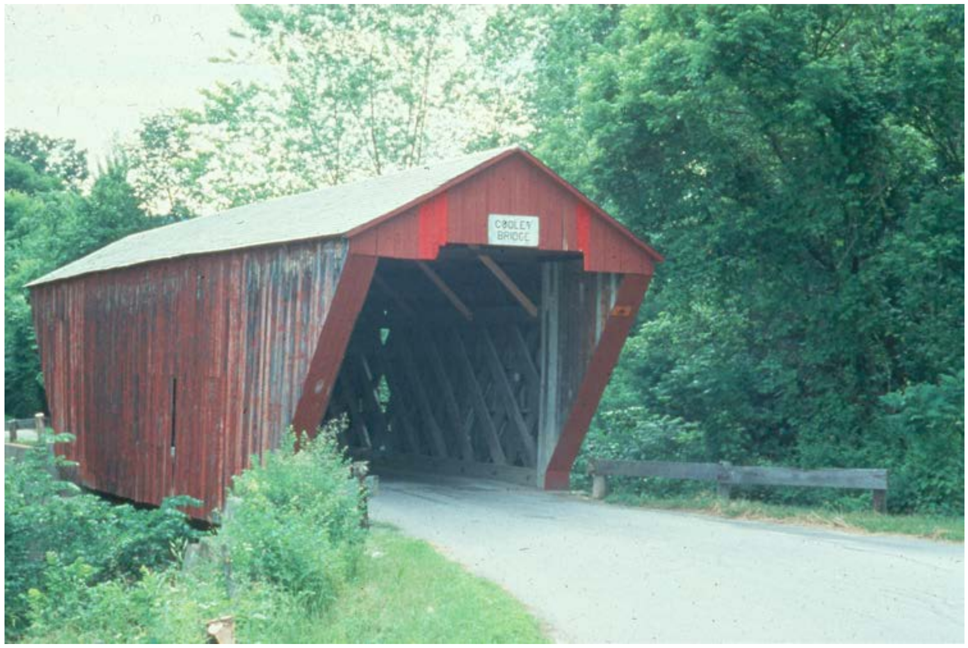

Figure 8.1.5 Town Truss Covered Bridge

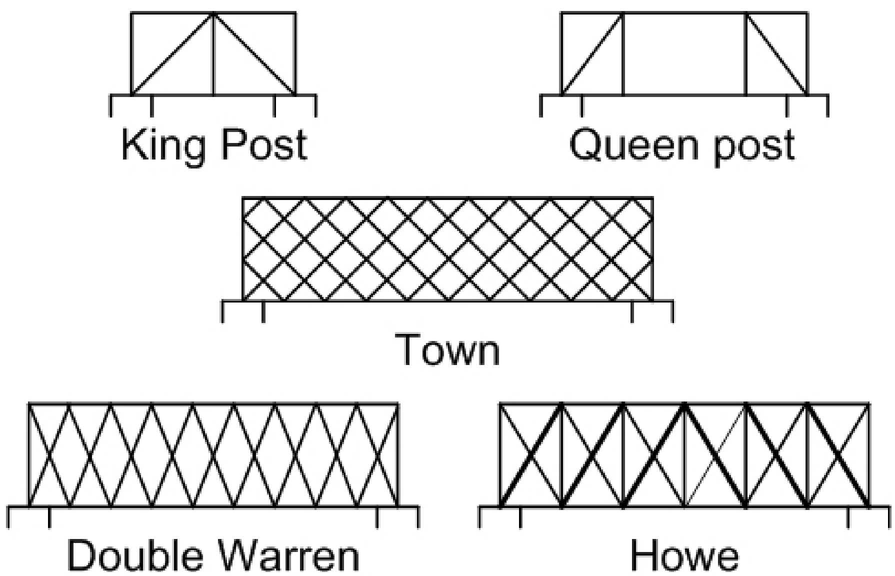

Figure 8.1.6 Common Covered Bridge Trusses

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 8.1.7, 8.1.8 and 8.1.9). The arch served as the main supporting element, and the king posts simply strengthened the arch. The span lengths for Burr-arch truss bridges generally range from 50 to 175 feet. Because of their greater strength, many of these structures still exist today.

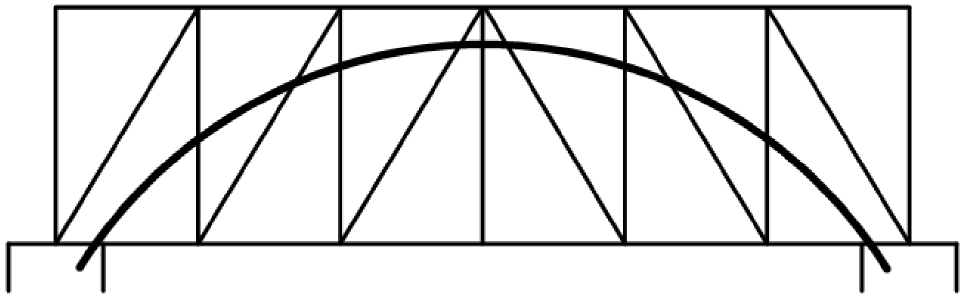

Figure 8.1.7 Schematic of Burr Arch-truss Covered Bridge

Figure 8.1.8 Burr Arch-truss Covered Bridge

Figure 8.1.9 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 8.1.2). 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 Figures 8.1.9 and 8.1.10). 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 8.1.10 Town Truss Design

8.1.3 Overview of Common Deficiencies

Common defeciencies that occur on solid sawn timber beams include:

- Inherent defects - Checks, splits, shakes, and knots
- Decay by fungi
- Damage by insects and borers
- Loose connections
- Surface depressions
- Damage from fire
- Damage from impact/collisions
- Damage from wear, abrasion, and mechanical wear
- Damage from overstress
- Damage from weathering/warping
- Failure of protective system

A less common deficiency that may be encountered by the inspector includes damage from chemical attack. Refer to Topic 6.1 for a more detailed presentation of the properties of timber, types and causes of timber deterioration, and the examination of timber.

8.1.4 Inspection Methods and Locations

Inspection methods to determine other causes of timber deterioration are discussed in detail in Topic 6.1.7.

Methods

Visual

The inspection of timber for checks, splits, cracks, shakes, fungus decay, deflections, crushing, delaminations, and loose connections is primarily a visual activity.

Physical

The physical examination of a timber 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 Methods

Several advanced methods are available for timber inspection. Nondestructive methods, described in Topic 15.1.2, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 15.1.3, include:

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

Locations

Bearing Areas

Check the bearing areas for crushing of the beams near the bearing seat (see Figure 8.1.11). 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 (see Topic 11.1).

Figure 8.1.11 Bearing Area of Typical Solid Sawn Beam

Shear Zones

As discussed in Topic 5.1, 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 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 8.1.12). Measure these cracks carefully for length, width, and if possible, the depth.

Figure 8.1.12 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. 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 because the tensile stress and horizontal shear stress combine to split the grain apart.

Areas Exposed to Drainage

Timber bridges with plank decks are exposed to drainage throughout the length of the span. 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 any fastener location.

(see Figure 8.1.13).

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, examine the drill hole area carefully for proper preservation treatment and dowel plug installations.

Figure 8.1.13 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. Perform further probing or drilling 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.

Areas Previously Repaired

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 8.1.14). Deteriorated secondary members may indicate problems in the primary members.

Figure 8.1.14 Typical Timber End Diaphragm

Fasteners and Connectors

Check the fasteners (e.g., nails, screws, bolts, and deck clips) for corrosion. Also inspect for loose or missing fasteners. Check for moisture and decay around the holes.

8.1.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

In an element level condition state assessment of a solid sawn timber bridge, possible AASHTO National Bridge Elements (NBEs) are:

NBE No.	Description
Superstructure	
111	Timber Girder/Beam
117	Timber Stringer
135	Timber Truss
146	Timber Arch
156	Timber Floorbeam

The unit quantity for the timber superstructures is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flag is applicable in the evaluation of solid sawn timber superstructures:

Defect Flag No.	Description
362	Superstructure Traffic Impact (load capacity)

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

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Topic 8.2 Glulam Timber Bridges

8.2.1 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 to fabricate these members. 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 3/4 to 1-1/2 inches thick (see Figures 8.2.1 and 8.2.2).

Figure 8.2.1 Elevation View of a Glulam Multi-beam Bridge

Figure 8.2.2 Underside View of a Glulam Multi-beam Bridge

8.2.2 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 8.2.1). They usually support a deck consisting of glulam panels with a bituminous wearing surface. Beam sizes typically range from 6 inches by 24 inches to 12-1/4 inches by 60 inches, and the beams are usually spaced 5'-6" to 6'-6" on center (see Figure 8.2.2).

These more modern multi-beam bridges can typically be used in spans of up to 80 feet, although some span as long as 150 feet have been constructed. This beam type can be used to form longer multiple span structures. They are generally found on local and secondary roads, as well as in park settings.

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, which are supported by the trusses (see Figures 8.2.3 and 8.2.4). 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 150 to 250 feet (see Figure 8.2.5).

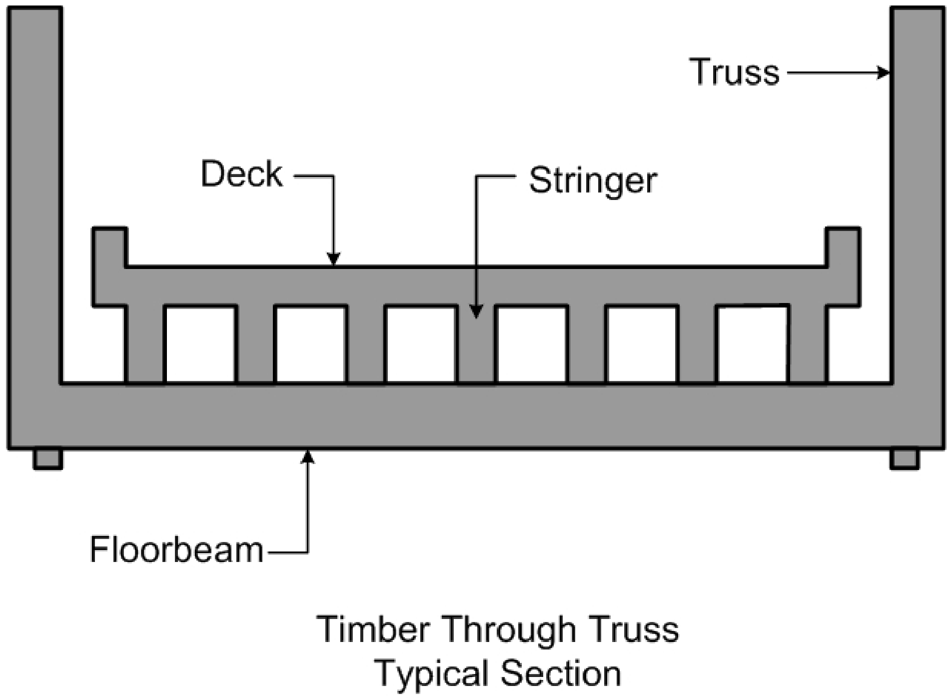

Figure 8.2.3 Timber Through Truss Typical Section

Figure 8.2.4 Bowstring Truss Pedestrian Bridge

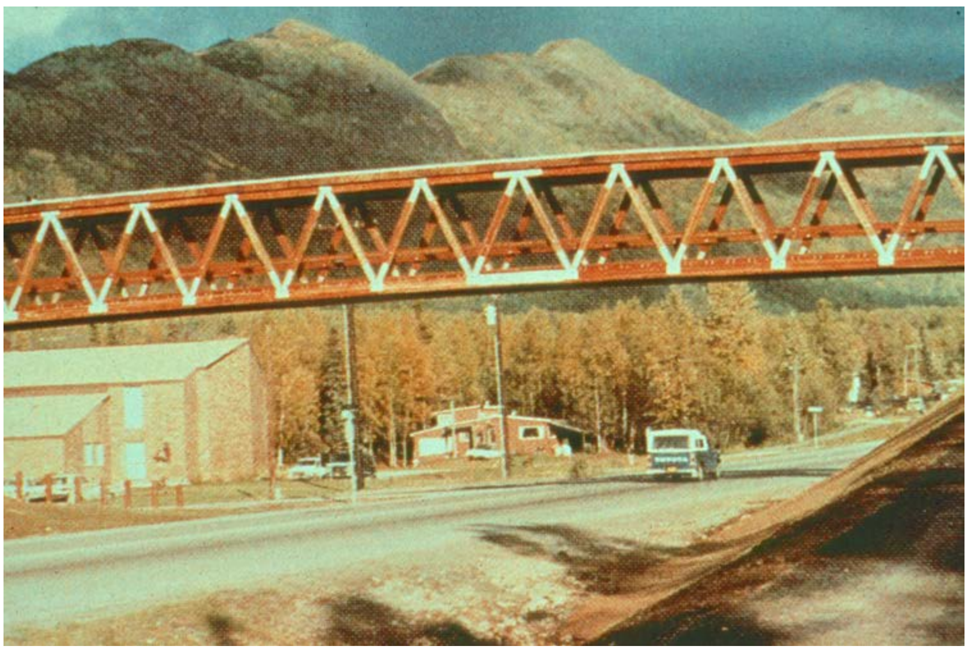

Figure 8.2.5 Parallel Chord Truss Pedestrian Bridge (Eagle River, Alaska)

Arch Bridges

Glulam arch bridges usually consist of two- or three-hinged deck arches, which support a glulam deck and floor system (see Figures 8.2.6 and 8.2.7). Glulam arches are practical for spans of up to about 300 feet. Arches are used in locations such as parks where aesthetics is important.

Figure 8.2.6 Glulam Arch Bridge over Glulam Multi-beam Bridge (Keystone Wye interchange, South Dakota)

Figure 8.2.7 Glulam Arch Bridge (West Virginia)

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 Figure 8.2.8). Due to the larger depth of the glulam beams, diaphragms or cross bracing are normally 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, spandrel bents and hangers. The secondary members include the diaphragms and cross bracing between the stringers and the lateral bracing between the arch or truss.

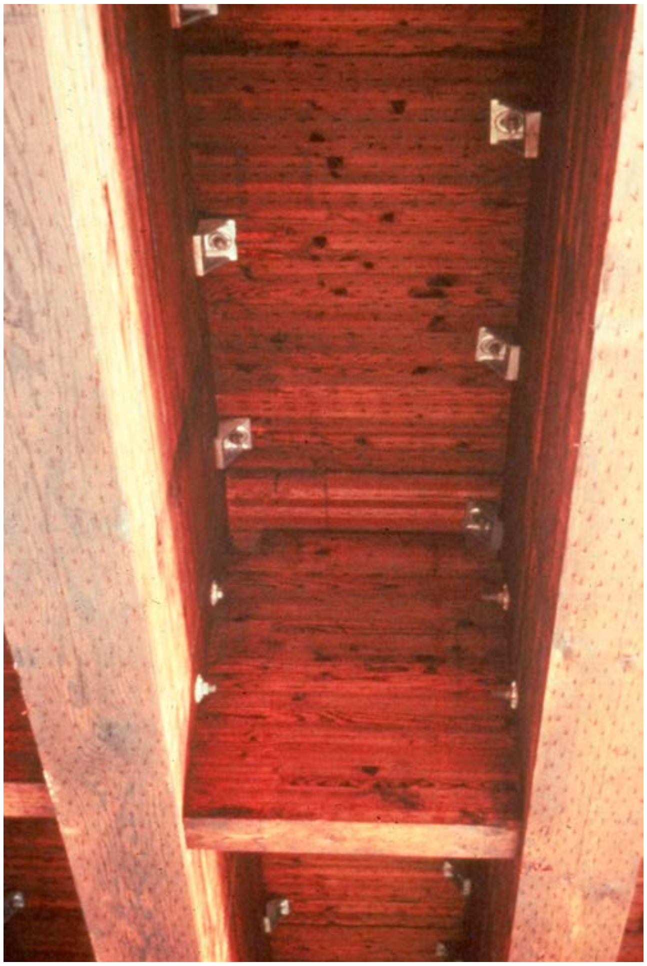

Figure 8.2.8 Typical Glulam Diaphragm

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 improve the mechanical properties of timber bridges.

8.2.3 Overview of Common Deficiencies

Common deficiencies that occur on glulam timber beams include:

- Inherent deficiency - Checks, splits, shakes
- Decay by fungi
- Damage by insects and borers
- Delaminations
- Loose connections
- Surface depressions
- Damage from fire

- Damage from impact/collisions
- Damage from wear, abrasion, and mechanical wear
- Damage from overstress
- Damage from weathering/warping
- Failure of protective system

A less common deficiency that may be encountered by the inspector includes damage from chemical attack. Refer to Topic 6.1 for a more detailed presentation of the properties of timber, types and causes of timber deterioration, and the examination of timber.

8.2.4 Inspection Methods and Locations

Inspection methods to determine other causes of timber deterioration are discussed in detail in Topic 6.1.7. The inspection locations and procedures for glulam bridges are similar to those for solid sawn bridges.

Methods

Visual

The inspection of timber for checks, splits, cracks, shakes, fungus decay, deflections, crushing, delaminations, and loose connections is primarily a visual activity.

Physical

The physical examination of a timber 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 Methods

Several advanced methods are available for timber inspection. Nondestructive methods, described in Topic 15.1.2, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 15.1.3, include:

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

Locations

Bearing Areas

Inspect the bearing areas for crushing of the beams (see Figure 8.2.9). 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 (see Topic 11.1).

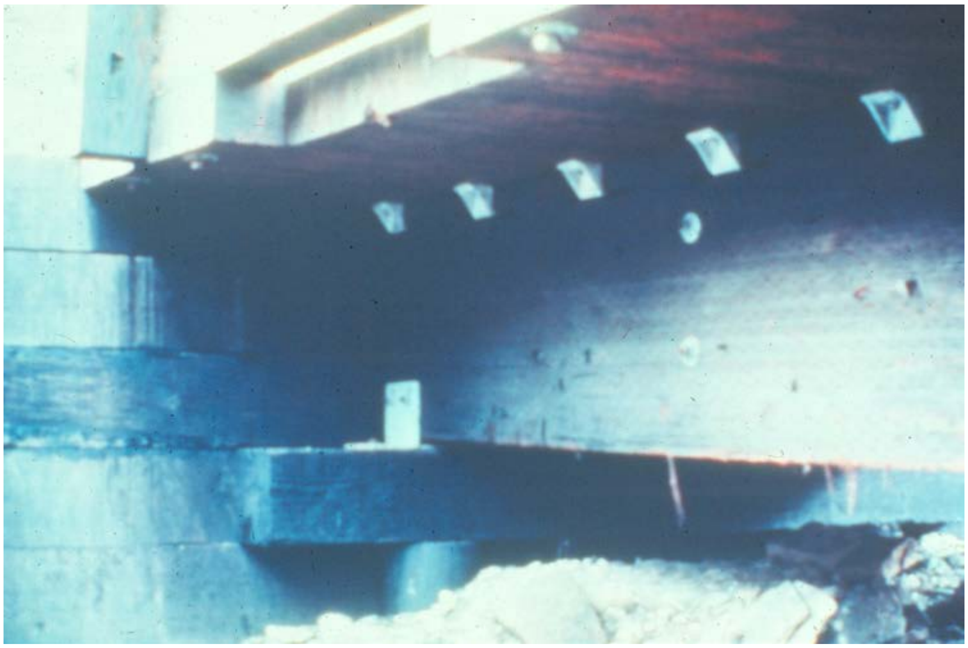

Figure 8.2.9 Bearing Area of Typical Glulam Beam

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 8.2.10). Delaminations that extend completely through the cross section of the member are considered severe 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 near the top or bottom of the beam. Delaminations directly through a connector are also undesirable.

Figure 8.2.10 Close-up View of Glulam Bridge Showing Laminations

Tension Zones

Examine the zone of maximum tension for signs of structural distress (see Figure 8.2.11). 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. Inspect for excessive deflection or sagging in the beams.

Figure 8.2.11 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 8.2.12). 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 8.2.12 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. Perform further probing or drilling 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.

Areas Previously Repaired

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

Secondary Members

Examine diaphragms for decay, fire damage, and insect damage (see Figure 8.2.13). Check steel cross bracing for corrosion, bowing, or buckling (see Figure 8.2.14). Examine connections for tightness, cracks and splits, and corroded, loose, or missing fasteners. Deteriorated secondary members may indicate problems in the primary members.

Figure 8.2.13 Typical Diaphragm for a Glulam Multi-beam Bridge

Fasteners and Connectors

Inspect any fastener for corrosion, tightness, and missing parts (see Figure 8.2.13).

Figure 8.2.14 Glulam Beams with Numerous Fastener Locations

8.2.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

In an element level condition state assessment of a glulam timber bridge, possible AASHTO National Bridge Elements (NBEs) are:

NBE No.	Description
Superstructure	
111	Timber Girder/Beam
117	Timber Stringer
135	Timber Truss
146	Timber Arch
156	Timber Floorbeam

The unit quantity for the timber superstructures is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flag is applicable in the evaluation of glulam timber superstructures:

Defect Flag No.	Description
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 8
Inspection and Evaluation of Timber Superstructures

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Topic 8.3 Stress-Laminated Timber Bridges

8.3.1 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 8.3.1). The compression induced frictional resistance within the timber laminations is the mechanism that makes this structural system effective.

Figure 8.3.1 Stress-Laminated Timber Slab Bridge Carrying a 90,000-Pound Logging Truck (Source: Barry Dickson, West Virginia University)

8.3.2 Design Characteristics

Loss of the compressive stress reduces the frictional resistance between members and reduces the load capacity of this structural system.

Stress-Laminated Timber Slab Bridges

Stress-laminated timber slab bridges can be used for simple spans of up to 50 feet and are capable of carrying modern highway loadings (see Figures 8.3.1 and 8.3.3). Stressed deck bridges have also been constructed using glulam members. Combining glulam technology with stress-lamination increases practical span lengths to 65 feet (see Figure 8.3.4).

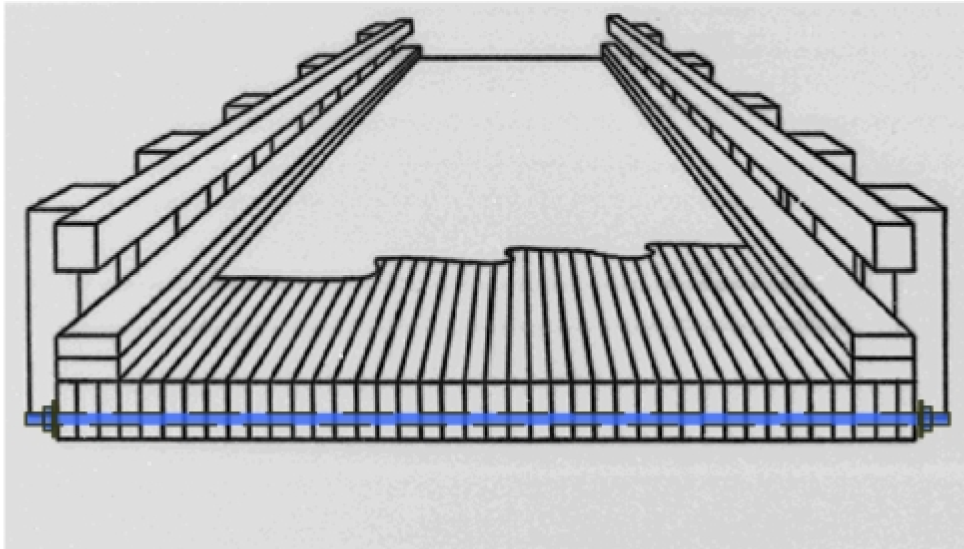

Figure 8.3.2 Typical Section of a Stress-Laminated Timber Slab Bridge

Figure 8.3.3 Stress-Laminated Timber Slab Bridge

Figure 8.3.4 Glulam Stress-Laminated Timber 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 8.3.5). 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 75 feet long and has stressing rods spaced at two feet. This bridge has performed well, which encouraged longer span lengths to be constructed. The average span length for stress-laminated tee beam bridges ranges between 25 and 85 feet (see Figure 8.3.6).

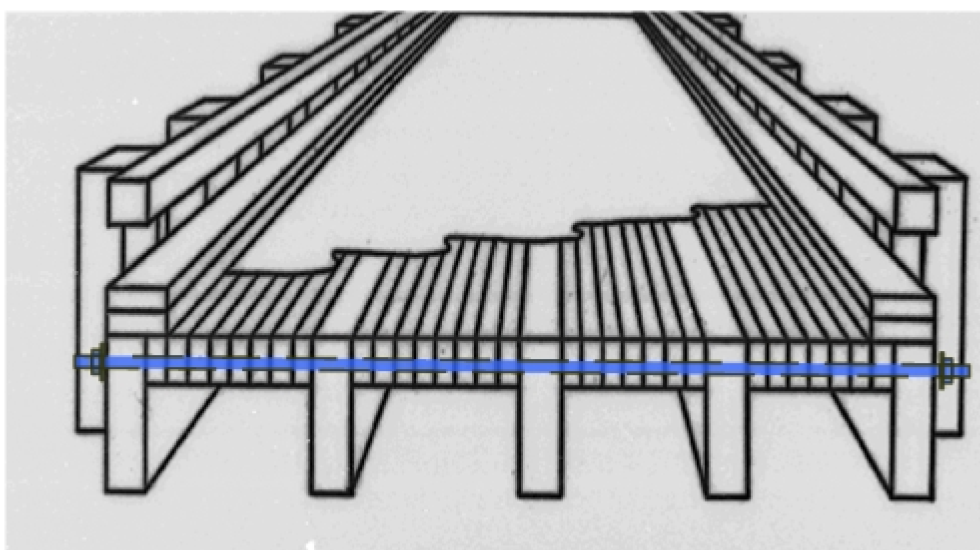

Figure 8.3.5 Typical Section of a Stress-Laminated Timber Tee Beam Bridge (Source: Barry Dickson, West Virginia University)

Figure 8.3.6 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 8.3.7). This bridge type is also known as a cellular stressed deck. Average span lengths range between 35 and 65 feet, with longer maximum spans (see Figure 8.3.8).

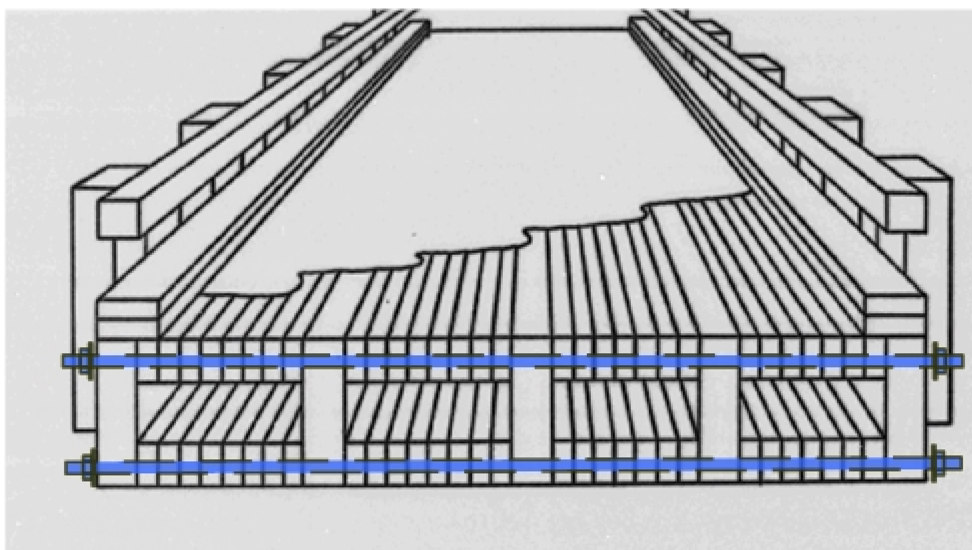

Figure 8.3.7 Typical Section of a Stress-Laminated Timber Box Beam (Source: Barry Dickson, West Virginia University)

Figure 8.3.8 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 8.3.9). This bridge type has been used for a bridge with a total length of excess of 100 feet, and it has a potential for center span lengths over 50 feet.

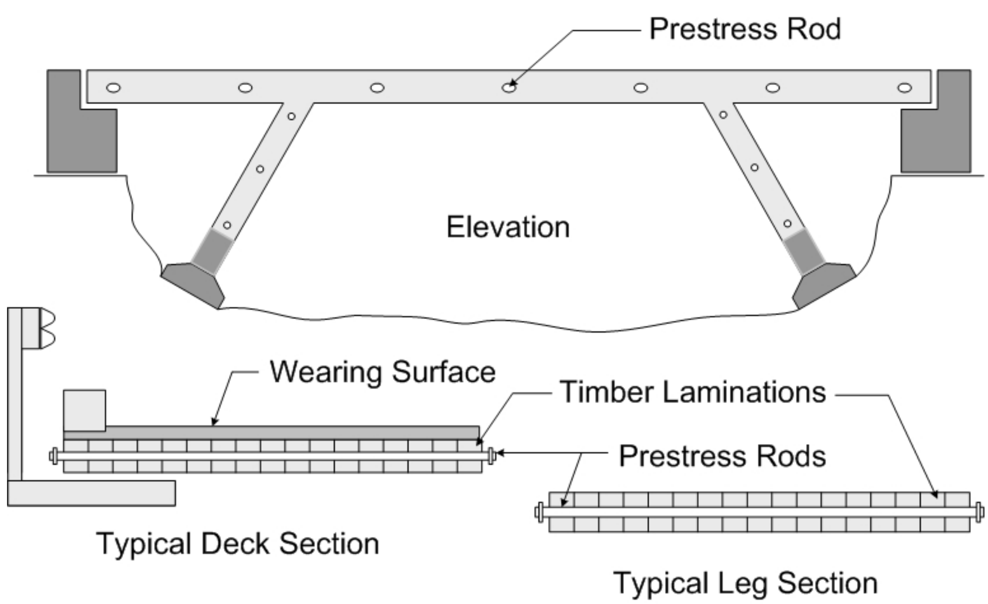

Figure 8.3.9 Stress-Laminated Timber K-frame Bridge

Primary and Secondary Members

The primary members are the decks, slabs, tee beams, box beams, and frame legs. The secondary members are the external diaphragms and cross bracing between beams.

8.3.3 Overview of Common Deficiencies

Common deficiencies that occur on stressed timber bridges include:

- Inherent defects - Checks, splits, shakes
- Decay by fungi
- Damage by insects and borers
- Loose connections
- Loose or deteriorated stressing rods or connectors
- Surface depressions
- Damage from fire
- Damage from impact/collisions
- Damage from wear, abrasion, and mechanical wear
- Damage from overstress
- Damage from weathering/warping
- Failure of protective system

A less common deficiency that may be encountered by the inspector includes damage from chemical attack. Refer to Topic 6.1 for a more detailed presentation of the properties of timber, types and causes of timber deterioration, and the examination of timber.

8.3.4 Inspection Methods and Locations

Inspection methods to determine other causes of timber deterioration are discussed in detail in Topic 6.1.7. The inspection of stress-laminated timber bridges is similar to those for glulam bridges.

Methods

Visual

The inspection of timber for checks, splits, cracks, shakes, fungus decay, deflections, crushing, delaminations, and loose connections is primarily a visual activity.

Physical

The physical examination of a timber 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 Methods

Several advanced methods are available for timber inspection. Nondestructive methods, described in Topic 15.1.2, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 15.1.3, include:

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

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 or loose rods. This may be observed when the bridge is subject to a moving live load.

Figure 8.3.10 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 (see Topic 11.1, 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 8.3.11). Delaminations that extend completely through the cross section of the member are the most severe since this makes the member act as two smaller members.

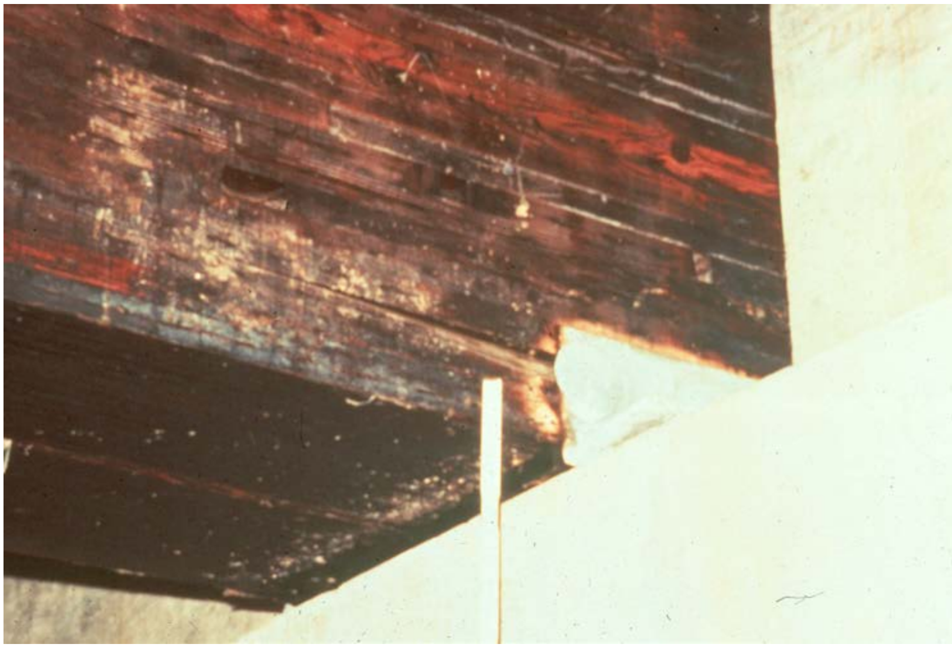

Figure 8.3.11 Close-up View of End of a Stress-Laminated Timber Bridge Showing Laminations

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. 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. Examine the curb line areas. Standing water soaks into the beam and corrodes the stressing rods.

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. Perform further probing or drilling in suspect areas.

Areas Exposed to Traffic

Check stress-laminated timber members for collision damage from vehicles passing below.

Areas Previously Repaired

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. Deteriorated secondary members may indicate problems in the primary members.

Fasteners and Connectors

Inspect any fastener for corrosion, tightness, and missing parts. Stressing rod hardware is the most important fastener system on a stress-laminated timber bridge.

8.3.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* used for element level condition state assessment.

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

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

NBE No.	Description
Deck/Slabs	
31	Timber Deck
54	Timber Slab
Superstructure	
111	Timber Girder/Beam
BME No.	
Wearing Surfaces and Protection Systems	
520	Deck/Slab Protection System

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

The following Deflect Flag is applicable in the evaluation of stress-laminated timber superstructures:

Deflect Flag No.	Description
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 9
Inspection and Evaluation of Concrete Superstructures

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Chapter 9
Inspection and Evaluation of Concrete Superstructures

Topic 9.1 Cast-in-Place Slabs

9.1.1 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 9.1.1 and 9.1.2). 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 and distributes loads to the superstructure. 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 methods and locations and, evaluation, are similar. See Topic 7.2 for further details.

Figure 9.1.1 Typical Simple Span Cast-in-Place Slab Bridge

Figure 9.1.2 Typical Multi-Span Cast-in-Place Slab Bridge

9.1.2 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 or more simply supported spans and spans are typically less than 30 feet long. Continuous multi-span slab bridges are also in service, but not as common as simply supported slabs.

Primary and Secondary Members

The only primary member in a cast-in-place slab bridge is the slab itself. There are no secondary members.

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 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 tension caused by negative bending moments.

Shear reinforcement is also considered to be primary reinforcement. Shear reinforcement, if required, is normally obtained by bending the tension bars at a 45 degree angle close to the slab supports. The shear reinforcement is perpendicular to diagonal tension/shear forces and therefore resists those forces.

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 9.1.3).

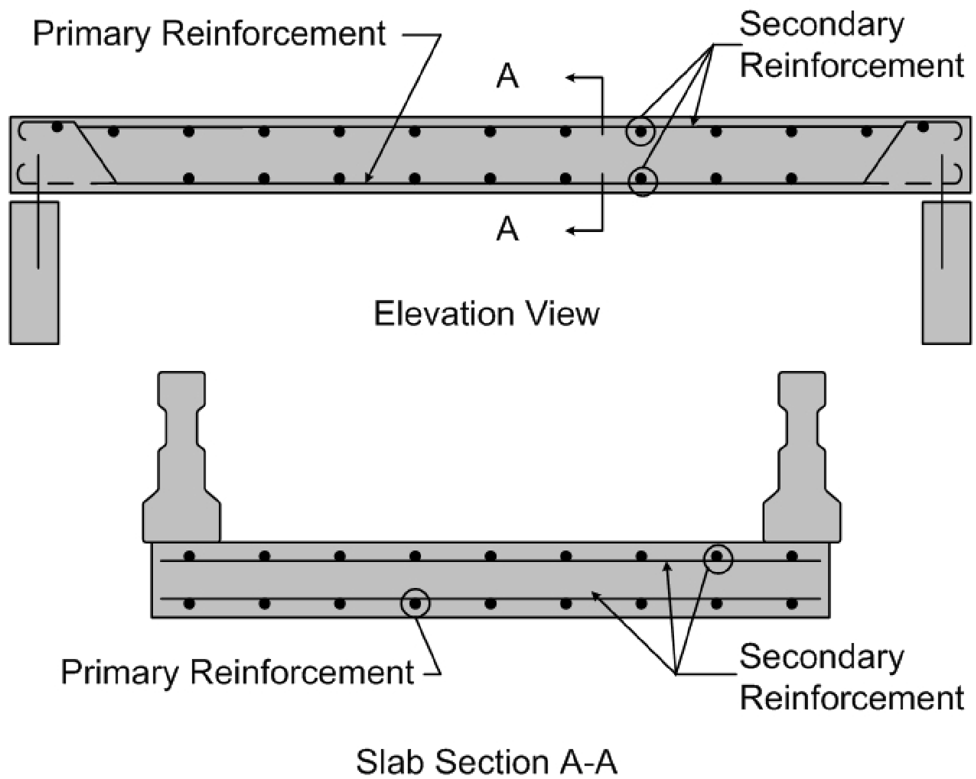

Figure 9.1.3 Steel Reinforcement in a Simply Supported Concrete Slab

9.1.3 Overview of Common Deficiencies

Common deficiencies that occur on cast-in-place slab bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear

- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Carbonation

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.1.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of concrete slabs for surface cracks, spalls, wear, and other deficiencies is primarily a visual activity.

Physical

The most common physical method for inspecting concrete is to use a hammer to "sound" the concrete. However, the physical examination of the top surface of the slab with a hammer can be a tedious operation. Instead, a chain drag is used to determine delaminated areas for most cases. A chain drag is typically 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 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. It is not possible to use a chain drag on the bottom surface, so hammer sounding is the primary physical procedure.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection.

Nondestructive methods, described in Topic 15.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Electrical methods
- Delamination detection machinery
- Ground-penetrating radar
- Electromagnetic methods
- Pulse velocity
- Flat jack testing
- Impact-echo testing
- Infrared thermography
- Laser ultrasonic testing

- Magnetic field disturbance
- Neutron probe for detection of chlorides
- Nuclear methods
- Pachometer
- Rebound and penetration methods
- Ultrasonic testing
- Smart concrete
- Carbonation

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Examine bearing areas for cracking, delamination, and spalling where friction from thermal movement and high edge or bearing pressure could overstress the concrete (see Figure 9.1.4). Check the condition and operation of any bearing devices.

As detailed in Topic 7.2.6, bearing areas are located where one element "sits" on top of another element. For slabs, the bearing areas are located above the substructure supports.

Figure 9.1.4 Bearing Area: Cast-in-Place Slab

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 an overstress for shear (see Figures 9.1.5 and 9.1.6). Carefully measure cracks as they may represent lost shear capacity.

Figure 9.1.5 Diagonal Shear Cracks Close to the Ends of a Slab Bridge

Figure 9.1.6 Shear Zone on the Underside of a Continuous Slab Bridge Near a Pier

Tension Zones

Examine tension zones for flexure cracks, which would be perpendicular to the tensile forces and 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 than 1/16 inch wide indicate overstress due to tensile forces caused by bending stresses. Check for efflorescence from cracks and, more significant, the 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 Figure 9.1.7).

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 9.1.8). Slab bridges which use hooks to develop the primary tension reinforcement are not as susceptible to debonding due to deterioration of the concrete.

Secondary Members

The slab bridge has no secondary members.

Figure 9.1.7 Inspector examining and documenting deficiencies in concrete slab

Figure 9.1.8 Concrete Slab Tension Zone: Delamination, Efflorescence, Rust, and Stains

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 9.1.9).

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. Also examine the top surface for signs of wear caused by vehicular traffic.

Areas Previously Repaired

Examine areas that have been previously repaired. Determine if the repairs are in place, and they are functioning properly.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Acute Angles on Skewed Bridges

Examine skewed bridges for lateral displacement and cracking of acute corners due to point loading and insufficient reinforcement.

Figure 9.1.9 Deteriorated Slab Fascia due to Roadway Deicing Agents

Camber

Using a string line, check for vertical alignment (camber) changes from the as-built condition of the slab. Downward deflection usually indicates reduced capacity. Upward deflection usually indicates shrinkage.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the slab. Cracking will typically be transverse in the thinner regions of the slab and longitudinal near changes in cross section thickness.

9.1.5 Evaluation

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

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBI component condition rating guidelines. For a slab bridge, these guidelines must be applied for both the deck component and the superstructure component.

Use previous inspection data along with current inspection findings to determine the correct component condition rating. For this type of structure, the deck and superstructure components may have the same rating.

Element Level Condition State Assessment

In an element level condition state assessment of a cast-in-place slab bridge, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Decks/Slabs	
38	Reinforced Concrete Slab* * Note that this element designation is used regardless of the type of riding surface

BME No.	Description
Wearing Surfaces and Protection Systems	
510	Wearing Surfaces
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

The unit quantity for decks, slabs, wearing surfaces, protection systems and protective coatings is square feet. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge

Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of cast-in-place slab superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 9

Inspection and Evaluation of Concrete Superstructures

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Topic 9.2 Cast-In-Place Tee Beams

9.2.1 Introduction

The concrete tee beam, a predominant bridge type during the 1930's and 1940's, is generally a cast-in-place monolithic deck and stem system formed in the shape of the letter "T" (see Figure 9.2.1).

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 (see Topic 9.8).

Figure 9.2.1 Simple Span Tee Beam Bridge

9.2.2 Design Characteristics

General

Care must be taken not to describe tee beam bridges as a composite structure. They do not meet the definition of composite, because the deck and stem are constructed of the same material. The deck portion of the beam is constructed to act integrally with the stem, providing greater stiffness and allowing increased span lengths (see Figure 9.2.2). Span lengths for tee beam bridges are typically between 30 and 50 feet.

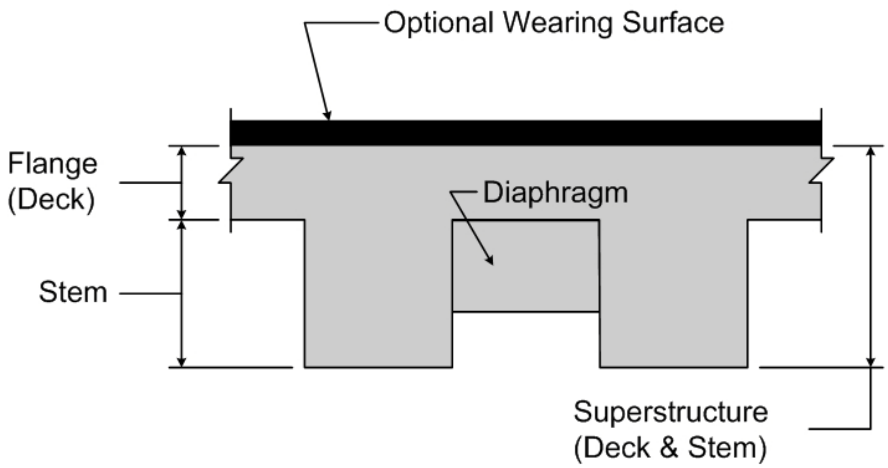

Figure 9.2.2 Tee Beam Cross Section

Spacing of the tee beams is generally 3 to 8 feet, center-to-center of beam stems. The depth of the stems is generally 18 to 40 inches. Simple span design was most common but continuous span designs were popular in some regions (see Figure 9.2.2). A 3 or 4 inch fillet at the deck-stem intersection identifies this older form of construction.

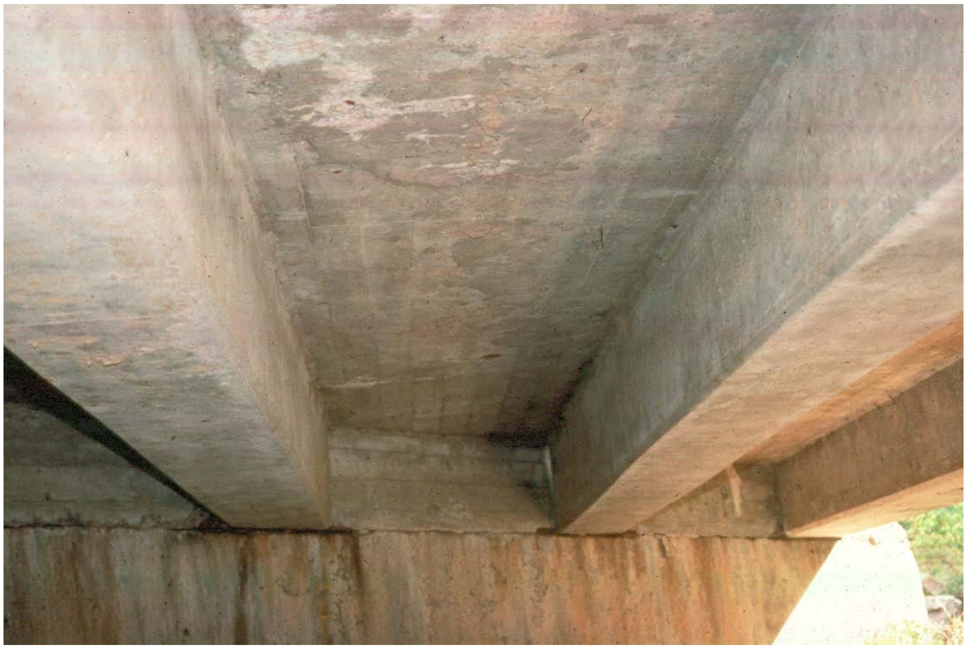

Figure 9.2.3 Typical Tee Beam Layout

It is important to 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 (see Figure 9.2.4). A spall on the bottom of the stem can also indicate if there are steel reinforcement bars of a tee beam or the bottom flange of a steel I-beam.

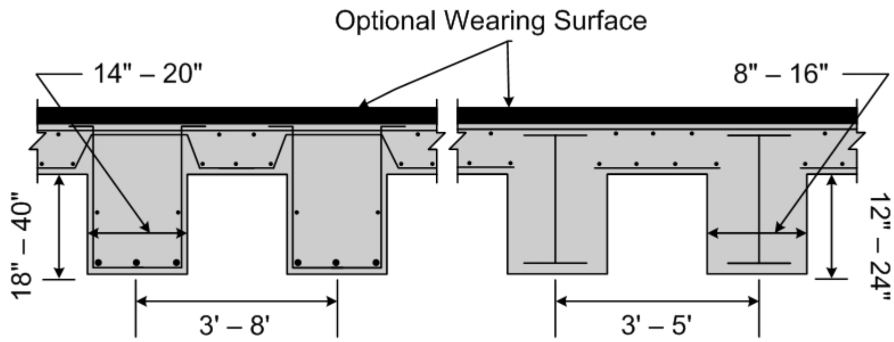

Figure 9.2.4 Comparison Between Tee Beam and Concrete Encased Steel I-beam

Figure 9.2.5 Concrete Encased Steel I-beam

Primary and Secondary Members

The primary members of a tee beam bridge are the tee beam stem (web) and deck (flange) (see Figure 9.2.6).

Diaphragms are the only secondary members on a cast-in-place tee beam bridge. End diaphragms 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 (see Figure 9.2.6).

Figure 9.2.6 Tee Beam Primary and Secondary Members

Steel Reinforcement

The primary reinforcing steel consists of main tension reinforcement located longitudinally and shear reinforcement in the form of stirrups. The main tension reinforcement is located in the bottom of the beam stem to resist tensile forces caused by positive moment (see Figure 9.2.7). If the concrete tee beams are continuous, there will be additional longitudinal reinforcement close to the top surface of the deck over the piers to resist tensile forces caused by negative moment. 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 9.2.7). The need for stirrups is greatest near the beam supports where shear stresses are the highest. Stirrup spacing is typically smaller in the stem close to the substructure supports.

The secondary (temperature and shrinkage) reinforcing steel for the stem is oriented longitudinally in the sides (see Figure 9.2.7). The primary and secondary reinforcing steel for the deck portion of the beam is the same as for a standard concrete deck (see Figure 9.2.7). Tension and shear reinforcement are transverse while temperature and shrinkage reinforcement are longitudinal.

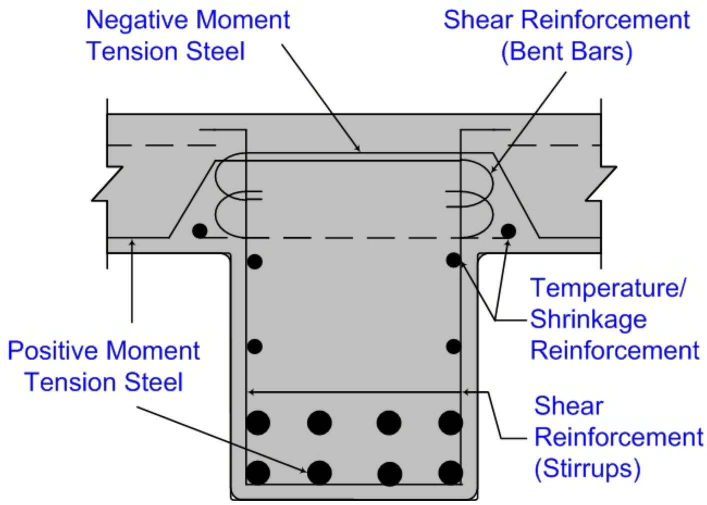

Figure 9.2.7 Steel Reinforcement in a Concrete Tee Beam

9.2.3 Overview of Common Deficiencies

Common deficiencies that occur on concrete tee beam bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Carbonation

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.2.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of concrete tee beams for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

The physical examination of a tee beam with a hammer can be a tedious operation. In most cases, a chain drag is used to determine delaminated areas on the top flange or deck surface. 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. Hammer sounding is used to examine the stem and bottom surface of the flange.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection.

Nondestructive methods, described in Topic 15.2.2, include:

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

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Examine bearing areas for cracking, delamination and spalling where friction from thermal movement and high bearing pressure could overstress the concrete. Check for crushing of the stem near the bearing seat. Check the condition and operation of any bearing devices (see Figures 9.2.8 through 9.2.11).

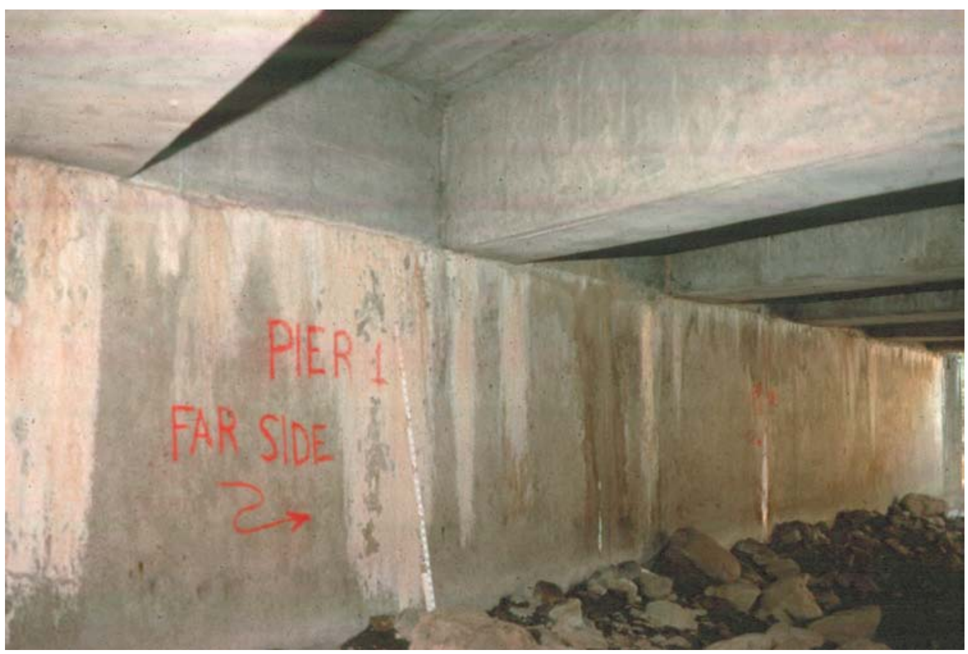

Figure 9.2.8 Bearing Area of Typical Cast-in-Place Concrete Tee Beam Bridge

Figure 9.2.9 Spalled Tee Beam End

Figure 9.2.10 Deteriorated Tee Beam Bearing Area

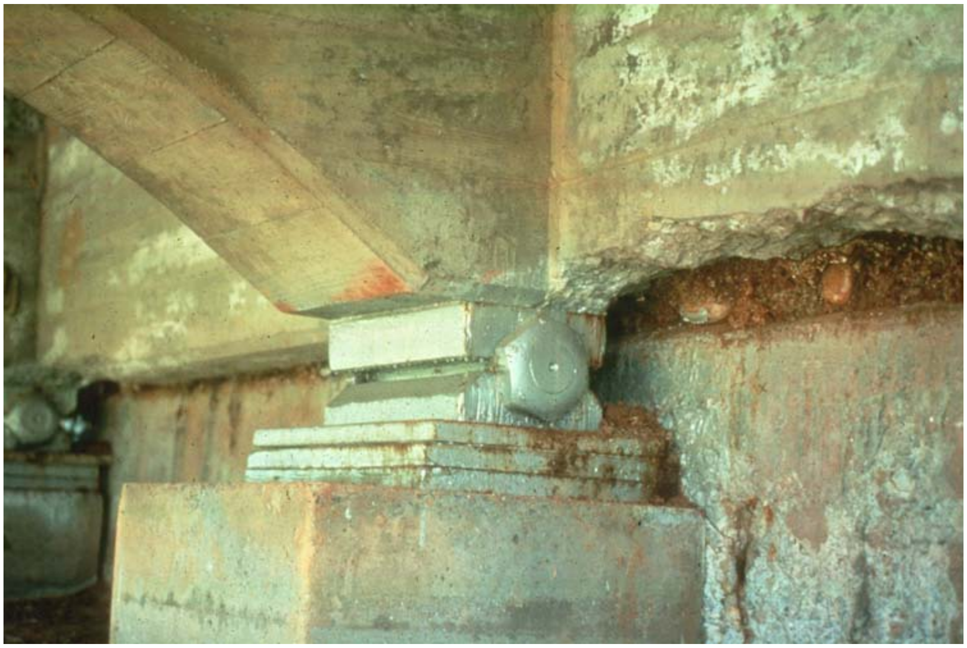

Figure 9.2.11 Steel Bearing Supporting a Cast-in-Place Concrete Tee Beam

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. Carefully measure cracks, as they may represent lost shear capacity (see Figure 9.2.12).

Figure 9.2.12 Shear Zone of Cast-in-Place Concrete Tee Beam Bridge

Tension Zones

Examine tension zones for flexure cracks, which would be vertical on the sides and transverse across the bottom of the stem (see Figure 9.2.13). 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 deck over the piers for continuous spans (see Figure 9.2.14). Cracks greater than 1/16 inch wide indicate overstress due to tensile forces caused by bending stresses. Check for efflorescence from cracks and discoloration of the concrete caused by rust stains from the reinforcing steel (see Figure 9.2.15). 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 9.2.16).

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 Figures 9.2.15 and 9.2.16).

Check for efflorescence from cracks and, and more significant, the 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. Document the remaining cross section of reinforcing steel since section loss will decrease live load capacity.

Figure 9.2.13 Flexure Cracks on a Tee Beam Stem

Figure 9.2.14 Flexure Cracks in Tee Beam Flange/Deck

Figure 9.2.15 Stem of a Cast-in-Place Concrete Tee Beam with Cracking and Efflorescence

Figure 9.2.16 Spall on the Bottom of the Stem of a Tee Beam with Corroded Main Steel Exposed

Secondary Members

Inspect diaphragms for flexure and shear cracks, as well as for typical concrete deficiencies. Deficiencies in the diaphragms may be an indication of differential settlement of the substructure or differential deflection of stems of the 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 (see Figure 9.2.17).

Check the deck soffit (underside of the bridge deck) for cracking, spalling and delaminations.

Check the exterior girder deck overhang for rotation, longitudinal cracking, settlement or misalignment of the rail. Cracking will occur parallel to the curb line (the negative moment for the deck portion).

Check around scuppers or drain holes and deck or stem fascias for deteriorated concrete (see Figure 9.2.18).

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 deck joints (see Figure 9.2.19).

Figure 9.2.17 Asphalt Covered Tee Beam Deck

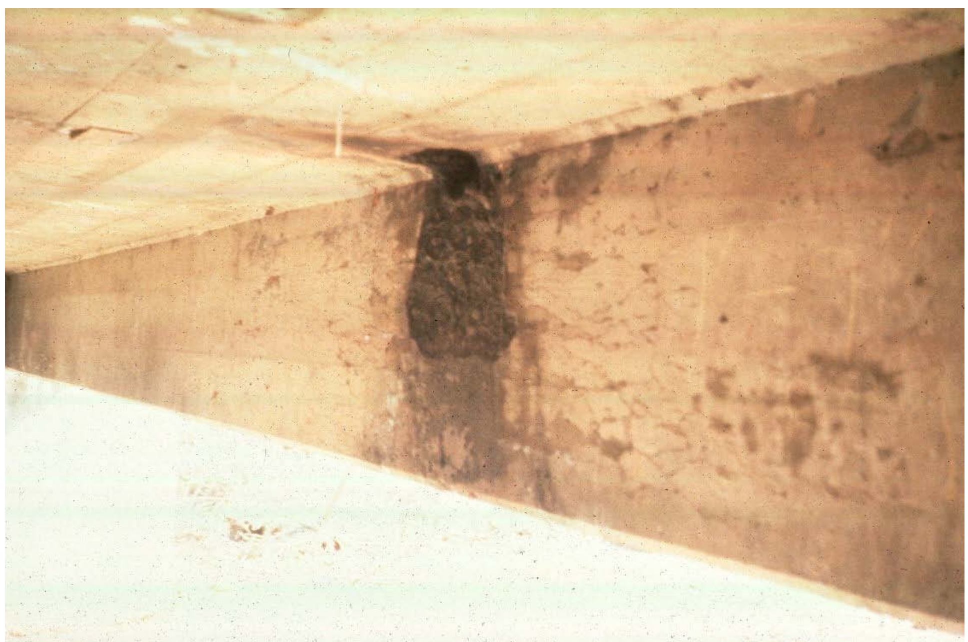

Figure 9.2.18 Deteriorated Tee Beam Stem Adjacent to Drain Hole

Figure 9.2.19 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 with section loss as well as the spalled and delaminated concrete. The loss of concrete due to such an accident is not always serious, unless the bond between the concrete and steel reinforcement is affected (see Figure 9.2.20). If the top flange is exposed to traffic, examine for signs of wear, especially at the wheel path locations.

Figure 9.2.20 Collision Damage to Tee Beam Bridge Over a Highway

Areas Previously Repaired

Examine areas that have been previously repaired. Determine if the repairs are in place, and if they are functioning properly.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Camber

Using a string line, check for vertical alignment (camber) changes from the as-built condition of the tee-beam. Downward deflection usually indicates reduced capacity. Upward deflection usually indicates shrinkage.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the tee-beam. Cracking will typically be transverse in the thinner regions of the tee-beam (such as the deck) and longitudinal near changes in cross section thickness.

9.2.5 Evaluation

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

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-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 (Items 58 and 59) for additional details about NBI component condition rating guidelines.

Use previous inspection data along with current inspection findings to determine the correct component condition rating. For concrete tee beams, the deck acts as the top flange and influences the superstructure NBI component condition rating (see Figure 9.2.21). When the deck component condition rating is 4 or less, the superstructure component condition rating may be reduced if the recorded deck defects reduce its ability to carry applied stresses associated with superstructure moments.

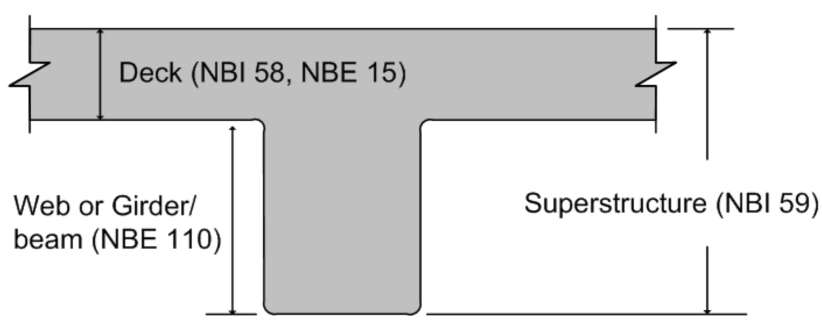

Figure 9.2.21 Components/Elements for Evaluation

Element Level Condition State Assessment

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

NBE No.	**Description**
Decks/Slabs	
15	Prestressed/Reinforced Concrete Top Flange* * Note that this element designation is used regardless of the type of riding surface.
Superstructure	
110	Reinforced Concrete Girder/Beam

BME No.	**Description**
Wearing Surfaces and Protection Systems	
510	Wearing Surfaces
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

The unit quantity for the top flange (deck), wearing surfaces, protection systems and protective coatings is square feet. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. For the beam (NBE No. 110), the evaluation is based solely on the web for tee beams. The unit quantity for the beam is feet, and the total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of tee-beam superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 9
Inspection and Evaluation of Concrete Superstructures

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Topic 9.3 Conventionally Reinforced Concrete Two-Girder System

9.3.1 Introduction

Concrete two-girder bridges generally consist of cast-in-place monolithic decks supported by a two-girder system. Concrete girders can be used as deck girders, where the deck is cast on top of the girders (Figure 9.3.1), or as through girders, where the deck is cast between the girders (see Figure 9.3.2). Through girders are very large in appearance and actually serve as the bridge's parapets, as well as the main supporting members. Many of the concrete deck and through two-girder bridges in service today were built in the 1940's.

Figure 9.3.1 Concrete Deck Two-Girder Bridge

Figure 9.3.2 Concrete Through Two-Girder Bridge

9.3.2 Design Characteristics

General

For the purpose of inspection, the deck 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. If floorbeams or stringers are present, they are considered part of the superstructure (see Figure 9.3.3).

Figure 9.3.3 Concrete Deck Two-Girder, Underside View

Concrete through girders are used for spans ranging from 30 to 60 feet at locations with a limited under-clearance (see Figure 9.3.4). They are, however, not economical for wide roadways and are usually limited to approximately 24 feet wide. Girders are usually 18 to 30 inches wide and 4 to 6 feet deep.

Figure 9.3.4 Concrete Through Two-Girder Elevation View

Care must be taken not to describe concrete two-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 steel reinforcement.

In both a deck and a through two-girder structure, the live loads from the roadway surface are carried to the girders through the deck and stringers/floorbeams if present. The girders in turn carry the loads to the substructure.

Primary and Secondary Members

The primary members of a two-girder bridge are the girders, floorbeams and stringers (if present). The secondary members consist of diaphragms or struts.

Sometimes there is confusion regarding a transverse member and whether it is a floorbeam or a diaphragm. If design drawings are available, look at the reinforcement of these members. Diaphragms will be minimally reinforced while floorbeams will have more steel reinforcement. In the absence of drawings, compare the spacing of the deck girders and the transverse members. Decks are normally reinforced to cover the shortest distance between supports. If the transverse member spacing is greater than the deck girder spacing, the deck is probably supported by the girders. For this situation, the transverse member is most likely a diaphragm. Alternatively, if the transverse member spacing is less than the deck girder spacing, the deck is probably supported by the transverse member or floorbeam. If stringers are present, the transverse members will be considered floorbeams.

Steel Reinforcement

The primary reinforcing steel consists of main longitudinal tension reinforcement and shear reinforcement in the form of stirrups or inclined rebars. The main tension reinforcement is located in the bottom of the girder (positive moment) and on the top (negative moment). The beam also contains shear reinforcement, called stirrups that are located throughout the girder length. A single stirrup is generally two U-shaped bars that run transversely across the top, bottom and sides of the girder (see Figure 9.3.5). The need for stirrups is greatest near the beam supports where shear stresses are the highest. Shear reinforcement is also provided by bending the longitudinal bars to resist diagonal tension caused by shear.

The secondary (temperature and shrinkage) reinforcing steel is oriented longitudinally in the sides of the girders (see Figure 9.3.5). The primary and secondary reinforcing steel for the deck portion of the beam is the same as for a standard concrete deck.

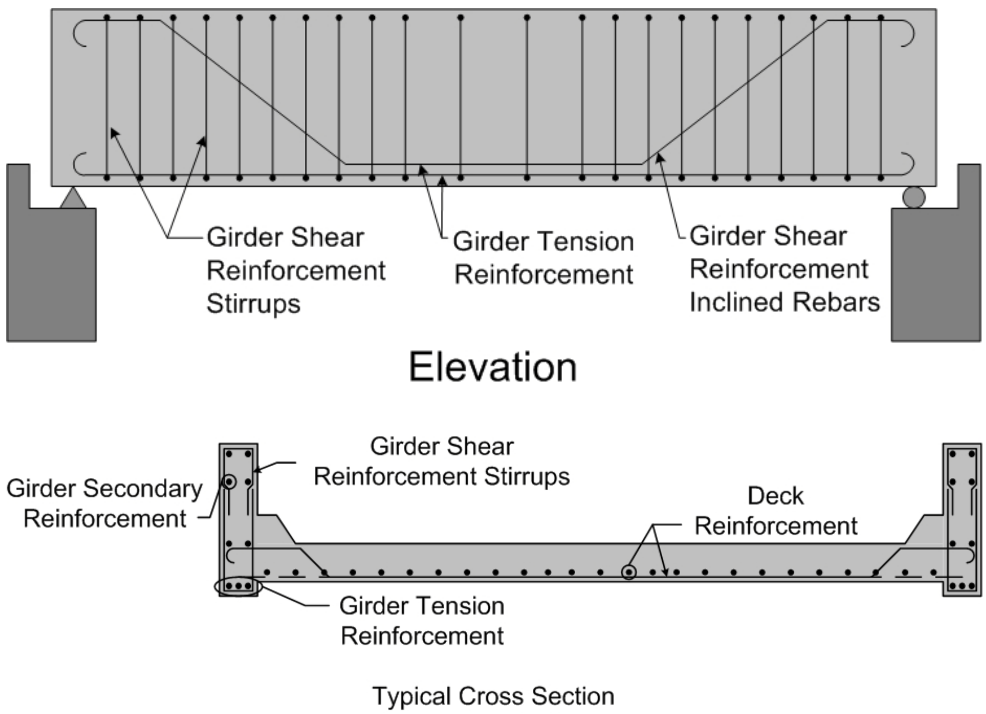

Figure 9.3.5 Steel Reinforcement in a Concrete Through Two-Girder

9.3.3 Overview of Common Deficiencies

Common deficiencies that occur on concrete two-girder bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)

- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Carbonation

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.3.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of concrete girders for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

Sounding by a hammer can be used to detect delaminated areas. 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.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

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

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

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Examine bearing areas for cracking, delamination or spalling where friction from thermal movement and high bearing pressure could overstress the concrete. Check for crushing of the girder near the bearing seat. Check the condition and operation of any bearing devices (see Figure 9.3.6).

Figure 9.3.6 Bearing Area of a Through Two-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. Carefully measure and document cracks, as they may indicate possible overstress due to shear.

Tension Zones

Examine tension zones for flexure cracks, which would be vertical on the sides and transverse across the girder and possibly the deck. The tension zones are at the midspan along the bottom of the through girders and possibly the deck for both simple and continuous span bridges (see Figure 9.3.7). Additional tension zones are located along the top of the girders and possibly the deck over the piers for continuous spans. Cracks greater that 1/16 inch wide indicate overstress due to tensile forces caused by 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, more significant, the 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 and debonded from the surrounding concrete due to spalling (see Figure 9.3.8). Document loss of bonding between reinforcement and concrete, section loss of concrete and remaining cross section of reinforcing steel since section loss and any debonding will decrease live load capacity.

Figure 9.3.7 Typical Elevation View of a Through Two-Girder Bridge with Tension Zones Indicated

Check similar bearing areas, shear zones, and tension zones for floorbeams and stringers if present

Secondary Members

Inspect diaphragms for flexural and shear cracks, as well as for typical concrete deficiencies. Deficiencies in the diaphragms may be an indication of differential settlement of the substructure or differential deflection of the concrete girders.

Figure 9.3.8 Exposed Reinforcement in a Through Two-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 9.3.9). Check around drain holes for deterioration of girder concrete.

Figure 9.3.9 Close-up of an Interior Face of a Through Two-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 with section loss as well as the spalled and delaminated concrete. The loss of concrete due to such an accident is not always serious, unless the bond between the concrete and steel reinforcement is affected.

Areas Previously Repaired

Examine areas that have been previously repaired. Determine if the repairs are in place and if they are functioning properly.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Camber

Using a string line, check for vertical alignment (camber) changes from the as-built condition of the deck and girders. Downward deflection usually indicates reduced capacity. Upward deflection usually indicates shrinkage.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces. Cracking will typically be transverse in the thinner regions of the deck and longitudinal near changes in cross section thickness.

9.3.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

In an element level condition state assessment of a concrete two-girder bridge, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	**Description**
Decks/Slabs	
12	Reinforced Concrete Deck * Note that this element designation is used regardless of the type of riding surface.
Superstructure	
110	Concrete Open Girder/beam
116	Concrete Stringer
155	Concrete Floorbeam

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

The unit quantity for decks and protective coating is square feet. The evaluation of NBE No. 12 is based solely on the top and bottom surface condition of the deck. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for the girder/beam, floorbeam or stringer is feet, and the total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of conventionally reinforced concrete two-girder superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)
366	Deck Traffic Impact (load capacity)

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

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Chapter 9
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Topic 9.4 Concrete Channel Beams

9.4.1 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 Figures 9.4.1 and 9.4.2). The channel beam can either be pre-cast or cast-in-place.

Figure 9.4.1 Underside View of Precast Channel Beam Bridge

Figure 9.4.2 Underside View of a Cast-in-Place Channel Beam Bridge

9.4.2
Design Characteristics

General

Channel beams are usually found on spans up to 50 ft.

Channel beams are generally precast and consist of a conventionally reinforced deck cast monolithically with two stems 3 to 4 feet apart (see Figure 9.4.1). Precast channel beam stems may be conventionally reinforced or may be prestressed. Stem tie bolts (see Figure 9.4.1) and shear keys (see Figure 9.4.4) are used to achieve monolithic action between precast channel beams.

Channel beams can also be cast-in-place with a curved underbeam soffit constructed over U-shaped beam forms (see Figure 9.4.2). These structures are sometimes referred as "pan bridges".

Figure 9.4.3 General View of a Precast Channel Beam Bridge

Primary and Secondary Members

The primary members of channel beam bridges are the channel beams. The secondary members of channel beam bridges are the end or intermediate diaphragms.

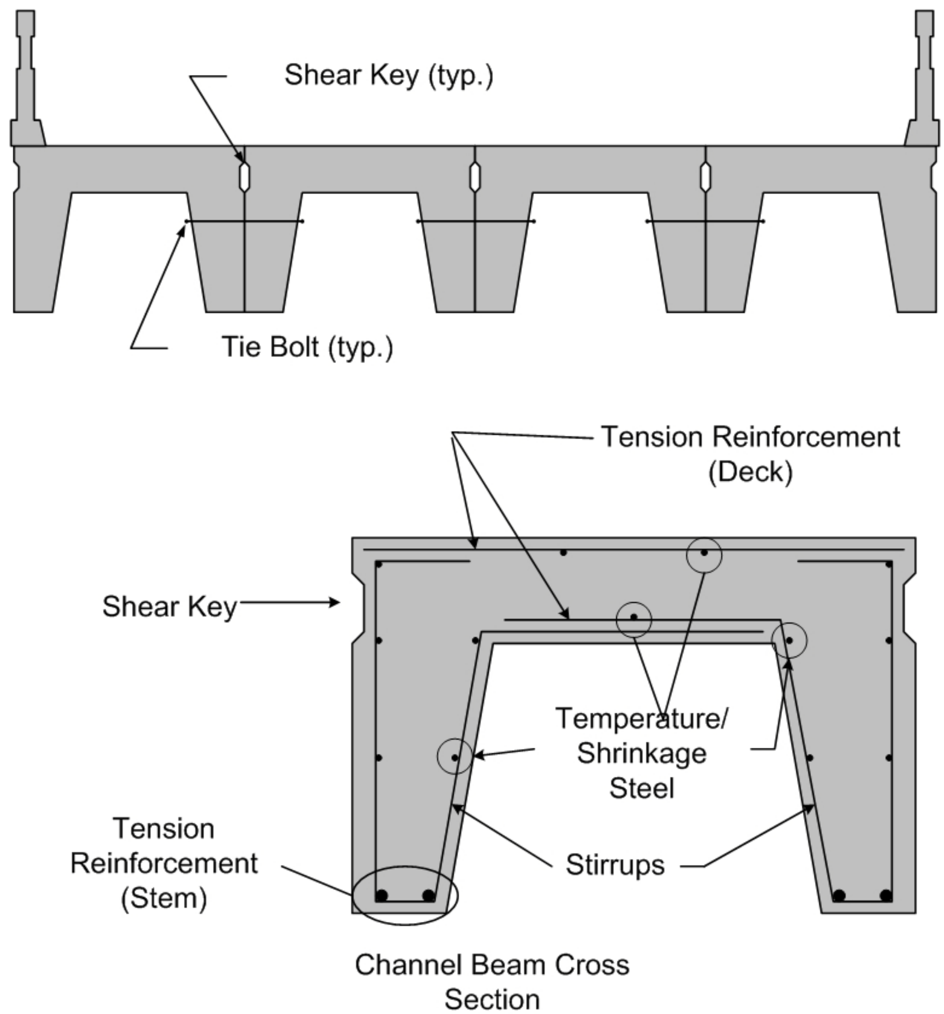
Steel Reinforcement

Reinforcement cover for older channel beam bridges is often less than today's cover requirements. Air entrained concrete was not specified in cast-in-place 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 deck portion of the beam is located in the bottom of the deck and is placed transversely, or perpendicular to the channel stems (see Figure 9.4.4).

The secondary (temperature and shrinkage) reinforcing steel is oriented longitudinally in the sides of deep channel stems and longitudinally in the deck. The primary and secondary reinforcing steel for the deck portion of the beam is the same as for a standard concrete deck (see Figure 9.4.4).



NOTE: Tension reinforcement in stem may consist of conventional or prestressing reinforcement

Figure 9.4.4 Cross Section of a Typical Channel Beam

9.4.3 Overview of Common Deficiencies

Common deficiencies that occur on concrete channel beam bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation
- Independent beam action

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.4.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of concrete channel beams for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

Sounding by hammer can be used to detect delaminated areas. 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. In most cases, a chain drag is used to check the top surface of a concrete deck. Hammers are used to sound the bottom of the deck and the stems and verify the integrity and tightness of the tie bolts.

Since prestressed beams are designed to maintain all concrete in compression, cracks are indications of serious problems. Carefully measure any crack in a prestressed channel beam with an optical crack gauge or crack comparator card and document the results.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

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

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Inspect bearing areas for cracking, delamination or spalling where friction from thermal movement and high bearing pressure could overstress the concrete. Check for crushing of the stem near the bearing seat. Check the condition and operation of any bearing devices.

Shear Zones

Inspect 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. Carefully measure and document cracks as they may indicate possible overstress due to shear.

Tension Zones

Examine superstructure tension zones 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 deck over the piers for continuous spans.

Flexure cracks caused by tension due to positive bending moment in the deck 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 deck.

Check for efflorescence from cracks and, more significant, discoloration of the concrete caused by rust stains from the reinforcing steel. In severe cases, the reinforcing steel may become exposed and debonded from the surrounding concrete due to spalling. Document loss of bonding between reinforcement and concrete, section loss of concrete and remaining cross section of reinforcing steel since section loss and any debonding will decrease live load capacity. Check for evidence of sagging or camber loss.

Secondary Members

Inspect diaphragms (see figures 9.4.5 and 9.4.9) for flexural and shear cracks as well as for typical concrete deficiencies. Deficiencies in the diaphragms may be an indication of differential settlement of the substructure or differential deflection of the concrete channel beams where only one beam is taking the load, not two or three as designed.

Joints

Inspect joints between adjacent beams for crushing and movement of the shear keys. The presence of open or loose joints 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 (if prestressed). The stress concentrations in these areas are very much different than a section away from the anchorages where a distributed stress pattern exists. Additionally, the effects of creep and tendon relaxation are somewhat higher in these regions.

Areas Exposed to Drainage

Inspect the seam or joint between adjacent precast beams for leakage. Leakage generally indicates a broken shear key between the channel beams (see Figure 9.4.5). If signs of leakage are present between beams, closely observe the shear keys for differential channel beam deflection under live load (see Figure 9.4.6). 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 stem tie-bolts for tightness and corrosion (see Figures 9.4.7 and 9.4.8). Do not confuse signs of corrosion with the epoxy used for the bolt.

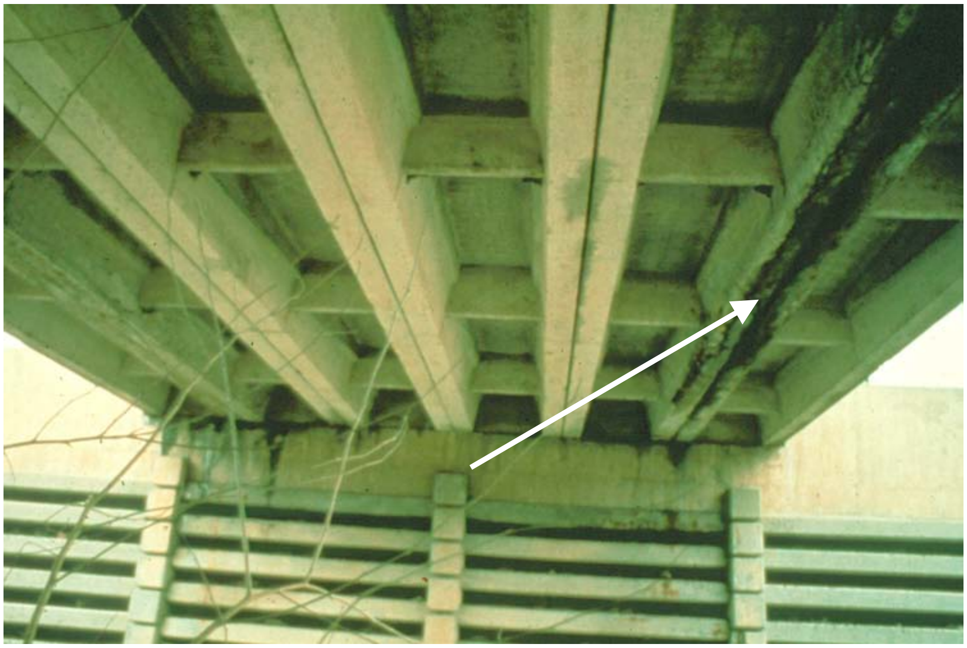


Figure 9.4.5 Joint Leakage Between Channel Beams

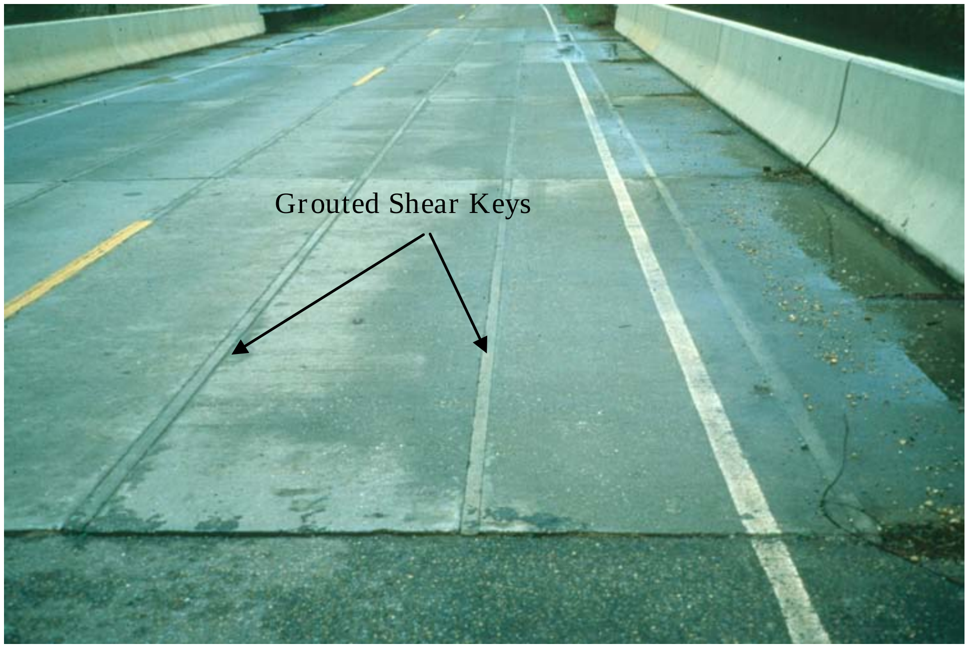


Figure 9.4.6 Top of Deck View of Precast Channel Beam Bridge

Figure 9.4.7 Stem Tie-Bolts

Figure 9.4.8 Close-up of Stem Tie-Bolt (Epoxy Resin)

Figure 9.4.9 Close-up of Intermediate Diaphragm

Areas Exposed to Traffic

Check areas damaged by collision. Document the number of exposed and severed reinforcing bars or prestressing strands with section loss, as well as the spalled and delaminated concrete. The loss of concrete due to such an accident is not always serious, unless the bond between the concrete and steel reinforcement is affected.

Areas Previously Repaired

Examine areas that have been previously repaired. Determine if the repairs are in place, and if they are functioning properly.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Camber

Using a string line, check for vertical alignment (camber) changes from the as-built condition of the channel beams. Downward deflection usually indicates reduced capacity or loss of prestress (if prestressed). Upward deflection usually indicates shrinkage or excessive initial prestressing force (if prestressed).

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the channel beam. Cracking will typically be transverse in the thinner regions of the channel beam and longitudinal near changes in cross section thickness.

9.4.5 Evaluation

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

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Items 58 and 59) for additional details about NBI component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating. For concrete channel beams, the deck condition influences the superstructure component rating (see figure 9.4.10). When the deck component condition rating is 4 or less, the superstructure component condition rating may be reduced if the recorded deck deficiencies reduce its ability to carry applied stresses associated with superstructure moments.

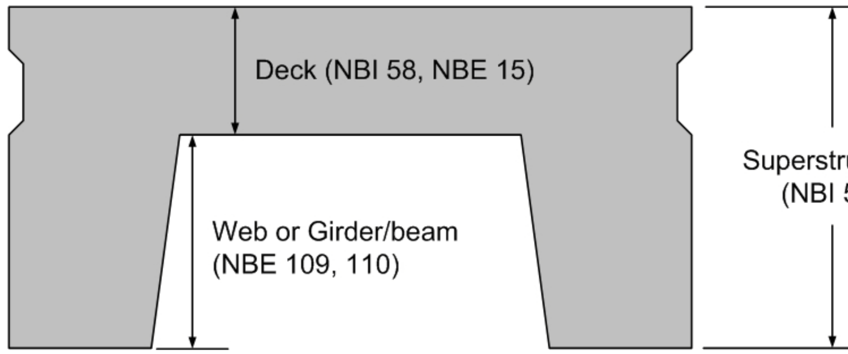


Figure 9.4.10 Components/Elements for Evaluation

Element Level Condition State Assessment

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

NBE No.	Description
Decks/Slabs	
15	Prestressed/Reinforced Concrete Top Flange* * Note that this element designation is used regardless of the type of riding surface.
Superstructure	
109	Prestressed Concrete Girder/Beam
110	Reinforced Concrete Girder/Beam

BME No.	Description
Wearing Surfaces and Protection Systems	
510	Wearing Surfaces
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

The unit quantity for the top flange, wearing surfaces, protection systems and protective coating is square feet. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency (see Figure 9.4.10). The unit quantity for the girder is feet, and the total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the AASHTO *Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of channel beam superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 9
Inspection and Evaluation of Concrete Superstructures

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Topic 9.5 Concrete Arches

9.5.1 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 due to its high compressive capacity. 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 bending moments, and shear in addition to axial compression.

9.5.2 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 deck or tee beam floor system (see Figure 9.5.1). This type of arch is generally for 200 feet and longer spans.

Figure 9.5.1 Open Spandrel Arch Bridge

Closed Spandrel Arch

Closed spandrel arches are deck arches since the roadway is above the arch. 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 9.5.2). This type of arch is efficient in short span applications.

Figure 9.5.2 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 is 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 9.5.3). These types of arches are sometimes referred to as "Rainbow Arches".

Figure 9.5.3 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 9.5.4). Segmental arches are oval or elliptical and can have several hinges along the arch (see Figure 9.5.5). 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 9.5.4 Precast Concrete Arch with Integral Vertical Legs

Figure 9.5.5 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. For this type of design, the deck and supporting members bear on the top or crown of the arch (see Figure 9.5.6).

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 9.5.6 Precast Post-tensioned Concrete Arch without Spandrel Columns

Primary 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 primary 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 deck 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 and reduce the unsupported compression length between supports (see Figure 9.5.7).

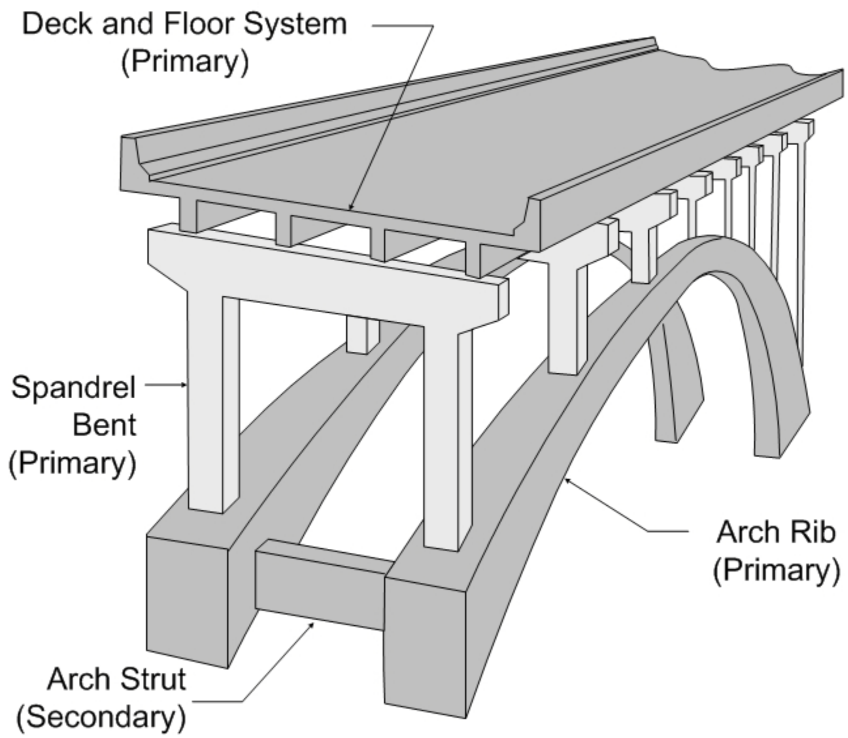

Figure 9.5.7 Primary and Secondary Members of an Open Spandrel Arch

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 9.5.8).

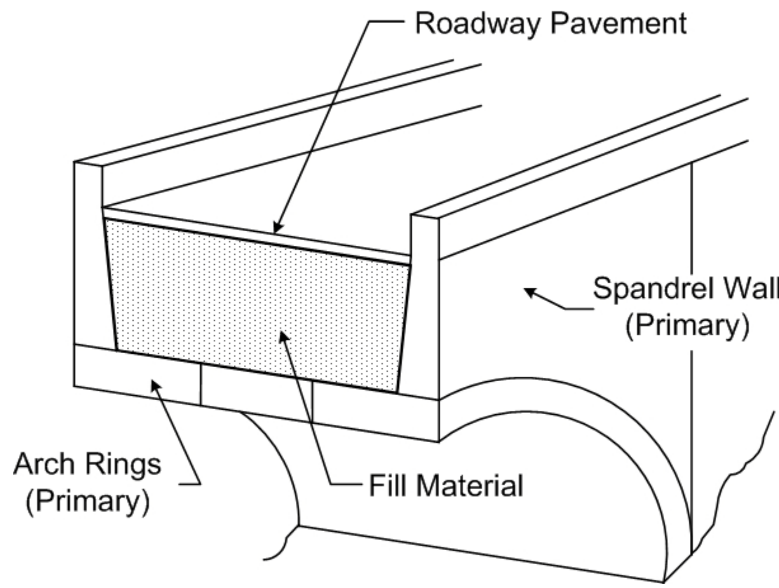

Figure 9.5.8 Primary Members of a Closed Spandrel Arch

Steel Reinforcement

For the proper inspection and evaluation of concrete arch bridges, 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 9.5.9).

In spandrel bent caps, the primary reinforcement is tension and shear steel. This is provided using "Z" shaped bars and stirrups since the cap behaves like a fixed end beam (see Figure 9.5.10).

The floor system is designed and reinforced similar to other concrete beams (e.g. tee beams).

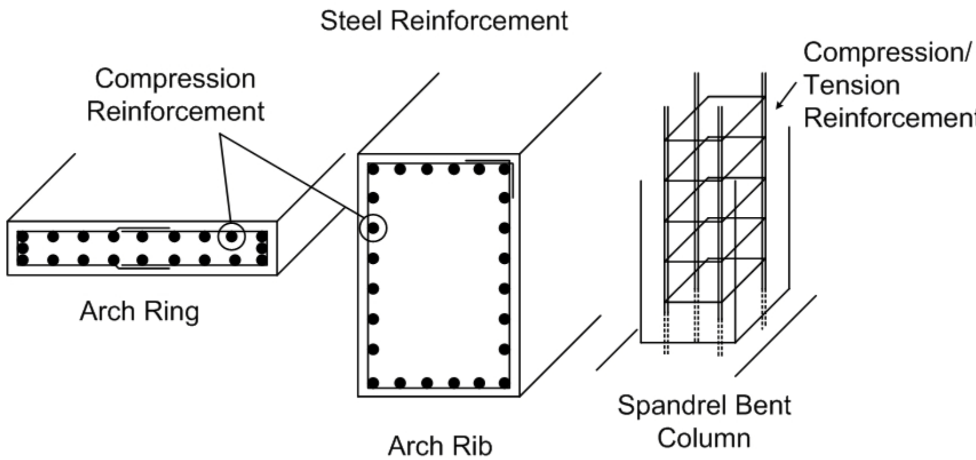

Figure 9.5.9 Open Spandrel Arch and Spandrel Bent Column Reinforcement

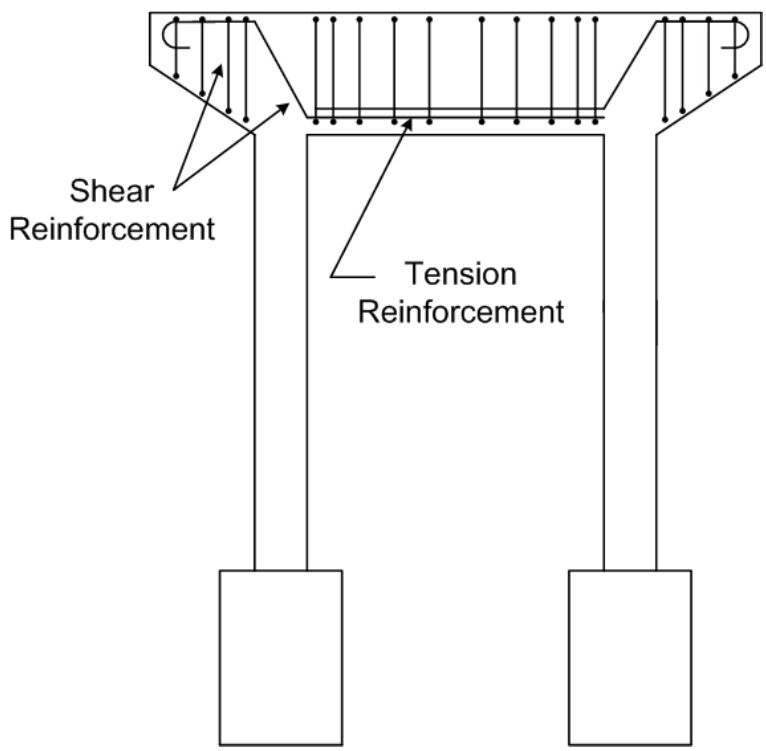

Figure 9.5.10 Spandrel Bent Cap Reinforcement

Note that tension reinforcement in spandrel bent columns and bent caps resist tensile forces caused by bending moments in Figures 9.5.9 and 9.5.10.

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 and resists tension caused by lateral earth pressure bending. Lateral ties or connections between the roadway slab and the spandrel walls are sometimes used to resist this tension. The front, or outside, face of the wall is reinforced in both directions with temperature and shrinkage steel (see Figure 9.5.11).

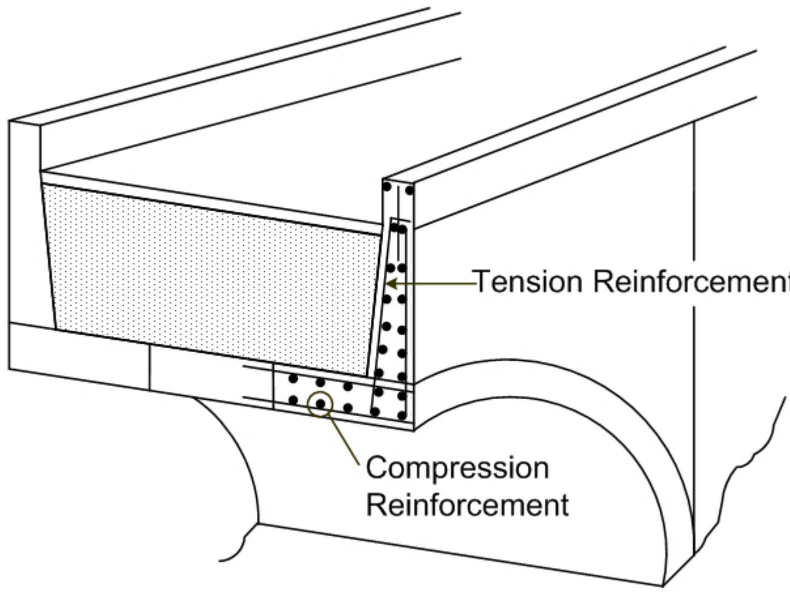

Figure 9.5.11 Reinforcement in a Closed Spandrel Arch

Other Reinforcement

Temperature and shrinkage reinforcement is used in the spandrel bent caps and spandrel walls.

Lateral ties are used to support compressive reinforcement in the arch rings and arch ribs.

9.5.3 Overview of Common Deficiencies

Common deficiencies that occur on concrete arches include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation

- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed presentation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.5.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete the deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of concrete arches for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

Sounding by hammer 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.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

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

- Rebound and penetration methods
- Ultrasonic testing
- Smart Concrete
- Carbonation

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

The arch/skewback interface has the greatest bearing load magnitude (see Figure 9.5.12). Inspect for loss of cross section of the reinforcement bars at the spalls. Examine the arch for longitudinal cracks. These may 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 feet 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 at spalled areas. Examine the column for cracks which begin at the inside corner and propagate upward. These indicate differential arch rib deflections (see Figure 9.5.13).

The floor system/bent cap interface has the smallest bearing load magnitude. Examine bearing areas as described in the deck, tee beam and girder sections.

Examine the arch ring for unsound concrete. Look for rust stains, cracks, discoloration, crushing, and deterioration of the concrete. Inspect the interface between the spandrel wall and the arch for spalls that could reduce the bearing area. Investigate the arch for transverse cracks, which indicate an overstress condition.

Figure 9.5.12 Arch/Skewback Interface

Figure 9.5.13 Spandrel Column Bent 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 in a fashion similar to tee beams and girders.

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 9.5.14). 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 9.5.15).

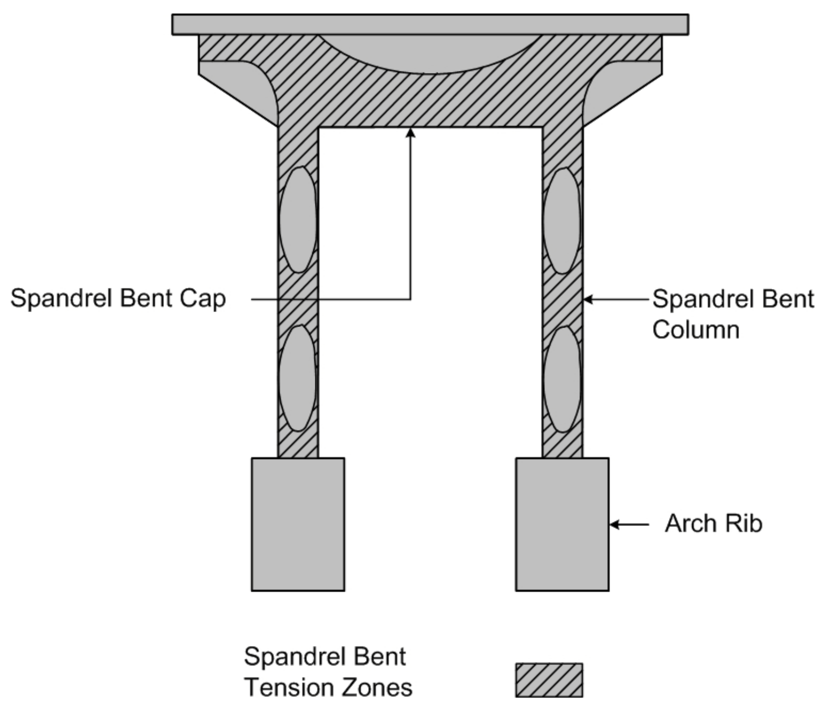

Figure 9.5.14   Spandrel Bent Tension Zones

Figure 9.5.15 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 9.5.16).

Figure 9.5.16 Severe Scaling and Spalling on a Spandrel Column

Movement of Spandrel Wall

Check for movement, or rotation, of the spandrel wall, which is the most common mode of failure for closed spandrel arches.

Secondary Members

Inspect diaphragms and struts for flexural and shear cracks, as well as for typical concrete deficiencies (see Figure 9.5.17). Deficiencies in the secondary members may be an indication of differential settlement of the substructure or differential deflection of the concrete arches.

Figure 9.5.17 Inspection and Documentation of Arch Strut Deficiencies

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 9.5.18).

Figure 9.5.18 Scaling and Contamination on an Arch Rib Due to a Failed Drainage System

For a closed spandrel arch, verify that the weep holes are working properly. Also, check that surface water drains properly and does not penetrate the fill material.

Areas Exposed to Traffic

Check deficiency areas by collision. Document the number of exposed and severed reinforcing bars with section loss as well as the spalled and delaminated concrete. The loss of concrete due to such an accident is not always serious, unless the bond between the concrete and steel reinforcement is affected.

Areas Previously Repaired

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.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

9.5.5 Evaluation

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

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. Consider the previous inspection data along with current inspection findings to determine the correct component condition rating. See Topic 4.2 (Item 59) for additional details about NBI component condition rating guidelines.

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

Element Level Condition State Assessment

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

NBE No.	Description
Superstructure	
	Open Spandrel Arch
109	Prestressed Concrete Girder/Beam
110	Reinforced Concrete Girder/Beam
154	Prestressed Concrete Floorbeam
155	Reinforced Concrete Floorbeam
115	Prestressed Concrete Stringer (stringer-floorbeam system)
116	Reinforced Concrete Stringer (stringer-floorbeam system)
143	Prestressed Concrete Arch
144	Reinforced Concrete Arch
	Closed Spandrel Arch
143	Prestressed Concrete Arch
144	Reinforced Concrete Arch

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

The unit quantity for concrete arch superstructure elements is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency (refer to Figure 9.5.19 for closed spandrel arch measurements). The unit quantity for protective coatings is square feet, with the total area distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions. Note the condition of the spandrel walls and spandrel columns in the concrete arch elements.

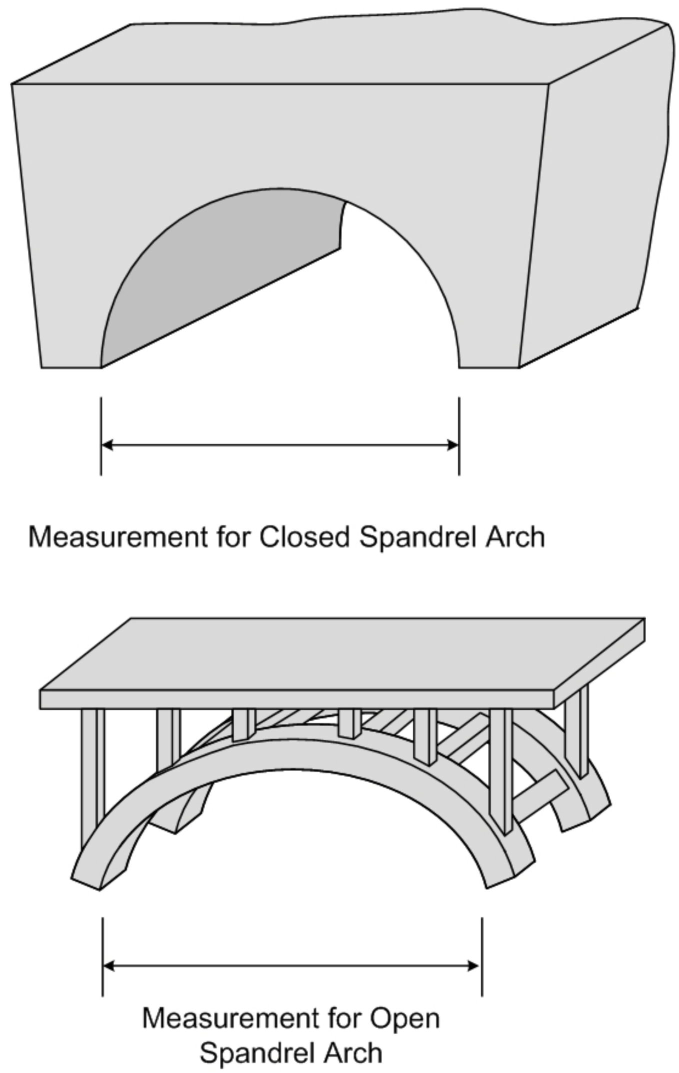

Figure 9.5.19 Measurements for Open and Closed Spandrel Arches

The following Defect Flags are applicable in the evaluation of arch superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
360	Settlement
361	Scour
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 9
Inspection and Evaluation of Concrete Superstructures

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Topic 9.6 Concrete Rigid Frames

9.6.1 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 are either cast-in-place or precast units.

9.6.2 Design Characteristics

General

The rigid frame bridge can either be single span or multi-span (see Figure 9.6.1). Single span frame bridges generally utilize slabs to span up to 50 feet. The basic single span frame shape is most easily described as an inverted "U" (see Figure 9.6.2).

Figure 9.6.1 Multi-span Concrete Rigid Frame Bridges

Multi-span frame bridges are used for spans over 50 feet with slab or rectangular beam designs. Other common multi-span frame shapes include the basic rectangle, and the slant leg or K-frame (see Figure 9.6.3). Due to frame action between the horizontal members and the vertical or inclined members, multi-span frames are considered continuous.

Figure 9.6.2 Single-span Rectangular Concrete Rigid Frame Bridge

Rigid frame structures are utilized both at grade and under fill, such as in concrete frame culverts (see Topics 14.1 and 14.2).

Figure 9.6.3 Three Span Concrete K-frame Bridge

Primary and Secondary Members

For single span frames, the primary member is considered to be the slab portion and the legs of the frame (see Figure 9.6.4). For state and federal rating evaluation, the slab portion is considered the superstructure while the legs are considered the substructure. Secondary member diaphragms may be present between frame legs or frame beams. They are not very common in this bridge type.

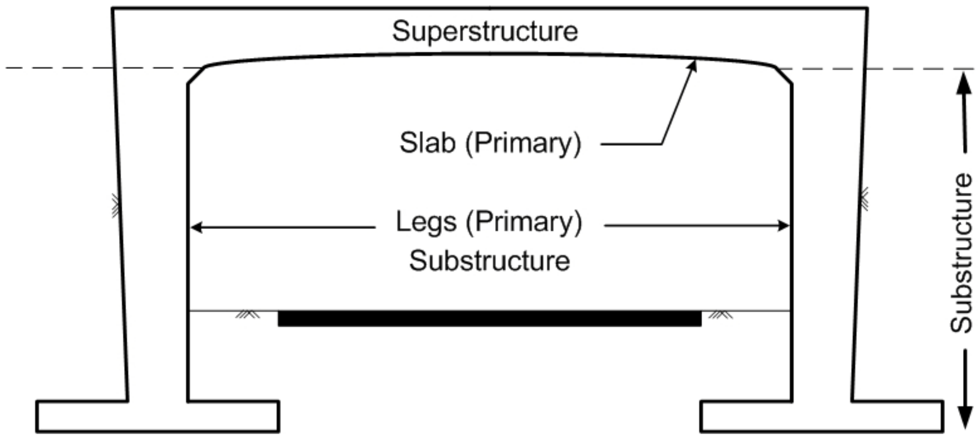

Figure 9.6.4 Elevation of a Single Span Frame

For multi-span frames, the primary members include the frame legs (the slanted beam portions which replace the piers) and the frame beams or slab (the horizontal portion which is supported by the frame legs and abutments) (see Figure 9.6.5). For state and federal rating evaluation, the frame beams or slabs and frame legs are considered the superstructure while the abutments are considered the substructure.

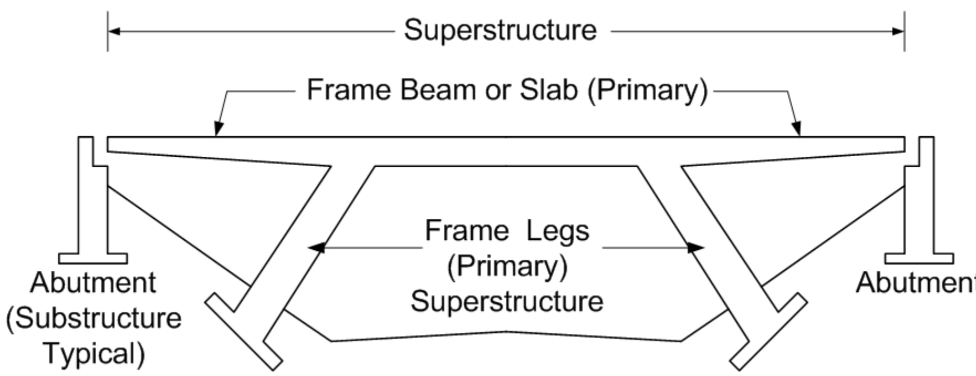

Figure 9.6.5 Elevation of a K-frame

Steel Reinforcement

Rigid frame structures develop positive and negative moment throughout due to the interaction of the frame legs and frame beams (see Figure 9.6.6). In slab or beam frames, the primary reinforcement is used to resist tension and possibly shear.

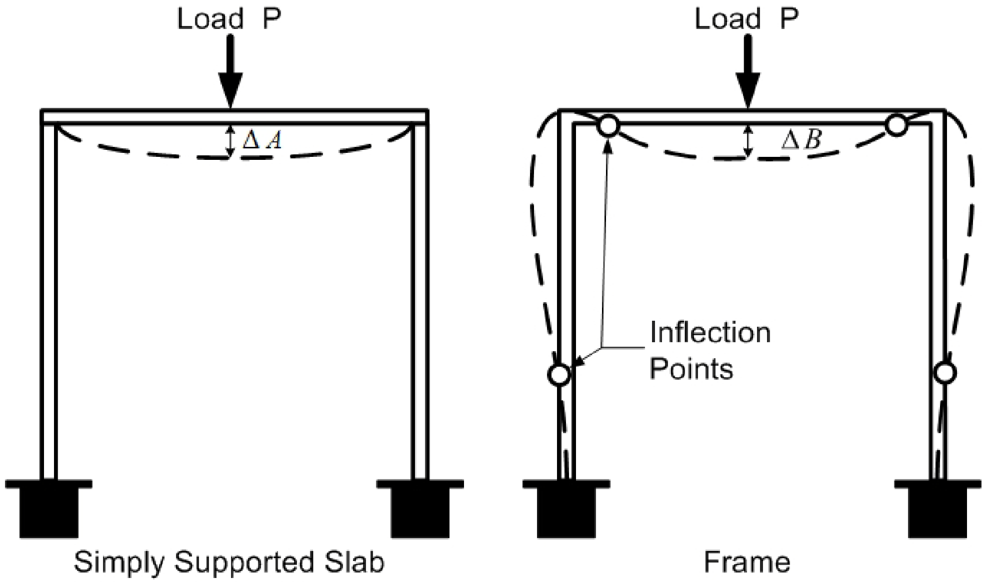

Figure 9.6.6 Deflected Simply Supported Slab versus Deflected Frame Shape

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 9.6.7).

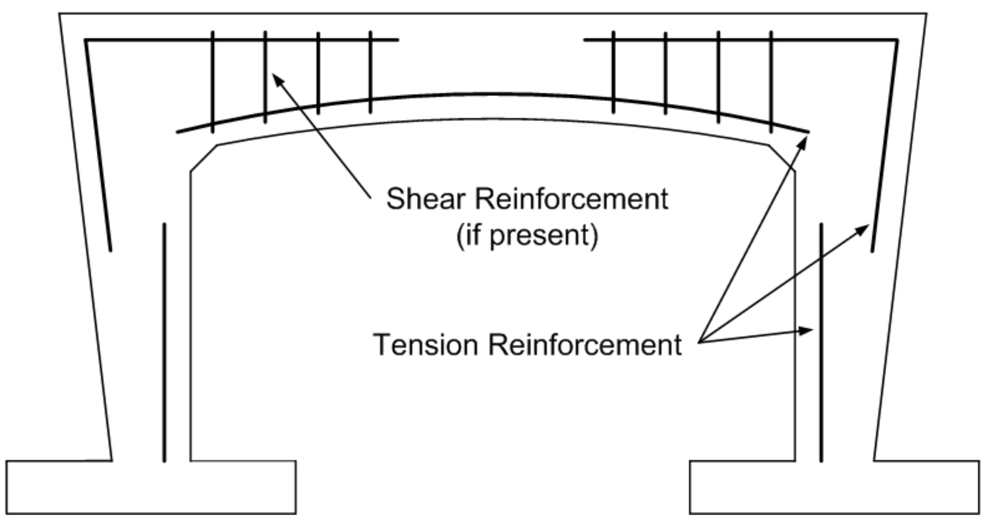

Figure 9.6.7 Primary Reinforcement in a Single Span Frame

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 9.6.8). If shear reinforcement is required, stirrups are provided.

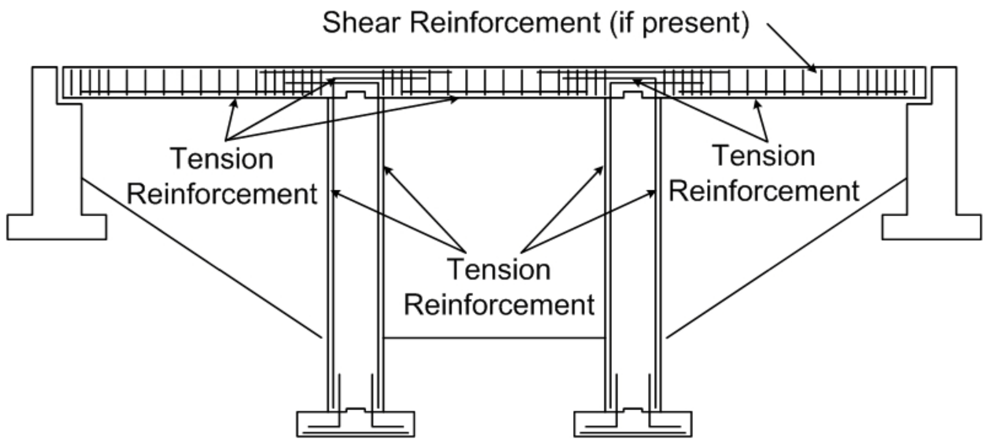

Figure 9.6.8 Primary Reinforcement in a Multi-span Slab or Beam Frame

The primary reinforcement in the frame beam portion is longitudinal tension and shear stirrup steel if required, similar to continuous beam reinforcement (see Topic 9.2.2).

In the frame legs, the primary reinforcement is tension and shear steel near the top and compression steel with ties for the remaining length (see Figure 9.6.9). See Topic 12.2 for a discussion of compression steel and column ties.

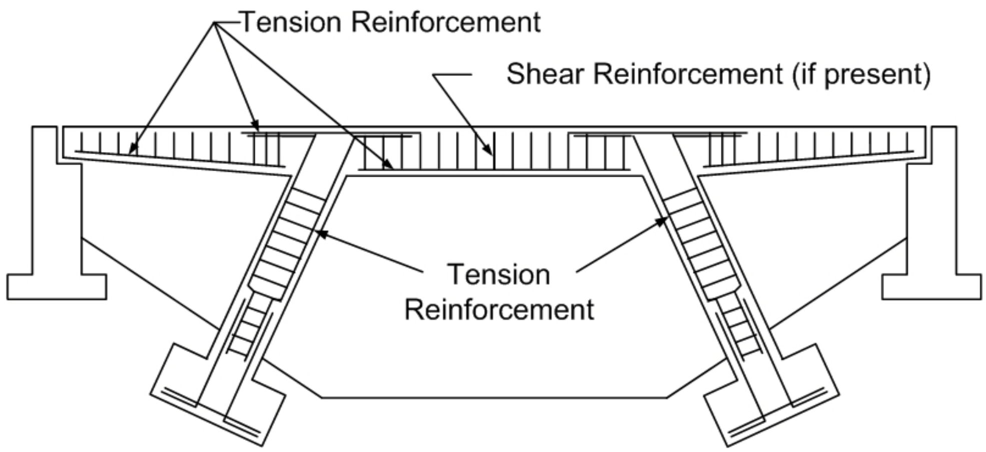

Figure 9.6.9 Primary Reinforcement in a K-frame

Secondary Reinforcement

Temperature and shrinkage reinforcement is distributed similar to that of a slab (see Topic 9.1.2) or tee-beam (see Topic 9.2.2) or box beams (see Topic 9.10.2).

9.6.3 Overview of Common Deficiencies

Common deficiencies that occur on concrete rigid frame bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.6.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of concrete rigid frames for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

Sounding by hammer or chain drag can be used to detect delaminated areas. 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.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

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

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Examine the bearing areas for cracking, delamination or spalling where friction from thermal movement and high bearing pressure could overstress the concrete. Check for crushing of the slab or frame beams over the frame legs. Check the condition of the bearings, if present.

Shear Zones

Inspect the area near the supports where the frame beams or slab meet the frame legs or abutments. Look for shear cracks in the frame beams or slab (beginning at the frame legs and propagating upward toward mid-span).

Inspect the frame legs for diagonal cracks that initiated at the frame beam/slab or footing (see Figure 9.6.10).

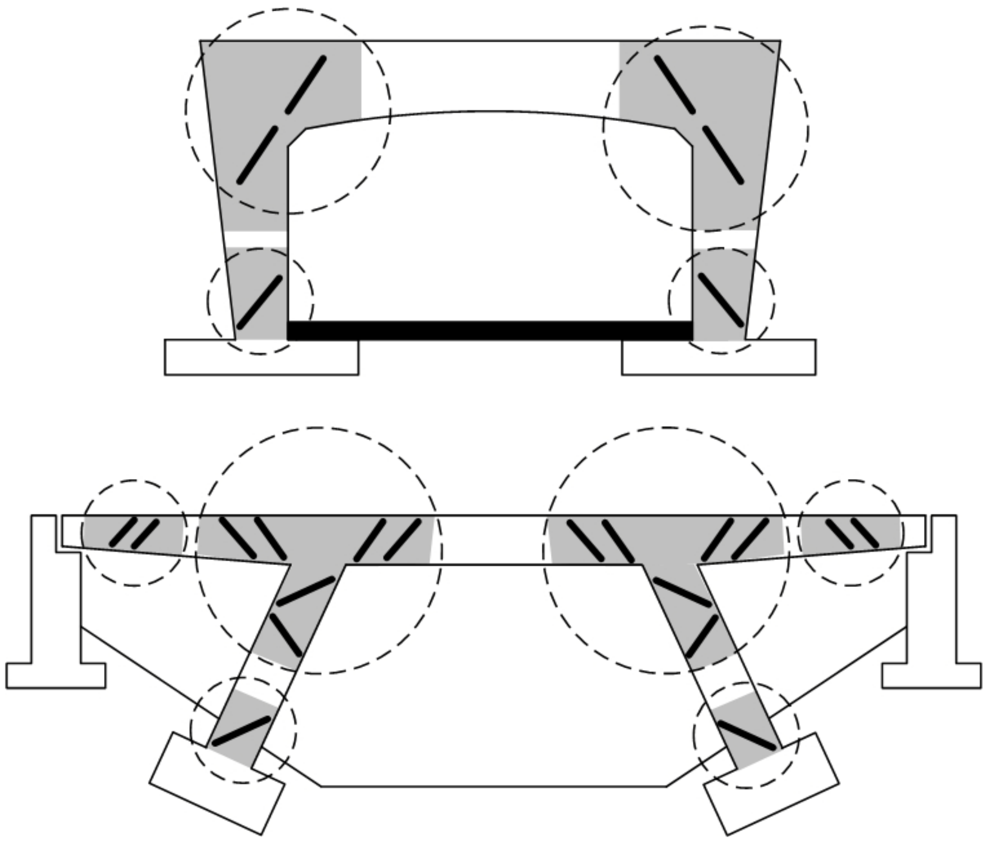

Figure 9.6.10 Shear Zones in Single Span and Multi-span Frames

Tension Zones

Inspect 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 Figures 9.6.11 and 9.6.12).

Compression Zones

Investigate the compression areas for delamination, spalling, scaling, crushing, and exposed reinforcement. The legs of a frame act primarily as columns with a moment applied at the top (see Figures 9.6.11 and 9.6.12). Check the entire length of the frame legs for horizontal cracks, which indicate crushing.

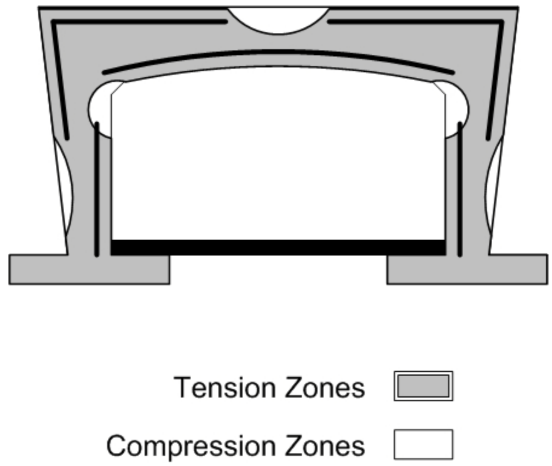

Figure 9.6.11 Tension and Compression Zones in a Single Span Frame

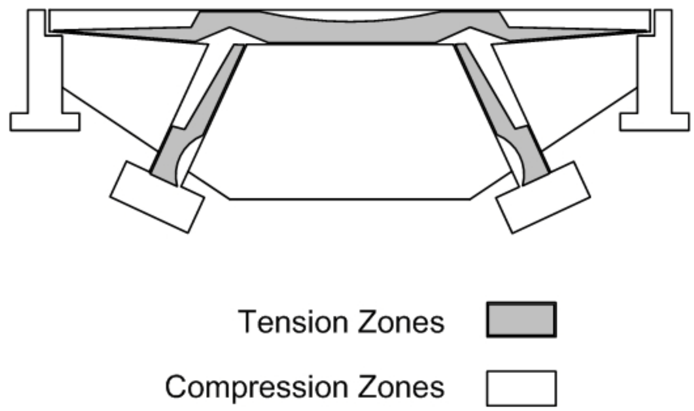

Figure 9.6.12 Tension and Compression Zones in a Multi-span Frame

Secondary Members

Secondary members are not common for this bridge type. Inspect secondary members (including diaphragms) for flexural and shear cracks as well as typical concrete deficiencies. Deficiencies in secondary members may be an indication of differential settlement of the substructure or differential deflection of the rigid frames.

Areas Exposed to Drainage

Examine the areas exposed to drainage for deteriorated and contaminated concrete. Check the roadway surface of the slab or frame beams for delamination and spalls (see Figure 9.6.13). Give special attention to the tension zones and water tables.

Figure 9.6.13 Roadway of a Rigid Frame Bridge with Asphalt Wearing Surface

Check longitudinal joint areas of adjacent slab or frame beams for leakage and concrete deterioration (see Figure 9.6.14). Check around scuppers and drain holes for deteriorated concrete. Check slab or frame beam ends for deterioration due to leaking deck joints at the abutments. Check to see if weep holes are functioning

Figure 9.6.14 Longitudinal Joint Between Slab Frames

Areas Exposed to Traffic

Check areas damaged by collision. Document the number of exposed and severed reinforcing bars with section loss as well as the spalled and delaminated concrete. The loss of concrete due to such an accident is not always serious, unless the bond between the concrete and steel reinforcement is affected.

Areas Previously Repaired

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.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Camber

Using a string line, check for vertical alignment (camber) changes from the as-built condition of the rigid frame beam/slab. Downward deflection usually indicates reduced capacity. Upward deflection usually indicates shrinkage.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the rigid frame. Cracking will typically be transverse in the thinner regions of the tee-beam (such as the deck) and longitudinal near changes in cross section thickness (such as within the knee area).

9.6.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

There is no specific element level condition state assessment of a concrete rigid frame bridges. Possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) that may be used to best describe a concrete rigid frame are:

NBE No.	**Description**
Decks/Slabs	
38	Reinforced Concrete Slab* * Note that this element designation is used regardless of the type of riding surface
Superstructure	
105	Reinforced Concrete Closed Web/Box Girder
110	Reinforced Concrete Girder/Beam
Substructure**	
205	Reinforced Concrete Column/Pile Extension
210	Reinforced Concrete Pier Wall
215	Reinforced Concrete Abutment

**Note: AASHTO does not have a National Bridge Element designation for superstructure columns, pier walls, and abutments. AASHTO substructure NBE designations are used to represent these elements. This is applicable for single span bridges.

BME No.	**Description**
Wearing Surfaces and Protection Systems	
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

The unit quantity for slabs, protection systems and protective coatings is square feet. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for the girder/beam, abutment, and pier wall is feet, and the total length is distributed among the four condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. The unit quantity for column/pile extensions is each and the entire element is placed in one of the four available condition states. Condition State 1 is the best possible rating. See the AASHTO *Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of concrete rigid frame superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
360	Settlement
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 9

Inspection and Evaluation of Concrete Superstructures

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Topic 9.7 Precast and Prestressed Slabs

9.7.1 Introduction

Precast and prestressed slabs have gained popularity since the 1950's. This type of design acts as a deck and superstructure combined (see Figure 9.7.1). Individual members are placed side by side and connected together so they act as one slab. When vertical clearances are lacking, this type of design is effective, due to the slab's shallow depth. Wearing surfaces are generally applied to the top of precast and prestressed slabs and are either concrete or bituminous.

Figure 9.7.1 Typical Prestressed Slab Beam Bridge

Although precast and prestressed slabs are different from concrete decks, the design characteristics, wearing surfaces, protection systems, common deficiencies, inspection procedures and locations, evaluation, and motorist safety concerns are similar to concrete decks. Refer to Topic 7.2 for additional information about concrete decks. For the purpose of this manual, decks are supported by superstructure members while slabs are supported by substructure units.

9.7.2 Design Characteristics

General

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 9.7.2). Precast slab bridges with very short spans may not contain voids. Precast slabs also contain drain holes, which are strategically placed in the bottom of the slab to allow accumulated moisture to escape.

Precast slab units are practical for spans up to 60 feet. The slabs can be single or multiple simple spans. The units are typically 36 or 48 inches wide and have a depth of up to 26 inches.

Prestressed precast units are generally comprised of 4,000 to 8,000 psi prestressed concrete and reinforced with 270 ksi pre- or post-tensioned steel tendons.

Conventional concrete precast slabs with compressive strength of 3000 to 4000 psi and reinforcement of 40 ksi or 60 ksi can also be fabricated but are less common than prestressed precast slabs.

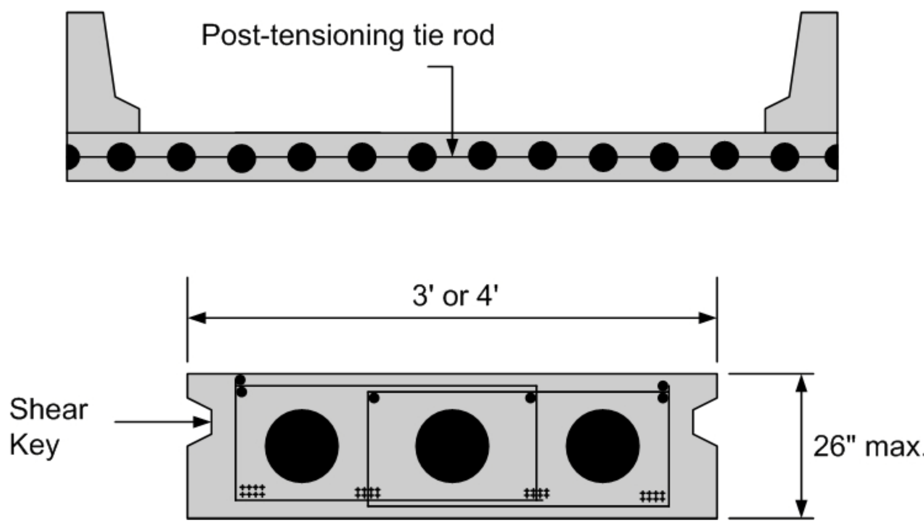

Figure 9.7.2 Cross Section of a Typical Voided Slab

Monolithic Behavior

Adjacent slab units may be post-tensioned together with tie rods having a tensile capacity of 145 ksi and grouted at the shear keys. Together these enable the slab units to act monolithically.

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 flange, bottom flange, and two webs. A typical box beam has a minimum depth of 12 inches, but may be as deep as 72 inches. A typical voided slab section has two or three circular voids through it. It is also possible to find precast solid slab units.

Poutre Dalle System

Recent technology has also produced the inverted tee beam, a new type of precast slab, for short to medium span bridges known as the Poutre Dalle system. Developed in France, the Poutre Dalle precast segments can span up to 82 feet with a maximum span of 105 feet. The overall composite depth of the spans is $1/28^{th}$ to $1/30^{th}$ of the span length with a width of 16 to 79 inches (see Figures 9.7.3 and 9.7.4). Beam depth may also be reduced for applications utilizing shorter spans. The combination of strength and size allows this beam type to compete as favorable alternative to precast solid slabs.

Figure 9.7.3 Poutre Dalle Precast Slab Bridge

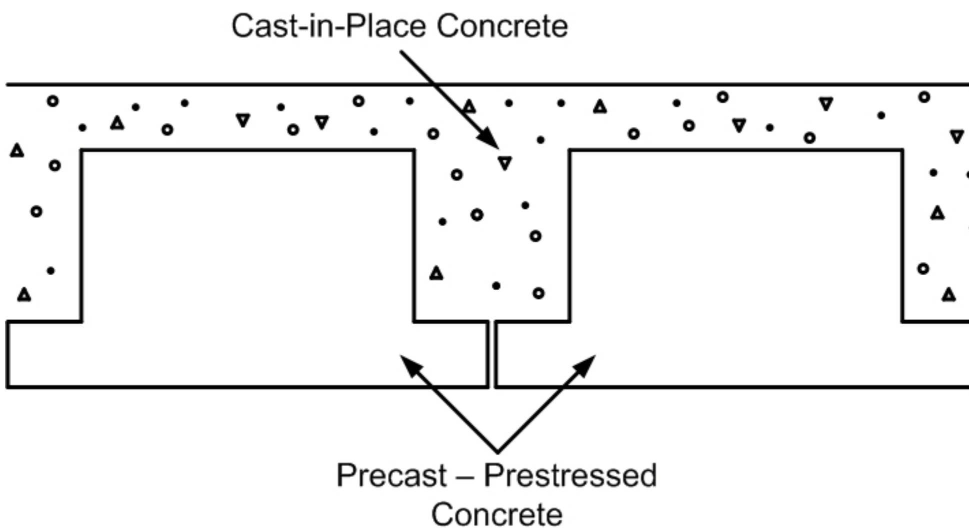

Figure 9.7.4 Poutre Dalle Bridge Schematic

Primary and Secondary 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. There are no secondary members.

Steel Reinforcement

Primary Reinforcement

The primary reinforcement consists of longitudinal tension steel and shear reinforcement or stirrups.

Prestressing strands placed near the bottom of the slab make up the main tension steel. Depending on the age of the structure, the strand size will be 1/4, 3/8, 7/16, or 1/2 inch diameter. Prestressing strands are normally spaced 2 inches on center (see Figure 9.7.5) and have a tensile strength of 270 ksi. Tension steel for conventionally reinforced precast slabs has a yield strength of 60 ksi or 40 ksi depending on the age of the structure.

Shear reinforcement consists of U-shaped or closed loop stirrups located throughout the slab at various spacings required by design.

Secondary Reinforcement

Secondary reinforcement is provided to control temperature and shrinkage cracking. This reinforcement is placed longitudinally in the beam, normally at the top of the slab units, and holds the stirrups in place during fabrication.

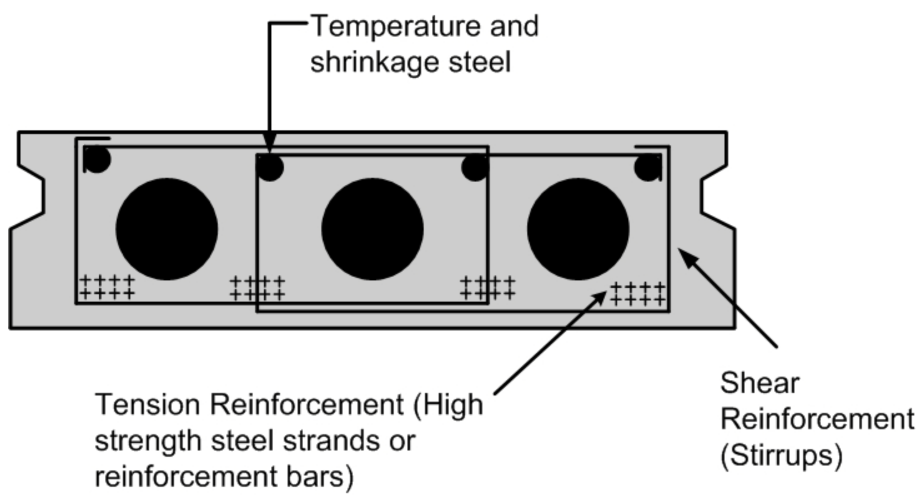

Figure 9.7.5 Slab Beam Bridge Tension and Shear Reinforcement

9.7.3 Overview of Common Deficiencies

Common deficiencies that occur on precast and prestressed slab bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs

- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed presentation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.7.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of concrete slabs for surface cracks, spalls, wear, and other deficiencies is primarily a visual activity.

Physical

The physical examination of the top surface of the slab with a hammer can be a tedious operation. In most cases, a chain drag is used to determine delaminated areas. A chain drag is typically 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 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.

Since prestressed beams are designed to maintain all concrete in compression, cracks are indications of serious problems. For this reason, carefully measure any crack with an optical crack gauge or crack comparator card and document the results.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Electrical methods
- Delamination detection machinery
- Ground-penetrating radar
- Electromagnetic methods
- Pulse velocity
- Flat jack testing
- Impact-echo testing
- Infrared thermography

- Laser ultrasonic testing
- Magnetic field disturbance
- Neutron probe for detection of chlorides
- Nuclear methods
- Pachometer
- Rebound and penetration methods
- Ultrasonic testing
- Smart Concrete
- Carbonation

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Examine the bearing areas for cracking, delamination or spalling where thermal movement and high bearing pressure could overstress the concrete. End spalling can eventually lead to the loss of bond in the prestressing tendons. Also check bearing areas for deficiencies due to leaking joints.

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 on the vertical surfaces and along the bottom of the slab units close to the supporting substructure units.

Inspect the top and bottom surface for longitudinal reflective cracking and between the slab sections for leakage (see Figure 9.7.6). These problems indicate failed shear keys and that the slab units are no longer tied together or acting monolithically. Observe if there is differential slab deflection under live load.

Figure 9.7.6 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 present. 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. Measure all cracks with an optical crack gauge or crack comparator card.

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 unit.

Inspect the slabs for exposed strands. Prestressed strands will corrode rapidly and fail abruptly. Therefore, any exposure is significant (see Figure 9.7.7).

Check for longitudinal cracking in slab members. Water freezing in the voids can cause longitudinal cracks. Skewed slab units may exhibit longitudinal cracks due to uneven prestressing force in the strands.

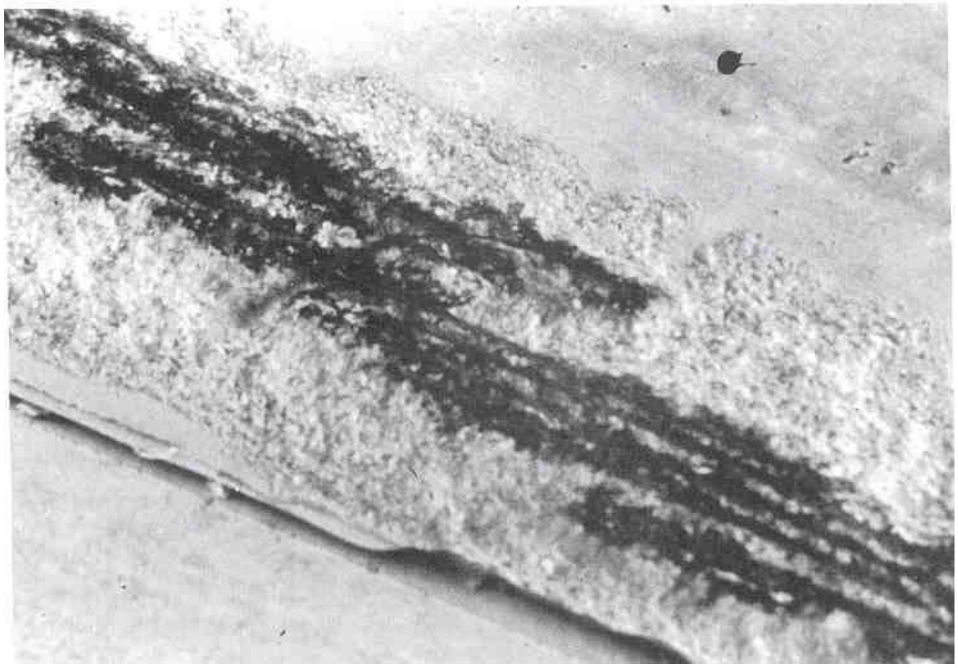

Figure 9.7.7 Exposed Strands in a Precast Slab Beam

Shear Keys

In precast and prestressed slab bridges, check each segment for independent vertical deflection, which may indicate failed shear keys, and lateral deflection on the exterior slab segments, which may indicate eccentric loading of the exterior slab segment. Inspect slab joints for signs of leakage between slabs, which is indicative of deteriorated shear key grout material. Independent movement of slab segments decreases the load capacity of the superstructure and need to be coded accordingly and referred to an engineer for evaluation.

Joints

Inspect joints for crushing and movement of the shear keys. The presence of open or loose joints 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 (if prestressed). The stress concentrations in these areas are very much different than a section away from the anchorages where a distributed stress pattern exists. Additionally, the effects of creep and tendon relaxation are somewhat higher in these regions.

Areas Exposed to Drainage

Inspect areas exposed to drainage for deteriorated and contaminated 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.

Inspect longitudinal joints between the precast slabs. Drainage through the joints indicates a broken shear key and loss of monolithic action.

Check that drain holes for voids in slabs are functioning. If not, water can accumulate and cause premature failure of the slab segment.

Areas Exposed to 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 good vertical clearance afforded by the relatively shallow slab units.

Check areas exposed to wear on the top surface.

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.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Acute Angles on Skewed Bridges

Examine skewed bridges for lateral displacement and cracking of acute corners due to point loading and insufficient reinforcement.

Post-Tensioned Grout Pockets

Check the condition of the lateral post-tensioning grout pockets and visible ends of the post-tensioning rods. Cracked grout or rust stains may indicate a failure of the post-tensioning rod or loss of monolithic action.

Camber

Using a string line, check for vertical alignment (camber) changes from the as-built condition of the slab units. Loss of positive camber indicates loss of prestress in the tendons.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the slab. Cracking will typically be transverse in the thinner regions of the slab and longitudinal near changes in cross section thickness (if applicable).

9.7.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the deck and the superstructure. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Items 58 and 59) for additional details about NBI component condition rating guidelines. For a precast or prestressed slab bridge, these guidelines must be applied for both the deck component and the superstructure component.

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

Typically, for this type of structure, the deck and superstructure components will have the same rating.

Element Level Condition State Assessment

In an element level condition state assessment of a precast or prestressed slab bridge, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	**Description**
Decks/Slabs	
38	Reinforced Concrete Slab* * Note that this element designation is used regardless of the type of riding surface
104	Prestressed Closed Web/box Girder

BME No.	**Description**
Wearing Surfaces and Protection Systems	
510	Wearing Surfaces
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

The unit quantity for slabs, wearing surfaces, protection systems and protective coatings is square feet. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. Closed web/box girder, NBE No. 104, is the closest choice in the AASHTO element list for precast prestressed voided slabs. The unit quantity for the girder is feet, and the total length is distributed among the available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element.

Condition State 1 is the best possible rating. See the AASHTO *Guide for Bridge Element Inspection* for condition state descriptions. States may decide to choose their own element number for precast prestressed voided slabs because AASHTO does not have a specific element number for prestressed slabs.

The following Defect Flags are applicable in the evaluation of precast and prestressed slab superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

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Chapter 9
Inspection and Evaluation of Concrete Superstructures

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Topic 9.8 Prestressed Double Tees

9.8.1 Introduction

A prestressed double tee beam, like the name implies, resembles two adjacent capital letter T's that are side by side (see Figure 9.8.1). 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 9.8.1 Typical Prestressed Double Tee Beam

9.8.2 Design Characteristics

General

Prestressed concrete double tee beams have a monolithic deck and stem design that allows the deck to act integrally with the stems to form a superstructure. The integral design provides a stiffer member, while the material-saving shape reduces the dead load. Lateral connectors enable load transfer between the individual double tee sections (see Figure 9.8.2).

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.

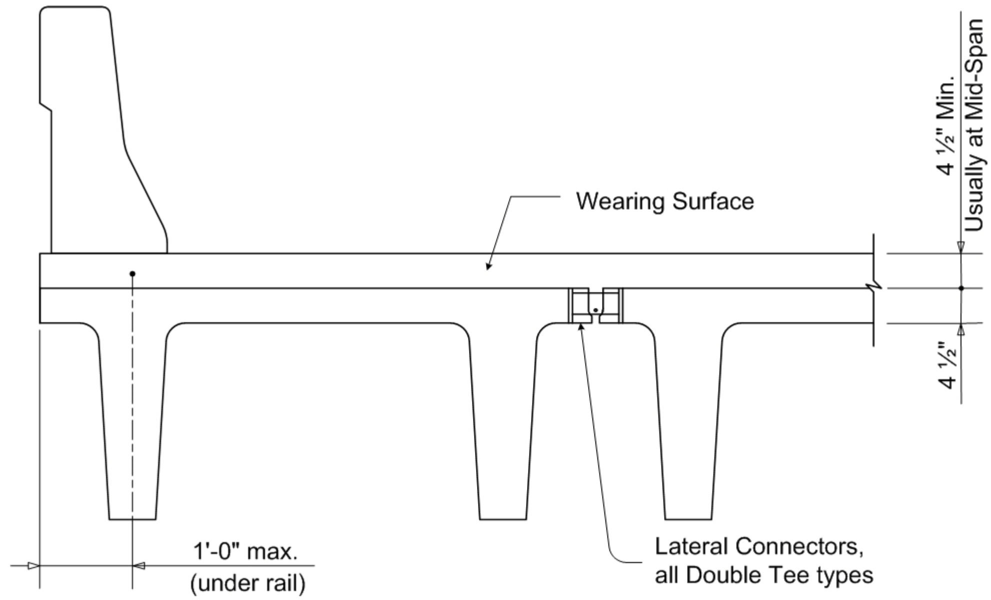

Figure 9.8.2 Prestressed Double Tee Beam Typical Section

Prestressed double tees have a typical stem depth of 12 to 34 inches. The average flange width is 8 to 10 feet, with a typical span length of approximately 25 to 55 feet. Prestressed double tees can be used in spans approximately 80 feet long with stem depths up to 5 feet and flange widths up to 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 9.8.3). This occurs so that the beam end can sit flush on the bearing seat.

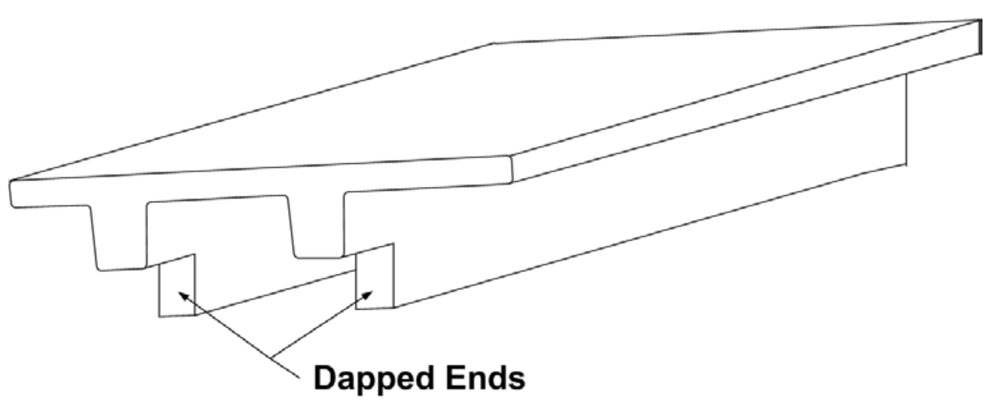

Figure 9.8.3 Dapped End of a Prestressed Double Tee Beam

The top of the flange or deck section of prestressed double tees can act as the integral wearing surface or be overlaid. Bituminous asphalt and concrete are typical examples of wearing surfaces that may be applied. See Topic 7.2.3 for a detailed description of the different types of concrete deck wearing surfaces.

Primary 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 and shear steel reinforcement consists of prestressing strands and mild reinforcement (see Figure 9.8.4). 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 7.2.2).

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 deck. In some newer designs, welded-wire-fabric is used as the secondary and shear reinforcement in the stems. 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.

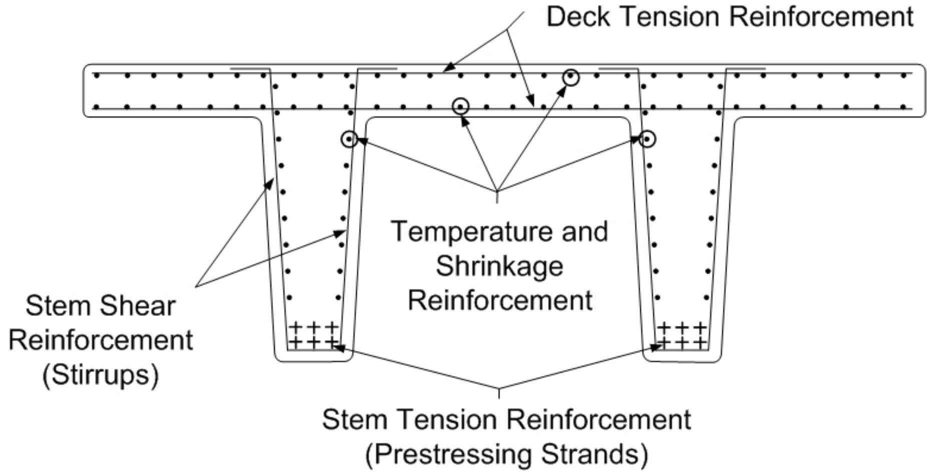

Figure 9.8.4 Steel Reinforcement in a Prestressed Double Tee Beam

9.8.3 Overview of Common Deficiencies

Common deficiencies that occur on prestressed concrete double tee beam bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.8.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of prestressed double tees for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

The physical examination of a double tee beam with a hammer can be a tedious operation. In most cases, a chain drag is used to determine areas on the top flange or deck surface. 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. Hammer sounding is used to examine the stem and bottom surface of the flange.

Since prestressed beams are designed to maintain all concrete in compression, cracks are indications of serious problems. Carefully measure any crack with an optical crack gauge or crack comparator card and document the results.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

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

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

- Concrete permeability
- Concrete strength
- Endoscopes and videoscopes

- Moisture content
- Petrographic examination
- Reinforcing steel strength
- Chloride test
- Matrix analysis
- ASR evaluation

Locations

Bearing Areas

Examine bearing areas for cracking, delamination or spalling where friction from thermal movement and high bearing pressure could overstress the concrete. Check for crushing of the stem near the bearing seat. Check the condition and operation of any bearing devices.

Shear Zones

Inspect 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. Carefully measure these cracks, as they may represent lost shear capacity.

For dapped-end double tee beams, look for 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 9.8.5).

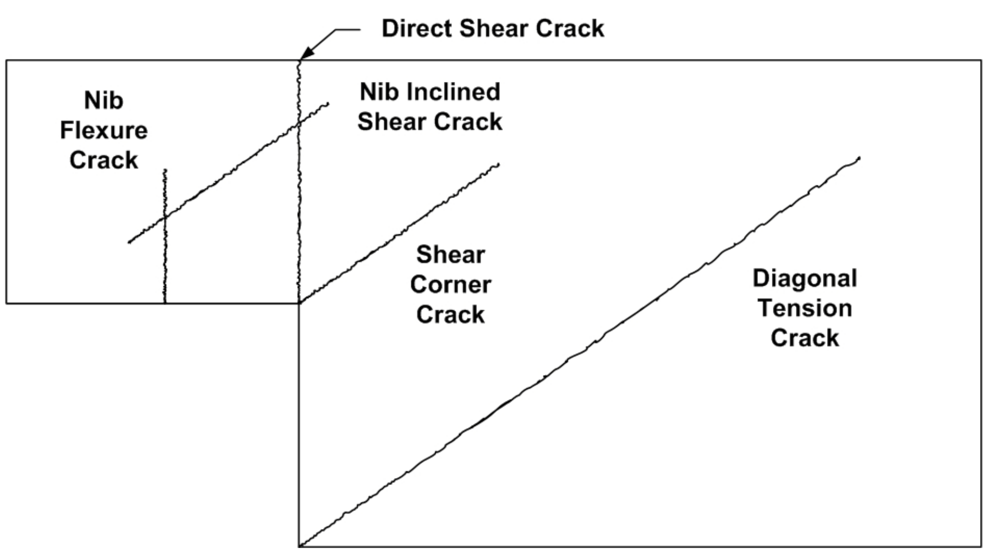

Figure 9.8.5 Crack Locations for Dapped End Double Tee Beams

Tension Zones

Examine tension zones 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 deck over the piers of continuous spans. For dapped-end double tee beams, look for vertical flexure cracks in the reduced depth section that sits on the bearing seat (see Figure 9.8.5).

Flexural cracks caused by tension due to deck loading will be found on the underside in a longitudinal direction between the stems.

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.

Secondary Members

Inspect diaphragms for flexure and shear cracks as well as typical concrete deficiencies. Cracks in the diaphragms could be an indication of overstress or excessive differential deflection in the double tee beams or differential settlement of the substructure.

Joints

Inspect longitudinal joints for crushing and movement of the shear keys. The presence of open or loose joints 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 (if prestressed). The stress concentrations in these areas are very much different than a section away from the anchorages where a distributed stress pattern exists. Additionally, the effects of creep and tendon relaxation are somewhat higher in these regions.

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. Inspect the seam or joint between adjacent beams for leakage.

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 Exposed to 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.

Areas Previously Repaired

Examine areas that have been previously repaired. Determine if the repairs are in place and if they are functioning properly.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

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.

Camber

Using a string line, check for horizontal alignment and vertical camber changes from the as-built condition 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.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the tee-beam. Cracking will typically be transverse in the thinner regions of the tee-beam and longitudinal near changes in cross section thickness.

9.8.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection for* element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the deck and the superstructure. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Items 58 and 59) for additional details about NBI component condition rating guidelines.

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

For prestressed double tees, the deck condition influences the superstructure component condition rating. When the deck component condition rating is 4 or less, the superstructure component condition rating may be reduced if the recorded deck deficiencies reduce its ability to carry applied stresses associated with superstructure moments.

Element Level Condition State Assessment

In an element level condition state assessment of a prestressed double tee beam bridge, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Decks/Slabs	
15	Prestressed/Reinforced Concrete Top Flange* * Note that this element designation is used regardless of the type of riding surface
Superstructure	
109	Prestressed Concrete Girder/Beam

BME No.	Description
Wearing Surfaces and Protection Systems	
510	Wearing Surface
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

The unit quantity for the top flange (deck), wearing surfaces, protection systems and protective coating is square feet. The evaluation of the top flange element is based on the top and bottom surface condition of the top flange (see Figure 9.8.7). The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for the prestressed double tee beam is feet, and the total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the AASHTO *Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of prestressed double tee beam superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

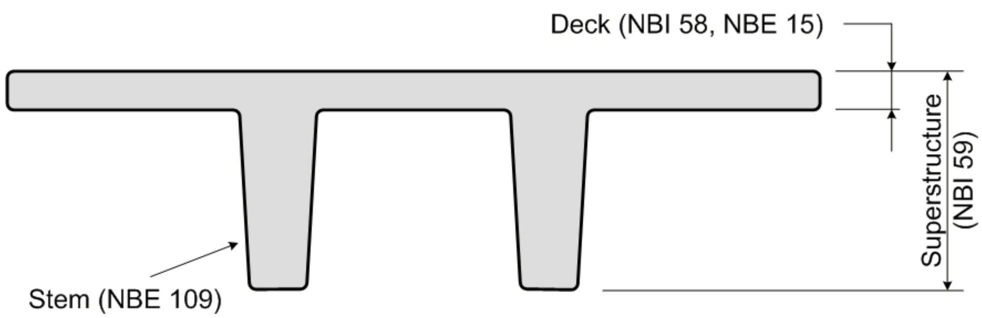

Figure 9.8.6 Components/Elements for Evaluation

Table of Contents

Chapter 9
Inspection and Evaluation of Concrete Superstructures

Topic 9.9 Prestressed I-Beams and Bulb-Tees

9.9.1 Introduction

Prestressed I-beams have been used since the 1950's, while prestressed bulb-tee beams have been used since the 1960's. These beam types have proven to be effective because of their material saving shapes. The I or T shape allows a designer to have enough cross-section to place the proper amount of reinforcement while reducing the amount of concrete needed (see Figure 9.9.1).

Figure 9.9.1 Prestressed I-beam Superstructure

9.9.2 Design Characteristics

Prestressed I-beams and bulb-tees make economical use of material since most of the concrete mass is located in the top and bottom flanges and away from the neutral axis of the beam.

General

The most common prestressed concrete I-beam shapes are the AASHTO shapes used by most state highway agencies (see Figures 9.9.2 and 9.9.3). Some highway agencies have also developed variations of the AASHTO I-beam shapes to accommodate their particular needs.

Prestressed I-beams are used in spans ranging from 40 to 200 feet. They are generally most economical at spans from 60 to 160 feet.

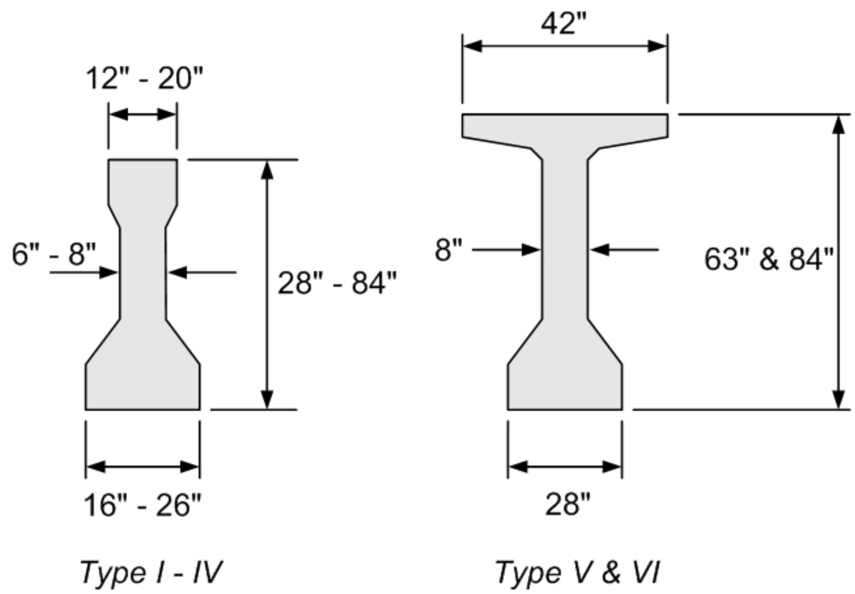

Figure 9.9.2 AASHTO Cross Sections of Prestressed I-beams

Figure 9.9.3 AASHTO Prestressed I-beam Bridge

Originally developed from the AASHTO Type V and VI shapes, AASHTO-PCI bulb-tee shapes utilize a smaller cross-section with fewer prestressing strands to achieve comparable span lengths between traditional AASHTO I-beams (see Figures 9.9.4 and 9.9.5). Due to the reduced volume of concrete used in fabrication, bulb-tee beams consequently have reduced material costs, lower shipping weights, and increased stability during transportation. Economical span lengths range between 80 and 160 feet. When higher strength concrete is used, maximum span lengths can approach 200 feet. Bulb-tee beams are also suited for span continuity using post-tensioning techniques (presented later in this topic). As with AASHTO I-beams, some highway agencies have also developed variations of the AASHTO-PCI bulb-tee shapes to accommodate their particular needs, including New England bulb-tee (NEBT) girders, variable depth bulb-tee beams, and bulb-tees with increased web depths and/or top flange widths.

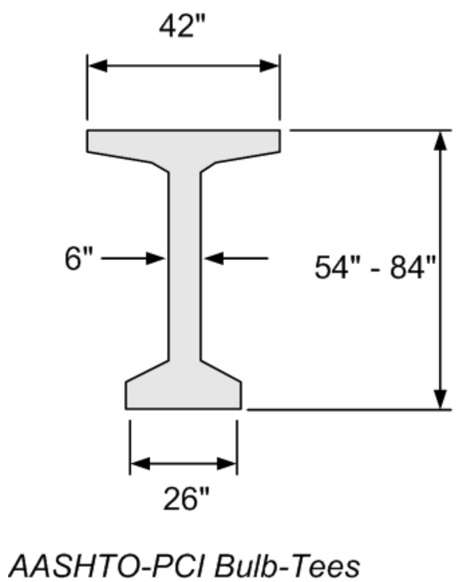

Figure 9.9.4 Cross Section of AASHTO-PCI Bulb-Tee Beams

Figure 9.9.5 Placement of an AASHTO-PCI Bulb-Tee Beam

A variation of AASHTO-PCI bulb-tee shapes, deck bulb-tee beams were developed to provide an integral deck-superstructure girder. Using similar geometry as the AASHTO-PCI bulb-tee with the exception of the top flange width (see Figure 9.9.6), these beams have become a viable alternative to adjacent box beam superstructures. Economical span lengths range between 80 and 160, with maximum span lengths of 200 feet depending on the girder depth and top flange width. Some highway agencies have developed variations of the deck bulb-tee girder to accommodate their particular needs, often with increased web depths to achieve maximum span lengths exceeding 200 feet. Deck bulb-tee beams are transversely post-tensioned and/or grouted to allow adjacent units to act integrally or together to carry the live loads. They may also be longitudinally post-tensioned together for splicing and/or continuity, as presented later in this topic.

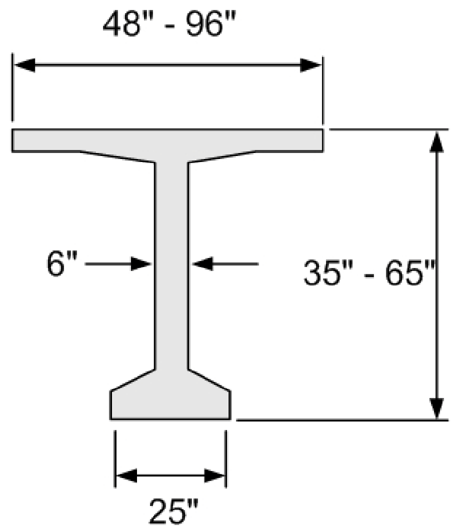

Figure 9.9.6 Cross Section of AASHTO-PCI Bulb-Tee Beams

Materials – Strength and Durability

Steel tendons with a tensile strength as high as 270 ksi are located in the bottom flange and web (depending on the application). These tendons are used to induce compression across the entire section of the beam prior to and during application of live load. The result is a crack free beam when subjected to live load (see Topic 6.2.5).

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 4,000 psi to 8,000 psi compressive strength. Concrete with ultimate compressive strengths up to 12,000 psi is available. In addition, concrete has a higher quality due to better control of fabrication conditions in a casting yard.

High Performance Concrete

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.

Reactive Powder Concrete

Reactive Powder Concrete (RPC) prestressed beams can come in an X-shape (see Figure 9.9.7) 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 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.

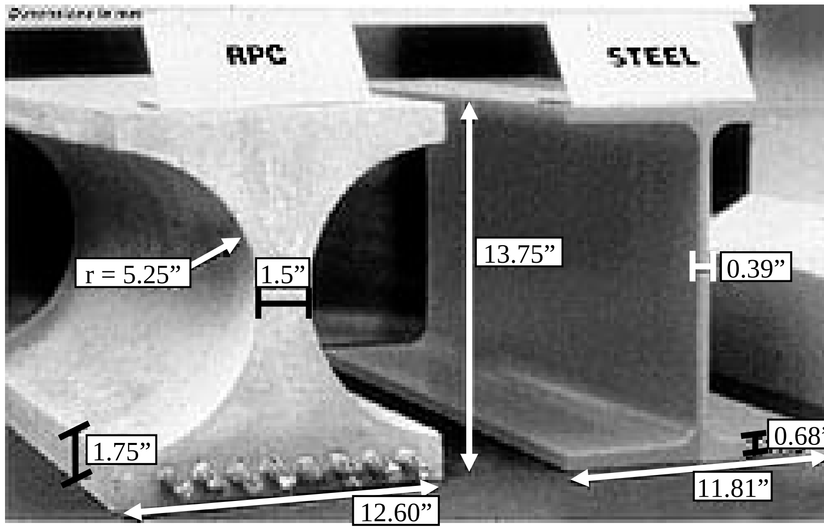

Figure 9.9.7 Reactive Powder Concrete (RPC) Prestressed X-beam

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 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 and bulb-tees can be made continuous for live load and/or to eliminate the deck joint. This may be done using a continuous composite action deck and anchorage of mild steel reinforcement in a common end diaphragm (see Figures 9.9.8 and 9.9.9).

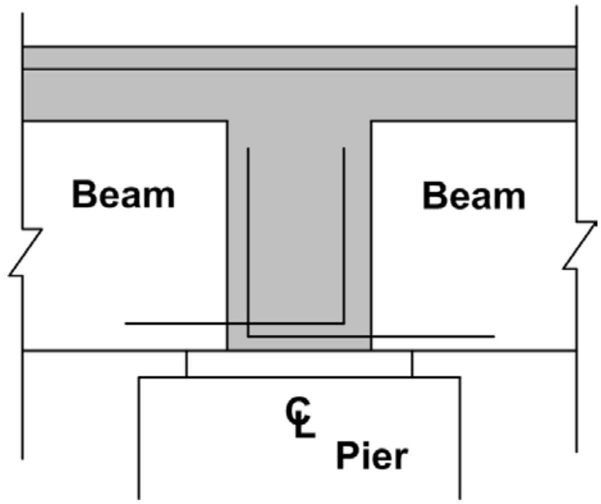

Figure 9.9.8 Continuous Prestressed I-beam Schematic

Figure 9.9.9 Continuous Prestressed I-Beam Bridge

Continuity has also been accomplished using post-tensioning ducts cast into pretensioned I-beams or bulb-tee 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.

Spliced Girders

Post-tensioning may also be used to splice I-beam and bulb-tee girder segments together, which are typically pretensioned to resist dead load and transportation stresses. The post-tensioning ducts are typically located in the web, while the bottom flange is pretensioned. This technique allows for span lengths up to 300 feet that can be easily transported in two or three smaller units. Girder splicing may also be used to create multi-span continuous bridges (see Figure 9.9.10).

Figure 9.9.10 Spliced Bulb-Tees with Haunched Girder Sections Over Piers

Composite Action

The deck is secured to and can be made composite with the prestressed beam by the use of extended stirrups which are cast into the I-beam or bulb-tee beam (see Figure 9.9.11).

Note that deck bulb-tee beams by themselves (including overlays) are not considered to be composite because the deck (top flange) and beam (web and bottom flange, or "bulb") are constructed of the same material. A composite topping (deck) may be used for some designs.

Figure 9.9.11 Extended Stirrups to Obtain Composite Action

Primary and Secondary Members

The primary members are the prestressed 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 9.9.12). Diaphragms are cast-in-place concrete or rolled steel sections and are placed at either the end points, mid points, or third points along the span.

Figure 9.9.12 Concrete Intermediate and End 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 pretensioned high strength prestressing strands or tendons placed symmetrically in the bottom flange and lower portion of the web. Strands are 3/8, 7/16, 1/2 or 0.6 inch in diameter and are generally spaced in a 2 inch grid. In the larger beams, main tension steel can include post-tensioned continuity tendons which are located in ducts cast into the beam web (see Figures 9.9.13 and 9.9.14).

Mild Steel

Mild steel stirrups are vertical in the beam and located throughout the web at various spacings required by design for shear (see Figures 9.9.13 and 9.9.14). Alternatively, welded wire fabric may be used for shear reinforcement in these beams.

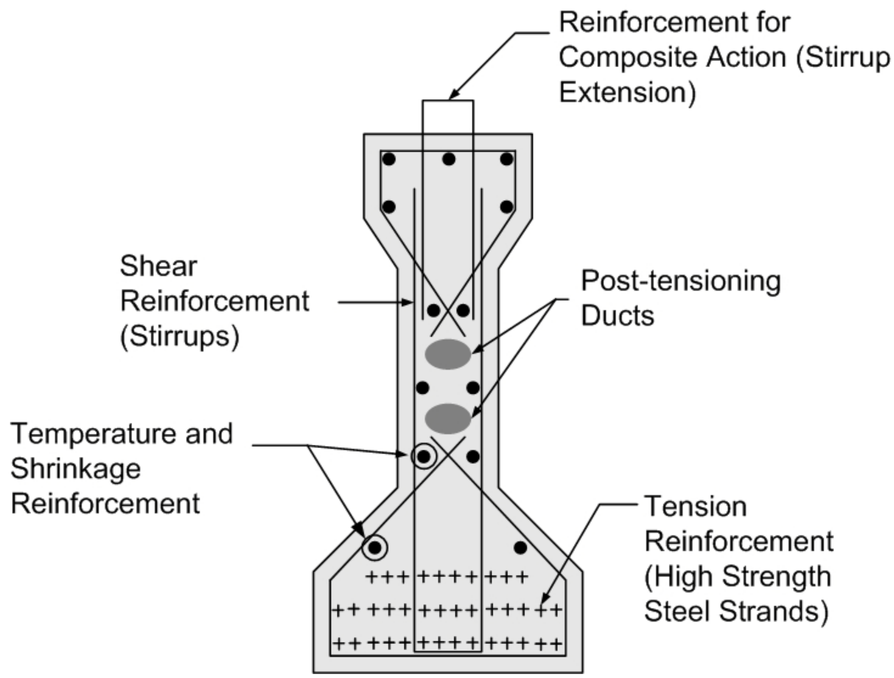

Figure 9.9.13 Prestressed I-beam Reinforcement

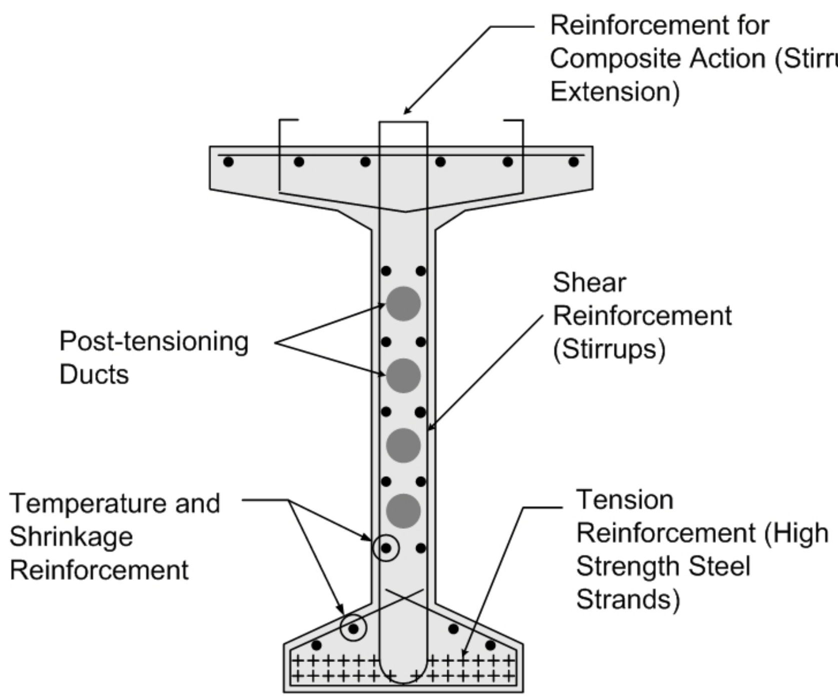

Figure 9.9.14 Prestressed Bulb-Tee Beam Reinforcement

Secondary Reinforcement

Secondary reinforcement includes mild steel temperature and shrinkage reinforcement which is longitudinal in the beam. Bulb-tee beams also contain transverse temperature and shrinkage steel in the top flange.

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.

9.9.3 Overview of Common Deficiencies

Common deficiencies that occur on prestressed I-beams and bulb-tees include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed presentation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.9.4 Inspection Methods and Locations

Inspection methods to determine causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of prestressed concrete I-beams and bulb-tees for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

Sounding by hammer can be used to detect delaminated areas. 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.

Since prestressed beams are designed to maintain all concrete in compression, cracks are indications of serious problems. Carefully measure any crack with an optical crack gauge or crack comparator card and document the results.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Electrical methods
- Delamination detection machinery
- Ground-penetrating radar
- Electromagnetic methods
- Pulse velocity
- Flat jack testing
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- Smart Concrete
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Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Check bearing areas for deficiencies such as delaminations, spalls or vertical cracks (see Figure 9.9.15). Deficiencies 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 placement (see Figure 9.9.16).

Check for crushing of flange near the bearing seat.

Check for rust stains which indicate corrosion of steel reinforcement.

Figure 9.9.15 Bearing Area of a Typical Prestressed I-beam

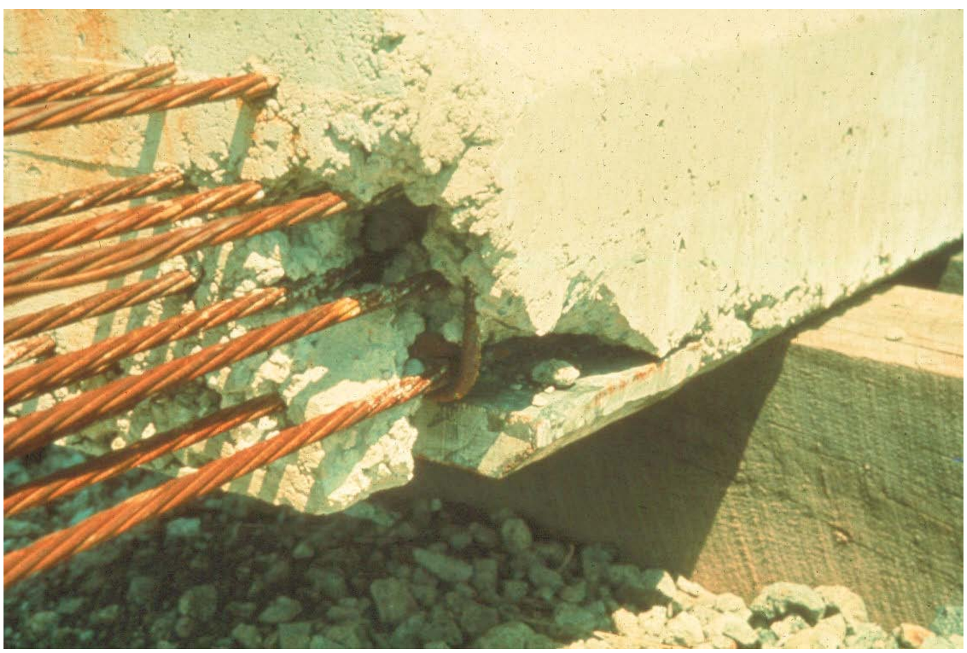

Figure 9.9.16 Spalling Due to Poor Concrete Placement

Shear Zones

Check beam ends and sections over substructure units for transverse cracks on the bottom flange and for diagonal shear cracks in webs. These web cracks will project diagonally upward from the support toward mid-span.

Tension Zones

Inspect the tension zones of the beams for structural cracks. Since these beams are designed to be in compression, cracking indicates a very serious problem resulting from overloading or loss of prestress. Check for rust stains from cracks, indicating corrosion of steel reinforcement or prestressing tendons.

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, unsymmetrical loading in the tendons, or possibly an overloading of the concrete due to use of prestressing strands that are too large.

Check bottom flange at mid-span for flexure cracks due to positive moment (see Figure 9.9.17). These cracks will be quite small and difficult to detect. Use an optical crack gauge or crack comparator card to measure any non-hairline cracks found.

For continuous bridges, check the deck area over the piers for flexure cracks due to negative moment.

Check for exposed tension reinforcement and document section loss on the tendons. Measurable section loss will decrease live load capacity. Exposed prestressing tendons are susceptible to stress corrosion and sudden failure.

Figure 9.9.17 Flexure Crack

Anchorage for Post-Tensioning System

Check for cracking propagating outward from anchor block or anchor plate on the exposed fascia on the beam webs. Document any cracks found and mark those cracks at the ends to monitor crack growth. Cracks allow moisture to access the transverse post-tensioning rods and accelerate corrosion and section loss of the rods.

Check for kinks, bulges or other deformities in the anchor block or anchor plate. This may be a result of improper installation of the post-tensioning system.

If the grout is visible, note the location and condition of the grout including color differences which may suggest that the initial grouting quality was poor.

Secondary Members

Inspect the end diaphragms for spalling or diagonal cracking (see Figure 9.9.18). This is a possible sign of overstress caused by substructure movement.

Inspect the intermediate diaphragms for cracking and spalling concrete. Flexure and shear cracks may indicate excessive differential movement of the primary superstructure members.

Figure 9.9.18 Concrete End Diaphragm

Areas Exposed to Drainage

Check around joints, scuppers, inlets or drain holes for leaking water or deterioration of concrete. Check the ends of beams that may be deteriorated due to water leaking through the deck joints (see Figure 9.9.19).

Figure 9.9.19 Leakage of Water at Joint between Spans

Areas Exposed to Traffic

Check areas damaged by collision. A significant amount of prestressed concrete bridge deterioration and loss of section is caused by 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, unless the bond between the concrete and steel reinforcement is affected (see Figure 9.9.20).

Figure 9.9.20 Inspectors Evaluating Collision Damage on Prestressed Concrete I-beam

Areas Previously Repaired

Examine thoroughly any repairs that have been previously made. Determine if repaired areas are sound and functioning properly. Effective repairs such as patching and epoxy injection of cracks are usually limited to protection of exposed tendons and reinforcement (see Figure 9.9.21).

Figure 9.9.21 Collision Damage Repair on Prestressed Concrete I-Beam. Note Epoxy Injection Ports and Gunite Repair

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Camber

Using a string line, check for horizontal alignment and vertical camber changes from the as-built condition of the prestressed beams. Signs of downward deflection usually indicate loss of prestress. Signs of excessive upward deflection usually indicate extreme creep and shrinkage.

Post-Tensioned Grout Pockets

Check the condition of the lateral post-tensioning grout pockets and visible ends of the post-tensioning rods. Cracked grout or rust stains may indicate a failure of the post-tensioning reinforcement.

Acute Angles on Skewed Bridges

Examine skewed bridges for lateral displacement and cracking of acute corners due to unsymmetrical strand release and insufficient reinforcement.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the prestressed beam. Cracking will typically be transverse in the thinner regions of the tee-beam and longitudinal near changes in cross section thickness.

Post-tensioning Tendon Lines

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 beam. This cracking may be the result of a bent tendon, a misaligned tendon with insufficient concrete cover or voids around the tendons. Shrinkage of concrete adjacent to large tendons has also caused this type of cracking.

In older post-tensioned bridges, problems due to insufficient grout placement in the conduit around the tendon have been reported. These voided areas can fill with water and the water will accelerate corrosion of the post-tensioning strands. In colder climates, the water may freeze and burst the conduit and surrounding concrete. Radiography and other nondestructive testing methods have been used successfully to locate these voids. These methods are also used to determine if the voids are present during construction of present-day bridges. If voids are found during construction, additional grout is added to eliminate these voids. States such as Florida have revised their polices for grouting materials and methods to eliminate these problems in bridges currently being constructed.

9.9.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

In an element level condition state assessment of a prestressed I-beam or bulb-T bridge, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Decks/Slabs	
15	Prestressed/Reinforced Concrete Top Flange* * Note that this element designation is used regardless of the type of riding surface
Superstructure	
109	Prestressed Concrete Girder/Beam

BME No.	Description
Wearing Surfaces and Protection Systems	
510	Wearing Surfaces
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

For bulb-tee beams without a topping slab, wearing surfaces, protection systems and protective coating, the unit quantity is square feet. The evaluation of the top flange element is based on the top and bottom surface condition of the top flange. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The quantity for the prestressed I-beam or bulb-tee beam is feet, and the total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the AASHTO *Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of prestressed I-beam and bulb-tee beam superstructures:

Defect Flag No.	Description
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

Table of Contents

Chapter 9
Inspection and Evaluation of Concrete Superstructures

Topic 9.10 Prestressed Box Beams

9.10.1 Introduction

Prestressed box beams are popular and have been used since the early 1950's (see Figure 9.10.1). These precast prestressed members provide advantages from a construction and an economical standpoint by increasing strength while decreasing the dead load.

Figure 9.10.1 Typical Box Beam Bridge

9.10.2 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 beam flanges, while the side walls act as webs. The prestressing reinforcement is typically placed in the bottom flange and into both webs (see Figure 9.10.2).

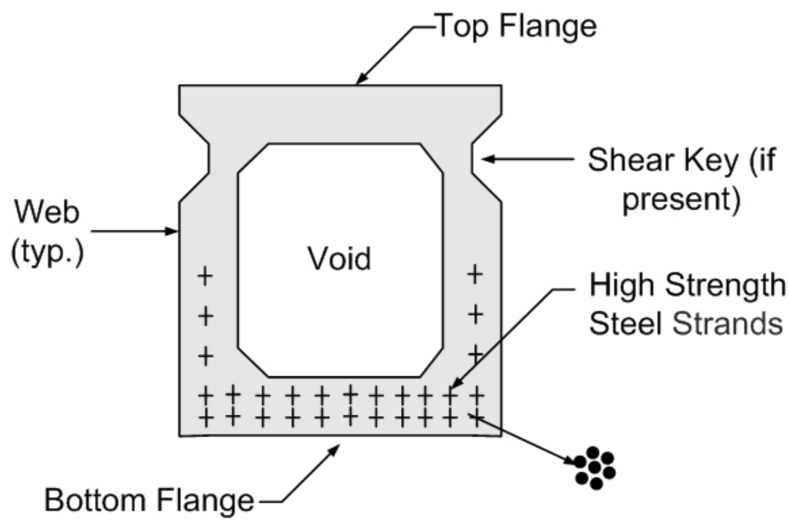

Figure 9.10.2 Box Beam Cross-Section

The typical span length for prestressed concrete box beams ranges from 20 to 90 feet depending on the beam size and their spacing.

Prestressed box beams are typically either 36 or 48 inches wide. The depth of a box beam ranges from 27 to 42 inches. Web wall thickness is normally 5 inches but can range from 3 to 6 inches.

Design

Simple/Continuous Spans

Prestressed box beams can be simple or continuous spans. In the case of simple spans, the ends of the beams at the piers are not connected together. 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 extending from the prestressed box beams (see Figure 9.10.3). Additional mild steel reinforcement is placed longitudinally in the deck to help achieve continuity. 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 considered composite or not composite. To obtain composite action, some prestressed box beams are constructed with stirrups extending out of the top flange (see Figures 9.10.3 and 9.10.10). These stirrups 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. This configuration can be considered composite since the compressive strength of the cast-in-place deck is significantly different than the precast prestressed box beams and the deck acts together with the beam to increase superstructure capacity.

Prestressed box beams can also not be considered composite. If the stirrups are not extended into the deck, the prestressed box beams cannot achieve composite action with the deck. Prestressed box beam bridges that do not utilize a concrete deck, but instead allow traffic to travel directly on the top flange or wearing surface, are always considered not to be composite.

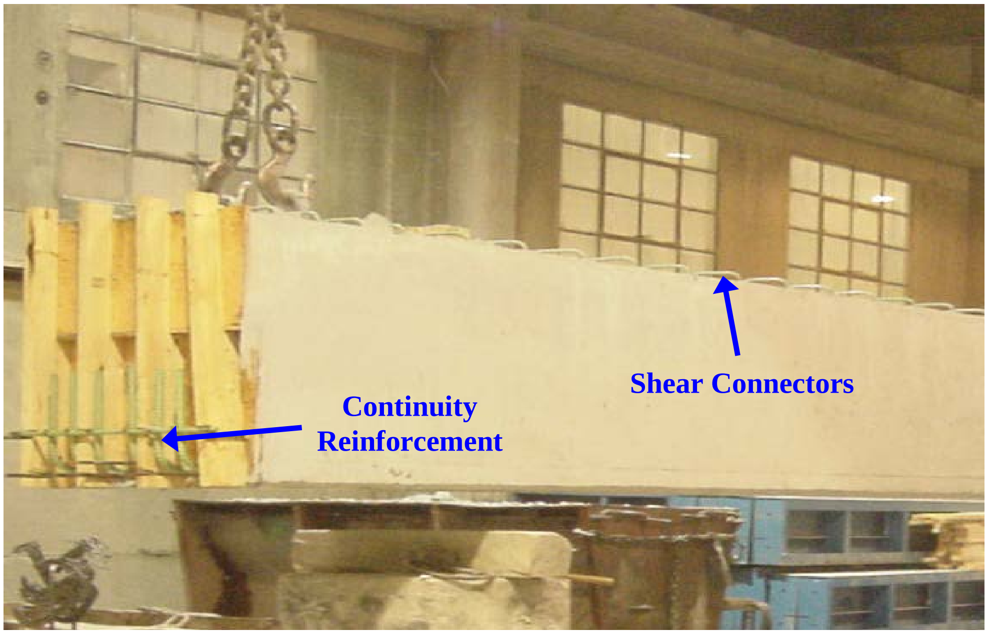

Figure 9.10.3 Box Beams at Fabrication Plant Showing Stirrups Extended as Shear Connectors and Extended Reinforcement for Continuity

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 270 ksi.

Concrete compressive strengths of 4000 to 8000 psi are typically used in prestressed box beams, but concrete with ultimate strengths up to 12,000 psi is available.

High Performance Concrete

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

The voided box beam reduces dead load while still providing flanges to resist the design moments and webs to resist the design shears.

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 structure to be erected with less traffic disruption than typical cast-in-place concrete construction.

Shallow Depth

Prestressed box beams are designed with a typical maximum depth of 42 inches. This shallow depth makes box beams viable solutions for field conditions where shallow vertical clearances exist.

Applications

There are two applications of prestressed box beams (see Figure 9.10.4):

- Adjacent box beams
- Spread box beams

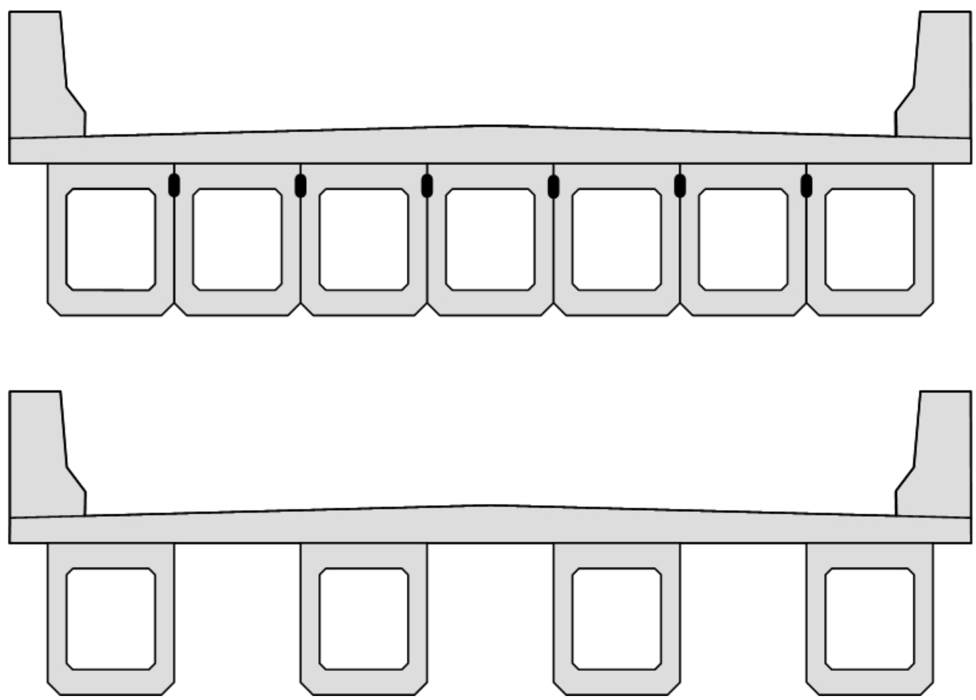

Figure 9.10.4 Prestressed Box Beam Cross Sections: Adjacent and Spread Box Beams

Adjacent Box Beams

On an adjacent box beam bridge, the adjacent box beams are placed side by side with no space between them. In some applications, the top flange of each box is exposed and functions as the deck (see Figure 9.10.5). The practical span lengths range from 20 to 130 feet.

Figure 9.10.5 Adjacent Box Beams: Top Flanges Acting as the Deck

In modern longer span applications, the deck is typically cast-in-place concrete and composite action with the box beam is achieved after the concrete hardens. For composite decks, stirrups extend above the top of the box to provide the transfer of shear forces. For the majority of shorter spans, nonstructural asphalt overlays are applied and are not considered composite. Sometimes a waterproofing membrane is applied prior to the overlay placement.

Monolithic Action

Like precast slab units, adjacent box beams are post tensioned transversely. This is generally done using 145 ksi threaded bars and lock nuts, or 270 ksi strands with locking wedges. Transverse post tensioning combined with grouted shear keys provides for monolithic action (see Figure 9.10.6).

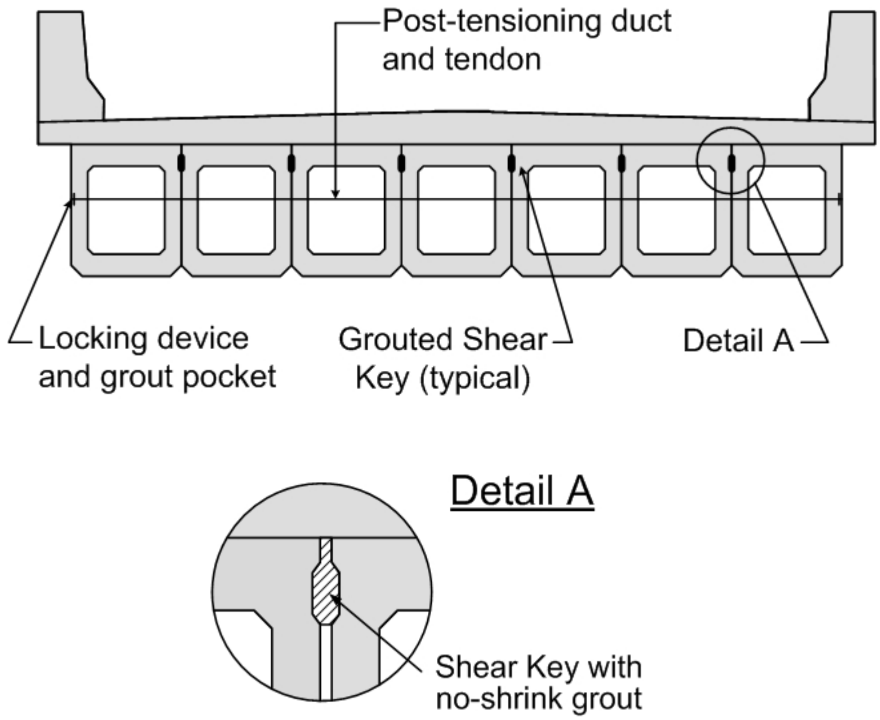

Figure 9.10.6 Transverse Post-tensioning of an Adjacent Box Beam Bridge

Spread Box Beams

On a spread box beam bridge, the box beams are usually spaced from 2 to 6 feet apart and typically use a composite cast-in-place concrete deck (see Figure 9.10.7). This application is practical for span lengths from 25 to 85 feet. Stay-in-place forms or removable formwork is used between the box beams to provide support for the concrete prior to curing.

Figure 9.10.7 Underside of a Typical Spread Box Beam

All modern box beams should have drain holes that are installed in the bottom slab during fabrication to allow any moisture in the void to escape.

Primary and Secondary Members

The primary members of box beam bridges are the prestressed 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 9.10.8). The diaphragms may be cast-in-place, precast, or steel and are placed at either the mid points or third points along the span and at the span ends. End diaphragms 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 usually partial depth.

Internal Diaphragms are considered a part of the prestressed box beams and not a secondary member (see Figure 9.10.9).

Figure 9.10.8 End and Intermediate Diaphragms on a Spread Box Beam Bridge

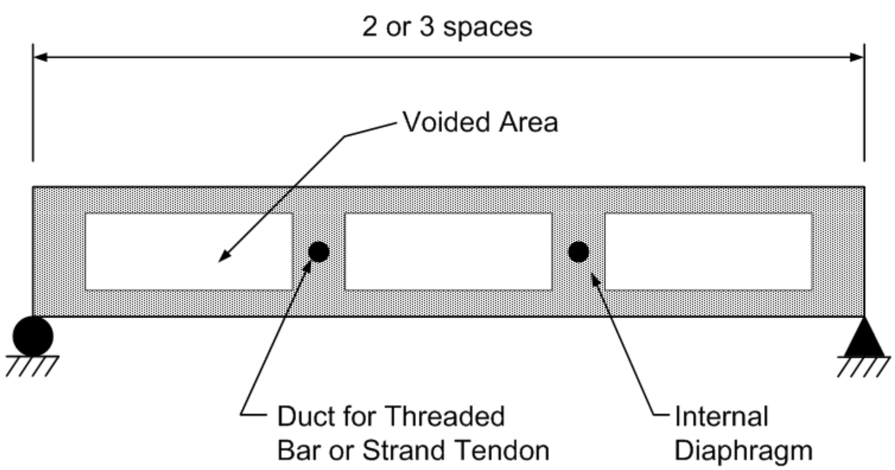

Figure 9.10.9 Schematic of Internal Diaphragms

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 flange and lower web of the box beam.

Depending on the age of the structure, the strand size will be 1/4, 3/8, 7/16, 1/2 inch in diameter and spacing is normally 2 inches apart (see Figure 9.10.10). In some newer applications of prestressed box beams using HPC, 0.6-inch strand sizes with a spacing of 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. Mild steel stirrups are more closely spaced near beam ends and typically Grade 60. Older designs may utilize 40 ksi reinforcement.

The current practice is to install the prestressing strands inside the shear stirrups (see Figure 9.10.10). Older designs (1960's) called for the stirrups to be placed between the prestressing strand rows (see Figure 9.10.26).

Secondary Reinforcement

Transverse post-tensioning strands through the diaphragms helps maintain monolithic action between the adjacent box beams. Temperature and shrinkage reinforcement consisting of mild steel is placed longitudinal in the beam webs and top flange.

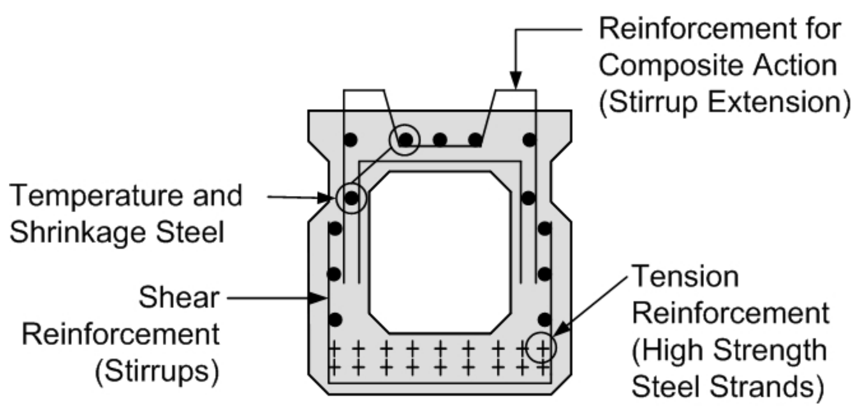

Figure 9.10.10 Typical Prestressed Box Beam Reinforcement

Fiber Reinforced Polymer 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. Refer to Topic 6.6 for more information.

9.10.3 Overview of Common Deficiencies

Common deficiencies that occur on prestressed box beams include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling

- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed presentation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

9.10.4 Inspection Methods and Locations

Inspection methods to determine other causes of concrete deficiencies are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of prestressed concrete box beams for surface cracks, spalls, downward camber and other deficiencies is primarily a visual activity.

Physical

Sounding by hammer can be used to detect delaminated areas. 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. In most cases, a chain drag is used to check the top surface of an exposed top flange.

Since prestressed box beams are designed to limit tensile stresses in concrete to specified thresholds, cracks are indications of serious problems. For this reason, carefully measure any crack with an optical crack gauge or crack comparator card and document the results.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

- Acoustic wave sonic/ultrasonic velocity measurements
- Electrical methods

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

Other methods or tests for material properties, described in Topic 15.2.3, include:

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

Locations

Bearing Areas

Check bearing areas for concrete delaminations, spalls or vertical/horizontal cracks. Spalls and cracks may be caused by corrosion of steel reinforcement due to water leakage or restriction of thermal movement due to a faulty bearing mechanism. Delaminations, spalls and cracks may also be caused by the stresses created at the transfer of the prestressing forces (see Figure 9.10.11).

Check for rust stains, which indicate corrosion of steel reinforcement (see Figure 9.10.12). This corrosion may accelerate due to lack of proper reinforcement bar cover.

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 box beam, or by the accumulation of water inside the box, freezing and thawing (see Figure 9.10.13).

Figure 9.10.11 Spalled Beam Ends with Exposed Prestressing Reinforcement

Figure 9.10.12 Exposed Shear Reinforcement at End of Box Beam

Figure 9.10.13 Longitudinal Cracks in Bottom Flange of Beam

Shear Zone

Check beam ends near abutments and piers for diagonal shear cracks in webs. These web cracks will project diagonally upward at approximately a 45 degree angle from the support toward midspan (see Figure 9.10.14). Transverse cracks along the beam bottom at these locations can also be an indication of overstress due to shear.

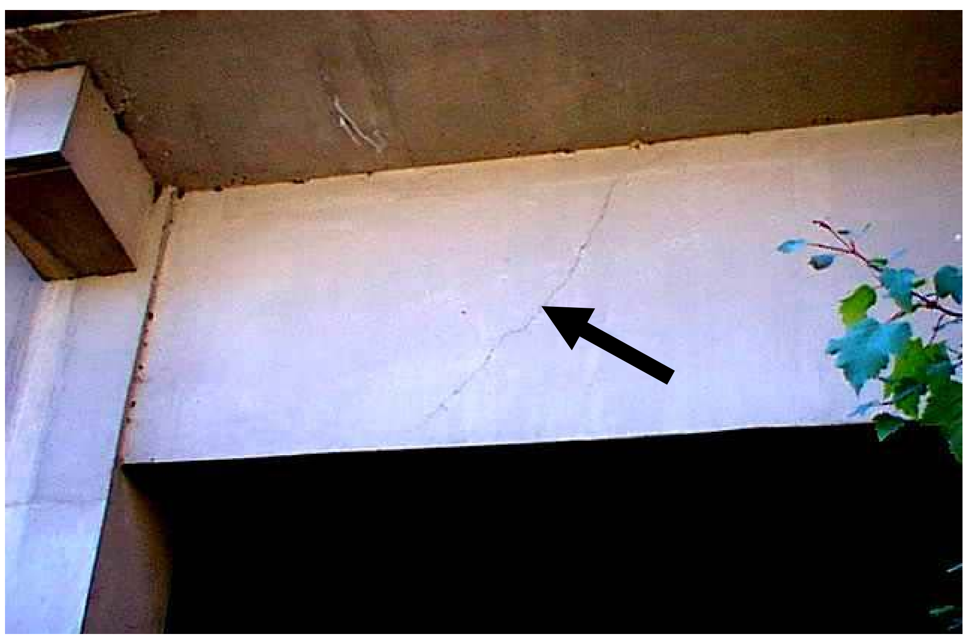

Figure 9.10.14 Diagonal Shear Crack in Web of Beam

Shear Keys

In adjacent box beam bridges, check each beam for independent vertical deflection, which may indicate failed shear keys, and lateral deflection on the exterior beams, which may indicate eccentric loading of the exterior beam. Inspect beam joints for signs of leakage between beams, which is indicative of deteriorated shear key grout material.

Anchorage for Post-Tensioning System

Check for cracking propagating outward from anchor block or anchor plate on the exposed fascia on the beam webs. Document any cracks found and mark those cracks at the ends to monitor crack growth. Cracks allow moisture to access the transverse post-tensioning rods and accelerate corrosion and section loss of the rods.

Check for kinks, bulges or other deformities in the anchor block or anchor plate. This may be a result of improper installation of the post-tensioning system.

If the grout is visible, note the location and condition of the grout including color differences which may suggest that the initial grouting quality was poor.

Tension Zones

Inspect the lower portion of the beam, particularly at mid span, for flexure cracks due to negative camber. This indicates a very serious problem resulting from overloading or loss of prestress.

Check for delaminations, spalls and exposed reinforcing steel. Exposed strands fail prematurely due to stress corrosion (see Figures 9.10.15 and 9.10.16).

Check for deteriorated concrete, which could cause debonding of the tension reinforcement. This would include spalls, delaminations, and cracks.

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 in the beam.

Figure 9.10.15 Spall and Exposed/Corroded Reinforcement

Figure 9.10.16 Close-up of Failed Strands due to Corrosion

Secondary Members

Inspect the end diaphragms of spread box beams for delaminations, spalling and cracking. Diagonal cracking is a possible sign of shear failure and can be caused by substructure movement.

Inspect the intermediate diaphragms of spread box beams for delaminations, spalls and cracks. Flexure and shear cracks may indicate excessive differential beam deflection.

Joints

Inspect joints for crushing and movement of the shear keys. The presence of open or loose joints 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 (if prestressed). The stress concentrations in these areas are very much different than a section away from the anchorages where a distributed stress pattern exists. Additionally, the effects of creep and tendon relaxation are somewhat higher in these regions.

Areas Exposed to Drainage

Examine joints between adjacent box beams for leakage and rust stains. Look for reflective cracking in the traffic surface and differential 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 9.10.17). These problems could also indicate the transverse post-tensioning is not acting as designed. The transverse post-tensioning may have failed due to section loss caused by water and de-icing agents leaking through the shear keys.

Check around scuppers and inlets for leaking water or deterioration of concrete. Check the underside of beam ends for leakage at the expansion joint areas and the fascia of exterior beams.

Drain Holes

Check drain holes for proper function as accumulated water can freeze and crack the beam. Older beams used cardboard to form the voids. The wet cardboard can clog the drain holes.

Figure 9.10.17 Joint Leakage and Rust Stain

Areas Exposed to Traffic

Check area 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, unless the bond between the concrete and steel reinforcement is affected (see Figure 9.10.18).

Figure 9.10.18 Close-up of Box Beam Collision Damage

Areas Previously Repaired

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.

Acute Angles on Skewed Bridges

Examine skewed bridges for lateral displacement and cracking of acute corners due to unsymmetrical strand release and insufficient reinforcement.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Camber

Using a string line, check for horizontal alignment and camber changes from the as-built condition of the prestressed beams. Downward deflection usually indicates loss of prestress or damage to the post-tensioning tendon, preventing cracks from closing. Excessive upward deflection usually indicates extreme initial prestressing forces or shrinkage. Note any changes in camber from previous report that may indicate problems.

Thermal Effects

These cracks are caused by non-uniform temperatures between two surfaces of the beam. Cracking will typically be transverse in the thinner regions of the deck and longitudinal near changes in cross section thickness (if applicable).

General

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.

9.10.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Items 58 and 59) for additional details about NBI component condition rating guidelines for decks and superstructures.

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

Element Level Condition State Assessment

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

NBE No.	**Description**
Decks/Slabs	
15	Prestressed/Reinforced Concrete Top Flange* * Note that this element designation is used regardless of the type of riding surface
Superstructure	
104	Prestressed Concrete Closed Web/Box Girder

BME No.	**Description**
Wearing Surfaces and Protection Systems	
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

For box beam bridges where the box girder top flange acts as a structural deck (see Figure 9.10.20), NBE No. 15, Prestressed/Reinforced Concrete Top Flange, is used. The unit quantity for this element, along with protection systems and protective coating, is square feet, with the total area distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for the prestressed box beam is feet, and the total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states must equal the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the AASHTO *Guide Manual for*

Bridge Element Inspection for condition state descriptions.

The following Defect Flags are applicable in the evaluation of prestressed box beam superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

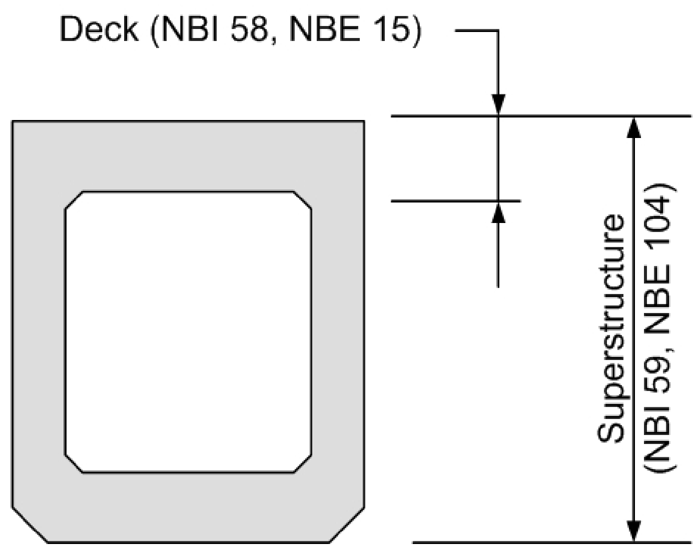

Figure 9.10.19 Components/Elements for Evaluation

9.10.6 Lessons Learned

Lake View Drive Bridge over Interstate I-70

General

Adjacent box beam bridges that are not considered composite have unique characteristics that must be considered during inspection. The following case history illustrates the potential for hidden deterioration that can exist in these types of bridges and locations where particular attention must be given. When combined with other factors, these deficiencies can lead to failure. In December 2005, the fascia beam of the Lake View Driver Bridge over Interstate 70 in Pennsylvania failed near mid-span (see Figures 9.10.20 and 9.10.21). The subsequent forensic investigation revealed a number of factors that contributed to the bridge fascia beam failure.

Figure 9.10.20 View Northeast of I-70 EB from Beneath Span 3

Figure 9.10.21 View East (Ahead Segments) from SR1014 Above Pier 2

Superstructure

The Lake View Drive bridge was a two lane, four span structure with Span 2 crossing the westbound lanes and Span 3 crossing the eastbound lanes of I-70. The beam that collapsed was the north elevation fascia beam of Span 3, which is designated as Beam 1. The bridge was constructed in 1960, comprised of eight precast prestressed adjacent box beams (42 inches deep by 48 inches wide) with no independent structural deck. Instead, the non-composite design incorporated a bituminous overlay (approximately 2.5 inches thick) that was placed directly on the beams without a waterproofing membrane. The structure measured 28'-0" curb to curb.

Following the collapse, the remaining superstructure beams (Beams 2 through 8) were visually inspected in the field for broken prestressing strands, structural cracks, joint leakage, and loss of prestress camber. Inspection findings included minor spalling, some exposed prestressing strands, some severed prestressing strands, and collision damage on the underside of Beams 2 and 3 of Span 3. Numerous scrape marks were measured up to 1.5 inches deep in these two beams. Additionally, joint leakage was evident from the bridge barrier, which allowed roadway runoff to travel down the exterior faces of the fascia beams and across the bottom flange. Leakage from the beam joints was also typical between the remaining beams, with the more concentrated leakage between the fascia and first interior beams (Beams 1 and 2, and 8 and 7). Otherwise, all remaining beams were in alignment with no structural cracking, loss of camber, or independent deflection to visually suggest a serious problem.

As per the shop drawings, Beam 1 was a Type 3 Beam with sixty 3/8 inch diameter prestressing strands located in the bottom flange and into the webs.

Collapsed Beam Findings

Samples were taken from both the failed beam and adjacent beams after the collapse. Material testing confirmed that both the strength of the concrete and prestressing steel were in fact greater than the design values (see Figure 9.10.22). The concrete mixture was also verified to be within specifications. It was determined that all beam construction materials tested did not contribute to the failure of Beam 1.

		Measured Value	
	Design Value	Minimum	Maximum
Concrete Strength, f'_c	5900 psi	6200 psi	8400 psi
Strand Tensile Strength, F_u	250 ksi	254 ksi	273 ksi

Figure 9.10.22 Post-Collapse Material Testing Assessment

Documented collision damage on the fascia beam existed near the point of fracture and resulted in several bottom strands being severed either directly from the collision or due to subsequent corrosion. Spalling was also noted in the vicinity of the collision damage which increased strand exposure to moisture and increased the extent of strand corrosion.

Post-collapse forensic laboratory testing concluded that a total of 39 strands in Beam 1 failed primarily due to environmental corrosion, 19 of which were not visible (see Figure 9.10.23). The sources of moisture contributing to the corrosion of all the strands appeared to be from roadway drainage escaping through a barrier deflection joint near the point of collapse and from the presence of typical moisture spray from traffic passing underneath the bridge. Longitudinal cracks on the bottom flanges allowed a single corroded strand to "transfer" the moisture which caused corrosion to the next row of prestressing strands (see Figure 9.10.24). Corrosion was also simultaneously transferred laterally to adjacent strands by corroding shear reinforcement stirrups (see Figure 9.10.25). This process allowed for corrosion and section loss of strands that were still completely encased in concrete. Wandering cracks induce corrosion of multiple strands in the same way (see Figure 9.10.26).

Strand corrosion resulted in a direct reduction in structural capacity.

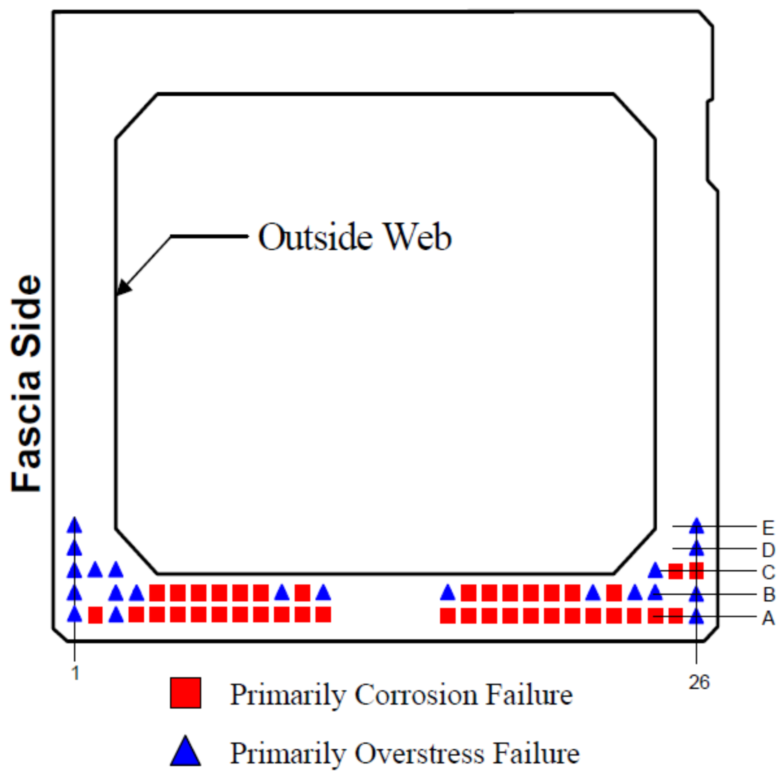

Figure 9.10.23 Post-Collapse Prestressing Strand Wire Fracture Laboratory Assessment

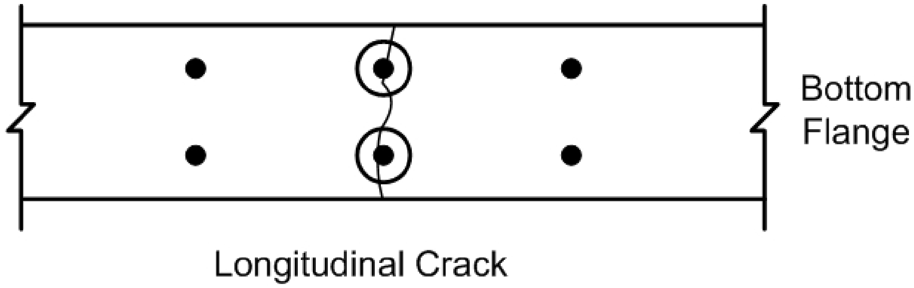

Figure 9.10.24 Prestressing Strand Corrosion and Section Loss Caused by Moisture Through Longitudinal Cracking

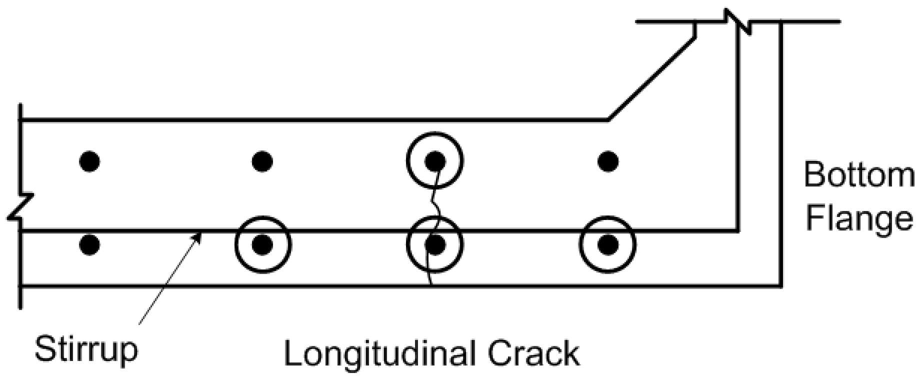

Figure 9.10.25 Prestressing Strand Corrosion and Section Loss Caused by Moisture Through Longitudinal Cracking and Shear Reinforcement Bars Transferring Moisture to Adjacent Longitudinal Reinforcement

Legend for Figures 9.10.24 and 9.10.25

◉ – Corroded prestressing strand

● – Non-corroded prestressing strand

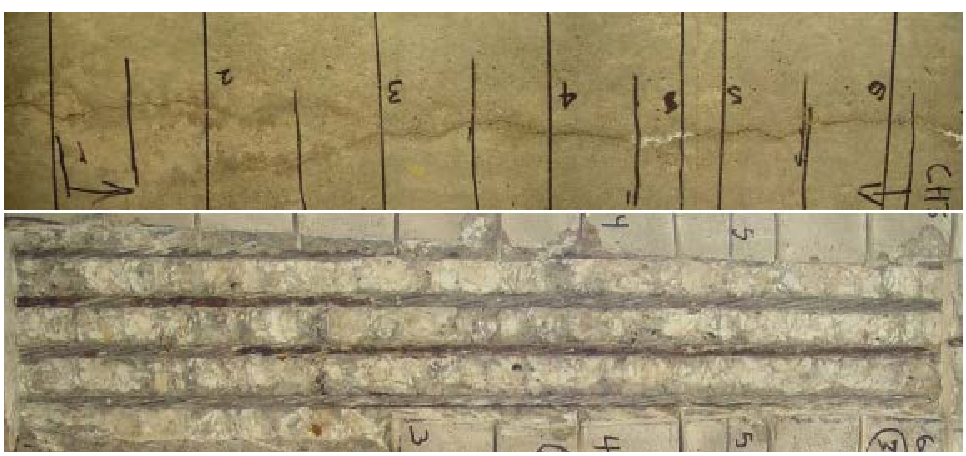

Figure 9.10.26 Longitudinal Cracks (Top) and Corresponding Corroded Prestressing Strands after Concrete Removed (Bottom)

Additional Inspection Locations and Requirements

The Lake View Drive Bridge Failure highlights additional inspection locations and requirements for adjacent box beam bridges.

Inspect areas near barrier joints for torsion-shear cracking. These areas on the fascia beams may be subjected to eccentric loading, which was determined to be a contributing factor in the failure of Beam 1. For the Lake View Drive Bridge, with the parapet located 7.25 inches from the outside face of the fascia beam, Beams 1 and 8 rely on shear keys to transform the eccentric dead load into vertical dead load. The vertical dead load is then resisted between multiple beams in the form of primary bending moment. Because the shear key between Beam 1 and 2 had failed, independent beam action was present and the parapet loading was eccentric. The diagonal shear crack in the exterior web of Beam 1 at midspan is indicative of excessive torsion-shear stress (see Figures 9.10.27 and 9.10.28). This overstress was the driving cause of the beam collapse.

Evaluate barriers and barrier connections, looking for water leakage through the barrier defection joints (see Figure 9.10.29). Because water has high chloride content due to application of years of de-icing agents, leakage into existing cracks will accelerate corrosion of the reinforcing steel and deterioration of the concrete, as with the Lake View Drive Bridge.

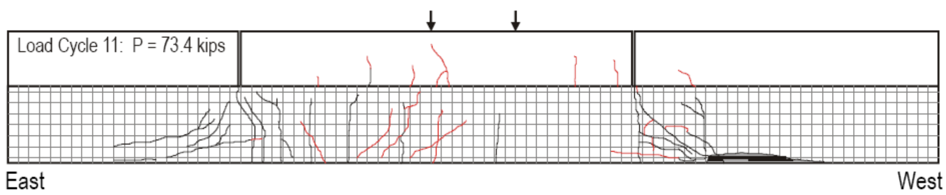

Figure 9.10.27 Laboratory Testing of Torsion-Shear Cracking Near Barrier Joints

Figure 9.10.28 Cracking Near Barrier Joints

Figure 9.10. 29 Water Leakage Through Parapet Deflection Joints

Unforeseen Fabrication Problems

Unforeseen fabrication problems also contributed to the Lake View Drive Bridge failure (see Figure 9.10.30). These problems are listed below:

- The minimum bottom flange thickness was found to be 86% of the design thickness.
- The average concrete cover over the strands was measured 0.87" average compared to 1- 9/16" as noted in the design drawings.
- The minimum wall thickness was measured to be 66% of the design value. It is thought the cardboard formwork shifted during fabrication. The decreased wall thickness reduces shear capacity of the beam.
- Lateral post-tensioning tie rods were heavily corroded, and there was poor consolidation of grout in the shear keys.
- Vent holes for curing in top flange were left open, and drain holes in the bottom flange were closed, resulting in an accumulation of moisture and even standing water.

Inspectors could not determine these fabrication problems without extensive use of nondestructive evaluation testing equipment. Prestressed concrete beams are currently constructed to tighter tolerances than 1960 (the date of fabrication for the beams in the Lake View Drive Bridge).

Figure 9.10.30 Unforeseen Fabrication Problems

Summary

In response to the findings, a revised and more detailed set of inspection methods and condition rating assessment guidelines are now followed for all adjacent box beam bridges that are not considered composite.

Focus on the barrier joint location for the presence of torsion and shear cracking. The torsion contribution comes from the eccentric loading of the barrier on the exterior beam and the loss of shear key resistance with the adjacent first interior beam.

Pay particular attention to the examination of fascia beams for present of cracking at drainage locations where runoff can come in contact with the sides and bottoms of the beams (see Figures 9.10.27, 9.10.28, and 9.10.29).

The effects of any longitudinal cracking in bottom strand locations, likely resulting in corrosion of non-visible strands including those above the bottom row, will now be accounted for in the condition assessment of adjacent box beam bridges that not considered to be composite.

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Table of Contents

Chapter 9
Inspection and Evaluation of Concrete Superstructures

Topic 9.11 Concrete Box Girders

9.11.1 Introduction

The popularity of box girder design is increasing. A trapezoidal box shape with cantilevered top flange extensions combines mild steel reinforcement and high strength post-tensioning tendons into a cross section capable of accommodating an entire roadway width. Both segmental and monolithic box girders are in service.

Older box girder bridges can be cast-in-place concrete with conventional steel reinforcement and post-tensioning reinforcement. Current designs for concrete box girders typically use post-tensioning (see Figures 9.11.1, 9.11.2 and 9.11.3).

Figure 9.11.1 Segmental Precast Concrete Box Girder Bridge

Figure 9.11.2 Cast-in-place Concrete Box Girder Bridge

9.11.2 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 9.11.3). Concrete box girder bridges are designed as either single span or continuous multi-span structures. Spans can have a straight or curved alignment and are generally in excess of 150 feet (see Figure 9.11.4).

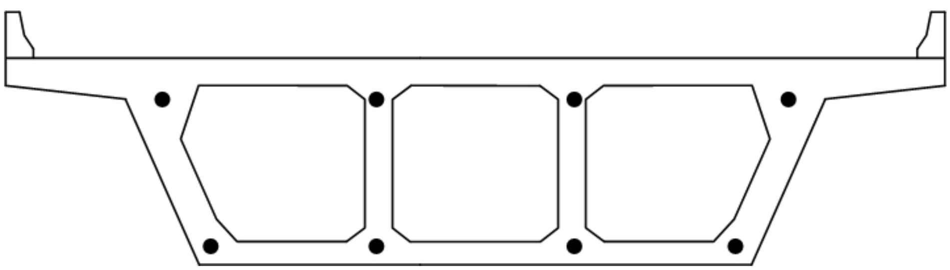

Figure 9.11.3 Multi-cell Girder: Post Tensioned

Construction Methods

The two basic construction techniques used for cast-in-place monolithic box girders are high level casting and at-grade casting.

The following description applies to monolithic box girder construction only. A detailed description of segmental concrete bridges appears later in this Topic.

Figure 9.11.4 Cast-in-place Concrete Box Girder Bridge

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 9.11.5).

Figure 9.11.5 High Level Casting Formwork on Falsework

At-grade Casting

The at-grade casting method employs formwork supported by fill material or the existing ground. When the construction is complete, the earth 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 9.11.6 and 9.11.7).

Figure 9.11.6 At-grade Formwork with Post-tensioning Ducts

Figure 9.11.7 At-grade Casting – After Supporting Earth Removed

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, the bottom flange, and both webs are all primary elements of the box girder (see Figure 9.11.8). The top flange is considered an integral deck component/element.

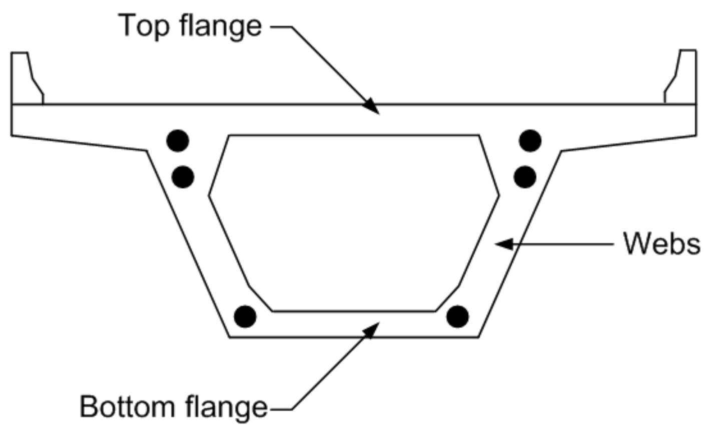

Figure 9.11.8 Basic Components/Elements of a Concrete Box Girder

In some multi-cell box girder applications, the top flange or deck must be removable for future replacement. The top flange in these cases functions similarly to an integral deck and is in fact considered a separate deck component/element. Most exterior webs are designed for higher stress levels than interior webs. The interior webs of the box also play a significant role in the box girder structure since they help support the deck (see Figure 9.11.9).

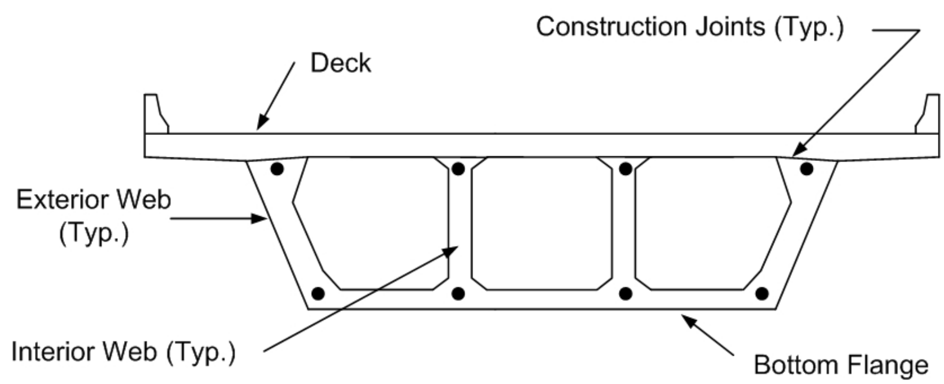

Figure 9.11.9 Replaceable Deck on a Multiple Cell Cast-in-place Box Girder

Steel Reinforcement

Box girder structures use a combination of primary mild steel reinforcement and high strength post-tensioning steel tendons to resist tension and shear forces (see Figure 9.11.10).

Tension reinforcement to resist flexure 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 9.11.6).

The top flanges or decks of precast or cast-in-place segmental boxes are often transversely post-tensioned. The multi-strand tendons in the webs and flanges are grouted after post-tensioning. The tendons anchor in block-outs in the edges of top slab cantilever wings. For precast units, the top flange tendons are generally tensioned and grouted in the casting yard. Wide bridges may have parallel twin boxes transversely post-tensioned. When this is the case, only about one-half of the transverse post-tensioning is stressed before shipment. The remainder of the post-tensioning is placed through ducts in adjacent box girders and the closure strip and stressed across the entire width of the bridge.

Conventional reinforcement bar stirrups in the web are 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.

The secondary (temperature and shrinkage) reinforcing steel is oriented longitudinally in the deck and webs and flanges in the box girder. The primary and secondary reinforcing steel for the deck portion of the girder is the same as for a standard concrete deck (see Figure 9.11.10).

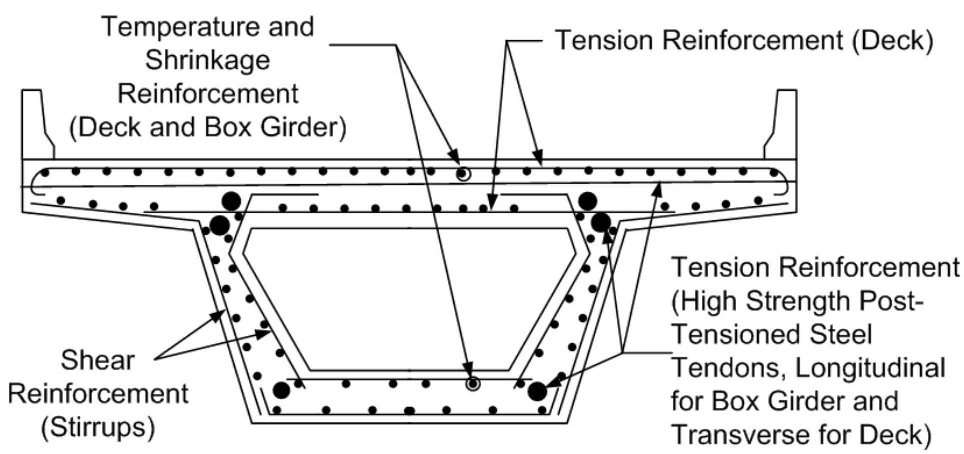

Figure 9.11.10 Primary and Secondary Reinforcement in a Concrete Box Girder

Special "confinement" reinforcement is also required at the anchorage locations to prevent cracking due to the large transfer of force to the surrounding concrete (see Figures 9.11.11 and 9.11.12).

Figure 9.11.11 Post-tensioning Spiral Anchorage Reinforcement Prior to Concrete Placement

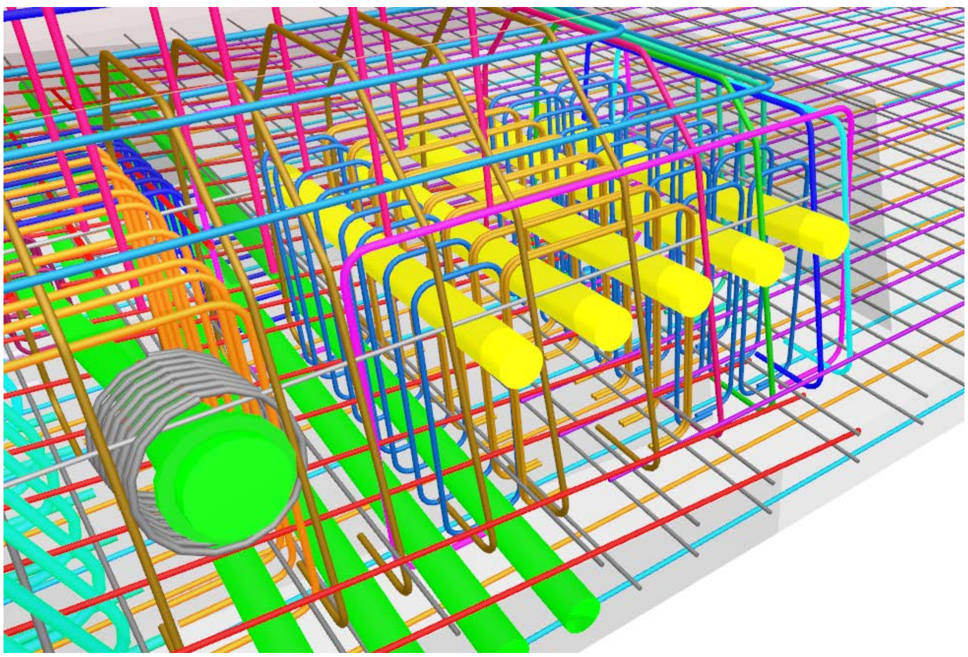

Figure 9.11.12 Three-Dimensional Model Illustrating Confinement Reinforcement Around Anchorage and Deviation Blocks (Click to Open Interactive Model)

Segmental Box Girder

Segmental box girders are similar to concrete box girders presented earlier in this subtopic. The proceeding portion of the subtopic highlights the similarities and especially the differences with concrete box girders.

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 Figures 9.11.14 and 9.11.15). The superstructure can be constructed of precast concrete or cast-in-place concrete segments. Several characteristics are common to most segmental bridges:

- Used for medium and long span bridges (spans can be as short as 130 feet, see Figure 9.11.14)
- 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 with a longitudinal cast-in-place concrete closure pour (see Figure 9.11.13)
- The length of the segments is determined by the construction methods, equipment available to the contractor, and local weight restrictions to transport the segments to the project site.
- Each new segment will be supported from previously erected segments during construction

Figure 9.11.13 Adjacent Single Cell Boxes with Closure Pour

Figure 9.11.14 Segmental Concrete Bridge

Figure 9.11.15 Close-up of Box Girder Segments

Segment Configurations

The majority of concrete segmental bridges use a box girder configuration (see Figure 9.11.16). The box girder is preferred due to the following:

- ➢ The top flange can be used as the roadway traffic surface (deck)
- ➢ The wide top and bottom flanges provide large areas to resist compression
- ➢ 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 deck/flange
- Bottom flange
- 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.

Figure 9.11.16 Box Girder Segment

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 segments (see Figure 9.11.17). This process proceeds until the bridge is completed.

Figure 9.11.17 Cast-in-place Box Girder Segment

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 an off-site 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 9.11.18).

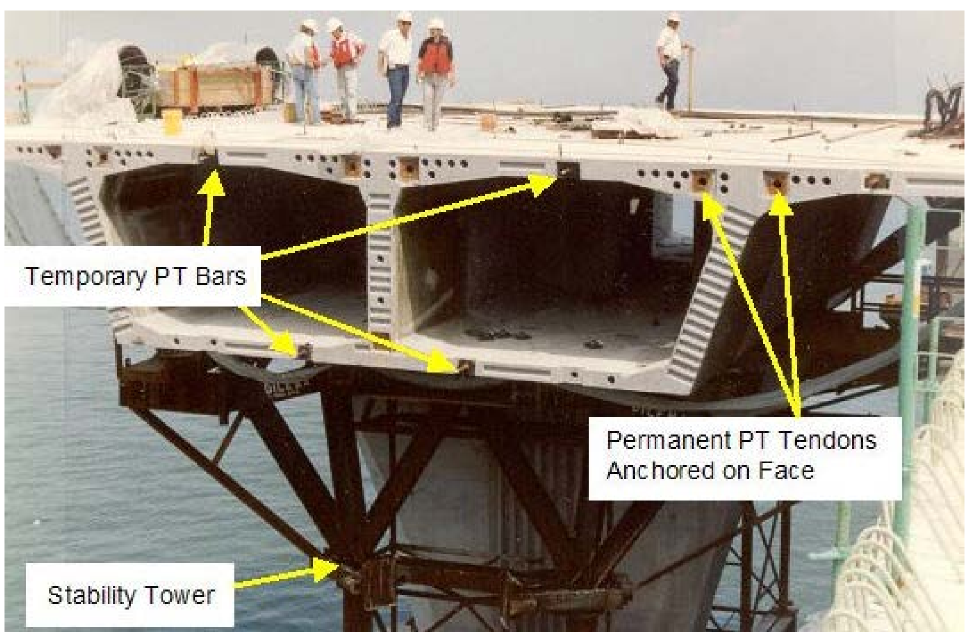

Figure 9.11.18 Box Girder Segment During Construction with Temporary and Permanent Post-Tensioning (PT) Bars.

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 the formwork for the next segment to ensure proper mating segments. Epoxy bonding adhesive is applied to the match-cast joints during initial erection.

Cast-in-place construction frequently does not benefit from 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 at the pier by the cantilever segments tend to balance each other. Once the mid-span is reached, a closure segment is cast together with the previously erected half-span from the adjacent pier. This method is repeated until all the spans have been erected (see Figures 9.11.19, 9.11.20 and 9.11.21). Both cast-in-place and precast construction is suitable for balanced cantilever construction.

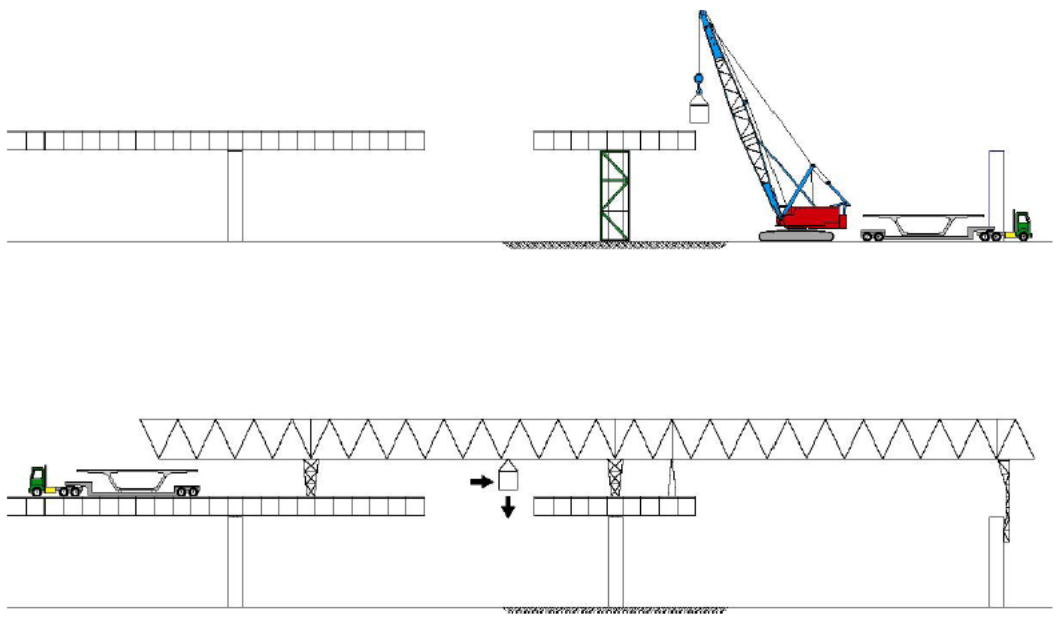

Figure 9.11.19 Two Balanced Cantilever Methods - Using Cranes with Stability Towers At Each Pier and Using An Overhead Launching Gantry

Figure 9.11.20 Balanced Cantilever Construction Using An Overhead Launching Gantry

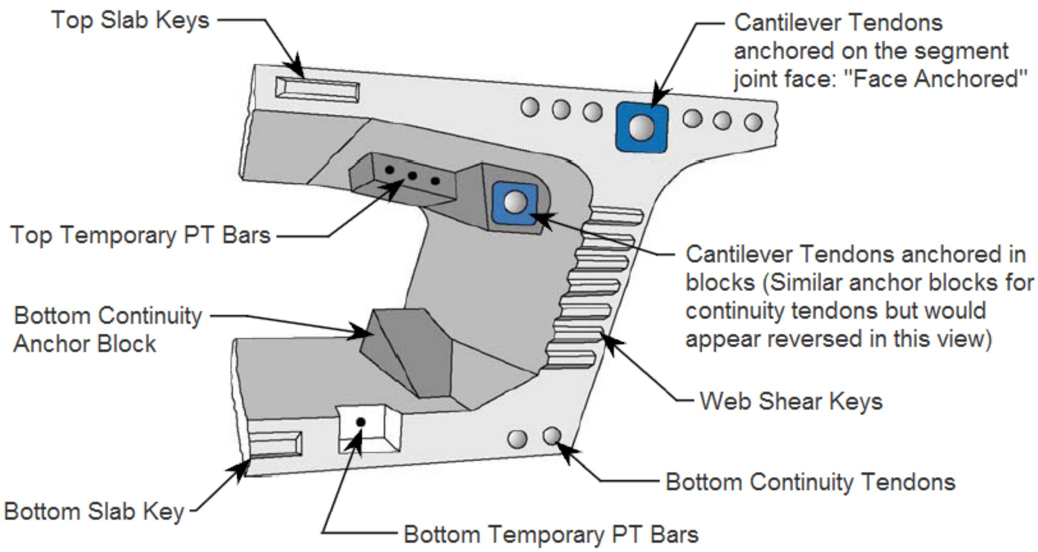

Figure 9.11.21 Typical Features of Precast Cantilever Box Girder Segments

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 method is repeated until all the spans have been erected (see Figures 9.11.22 and 9.11.23).

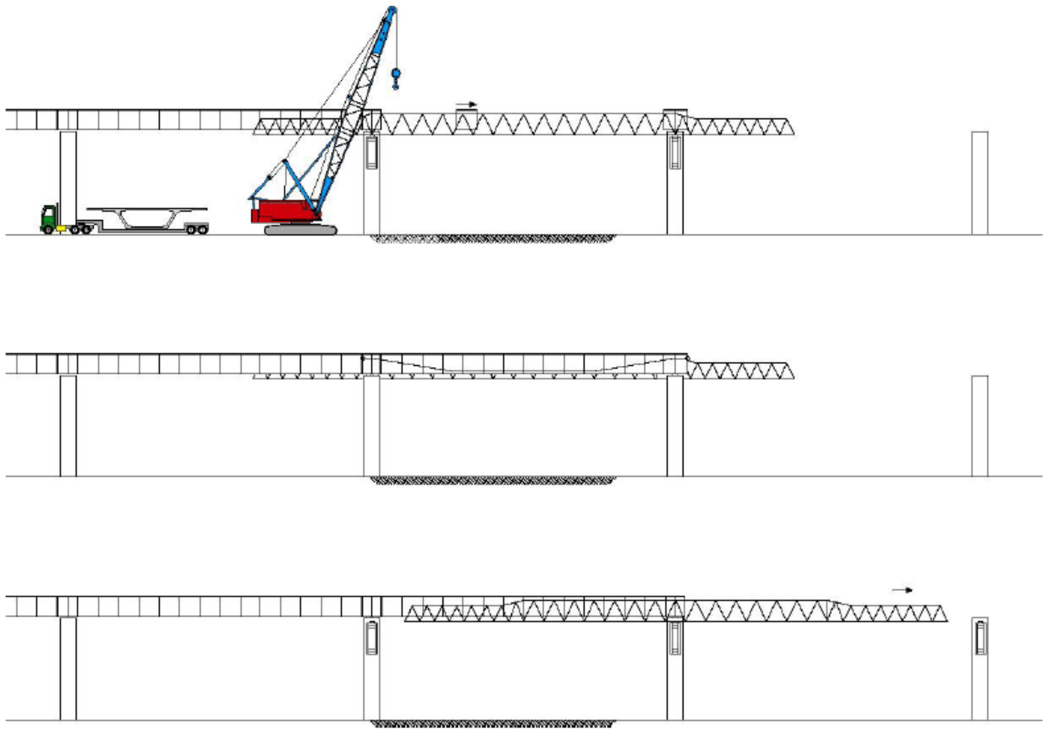

Figure 9.11.22 Span-by-Span Construction (with Erection Truss)

Figure 9.11.23 Span-by-span Construction (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 cranes or 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 9.11.24).

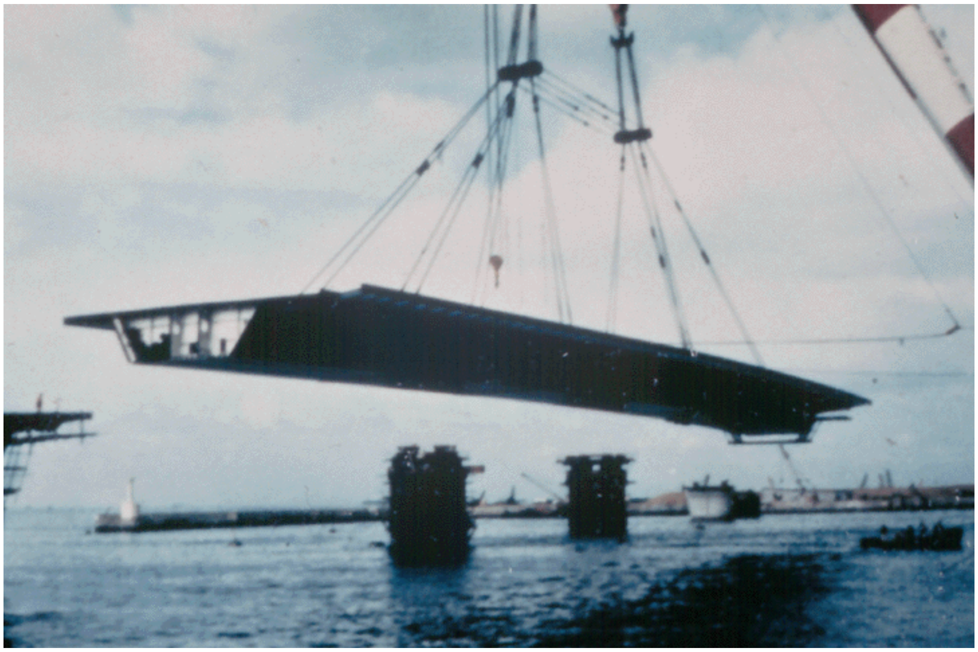

Figure 9.11.24 Span-by-span Total Span Erection (Lifting)

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 (see Figure 9.11.25). 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.

Figure 9.11.25 Progressive Placement Construction

Incremental Launching Construction

This form of construction permits the individual segments to be fabricated or positioned behind an abutment and then launched forward towards an adjacent pier by means of hydraulic jacks. Both cast-in-place and precast construction is suitable for this type of construction. This process is repeated until the entire bridge is constructed (see Figure 9.11.26).

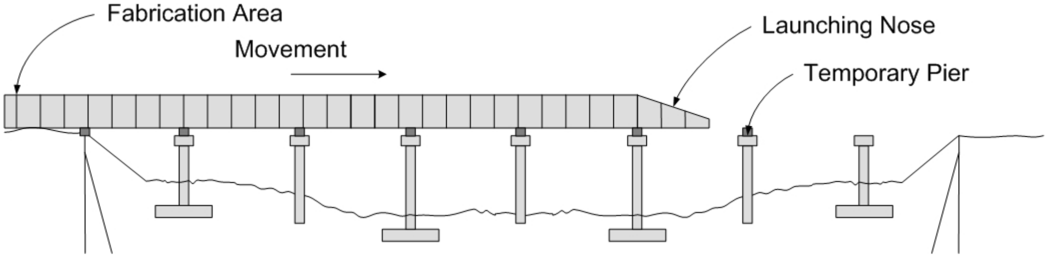

Figure 9.11.26 Incremental Launching Method

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 9.11.27).

Figure 9.11.27 Incremental Launching Overview (Note Temporary Pile Bent)

9.11.3 Overview of Common Deficiencies

Common deficiencies that occur on concrete box girder bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

9.11.4 Inspection Methods and Locations

Inspection methods to determine other causes of concrete deterioration are presented in detail in Topic 6.2.8.

Methods

Visual

The inspection of prestressed concrete box girders for surface cracks, spalls, and other deficiencies is primarily a visual activity.

Physical

Sounding by hammer can be used to detect delaminated areas. 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. In most cases, a chain drag is used to check the top surface of a concrete deck or top flange.

Since prestressed box girders are designed to limit tensile stresses in concrete to specified thresholds, cracks are indications of serious problems. For this reason, carefully measure any crack with an optical crack gauge or crack comparator card and document the results.

Advanced Inspection Methods

Several advanced methods are available for concrete inspection. Nondestructive methods, described in Topic 15.2.2, include:

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

- Smart concrete
- Carbonation

Other inspection methods or tests for material properties, described in Topic 15.2.3, include:

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

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 in 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 or initial inspection. This inspection will serve as a benchmark for all future inspections. Since the initial inspection is so important, schedule it 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.

Bearing Areas

Check the bearing area for delaminations, spalls and cracks. Delaminations, spalls and cracks may be caused by corrosion of steel reinforcement due to water leakage or restriction of thermal movements.

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 9.11.28).

Figure 9.11.28 Bearing Area of a Box Girder Bridge

Shear Zones

Check girder ends and sections close to piers for diagonal shear cracks in webs. These web cracks will project diagonally upward at approximately a 45 degree angle from the support toward midspan (see Figure 9.11.29).

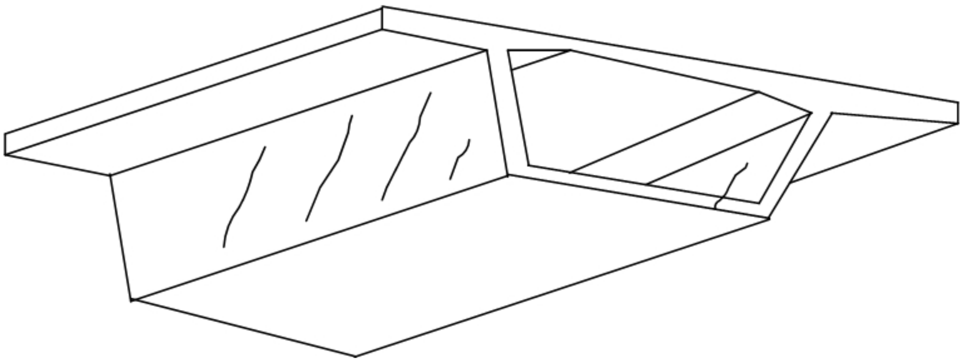

Figure 9.11.29 Box Girder Cracks Induced by Shear

Tension Zones

Direct Tension **-** Tension cracks can appear as a series of parallel cracks running transverse to the longitudinal axis of the bridge. The duct cracks are normally located on both sides of the longitudinal or neutral axis. 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 component (see Figure 9.11.30).

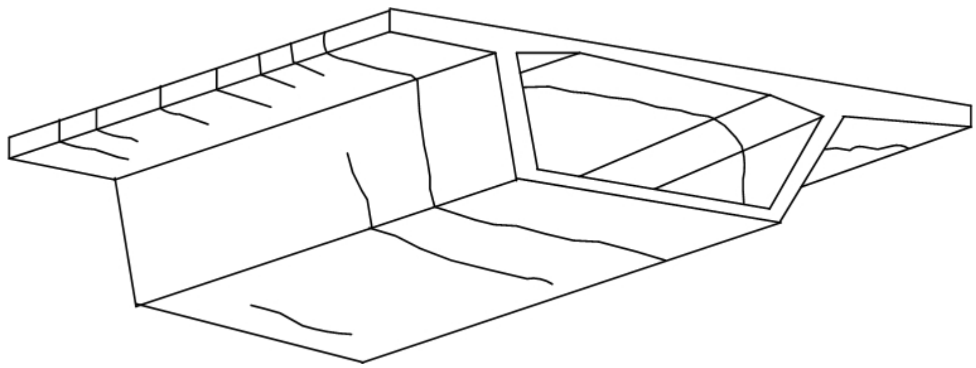

Figure 9.11.30 Box Girder Cracks Induced by Direct Tension

Flexure - These cracks can 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 the neutral axis or to an area around the half-depth of the section. Examine flexural cracks found in post-tensioned members very carefully. This could indicate that the member is overstressed. Accurately identify the location of the crack, the length and width of the crack, and the spacing to adjacent cracks (see Figures 9.11.31 and 9.11.32).

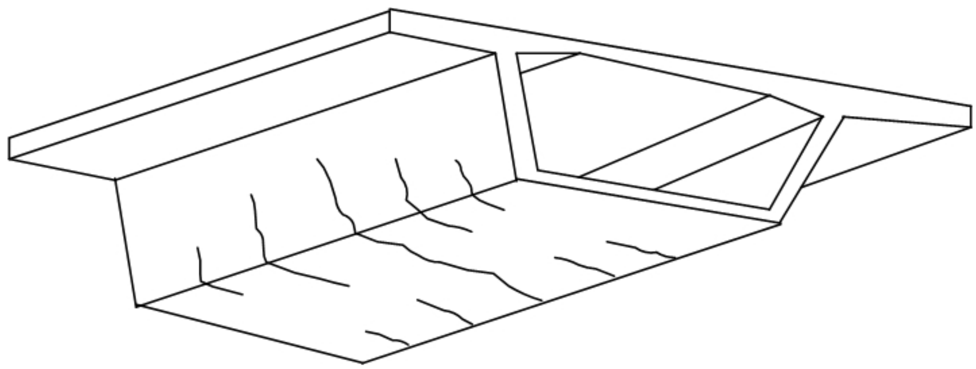

Figure 9.11.31 Box Girder Cracks Induced by Flexure (Positive Moment)

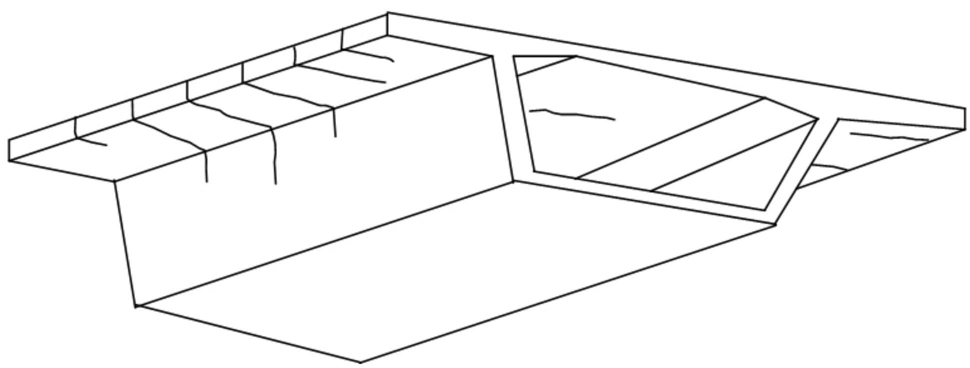

Figure 9.11.32 Box Girder Cracks Induced by Flexure (Negative Moment)

Flexure-shear **-** These cracks can appear close to pier support locations. They initiate on the bottom flange oriented transverse to the longitudinal axis of the bridge. The cracking will propagate up the webs approximately 45 degrees to the horizontal and toward mid-span (see Figure 9.11.33).

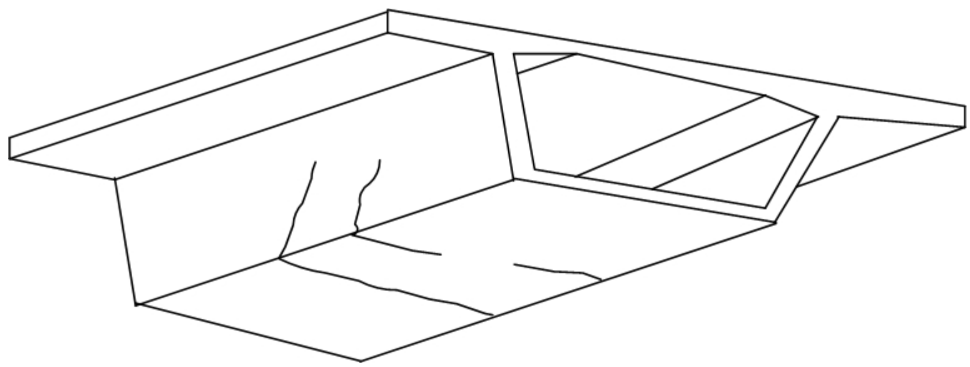

Figure 9.11.33 Box Girder Cracks Induced by Flexure-shear

Inspect the top side of the top flange for longitudinal flexure cracking directly over interior and exterior girder webs. Inside the box, examine the bottom of the top flange for longitudinal flexure cracking between the girder webs. These longitudinal cracks are caused by overstressing of the deck. Document any efflorescence or leakage through the top flange.

Inspect the girder 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 cracks may be present. As with all prestressed concrete members, carefully measure any cracks with an optical crack gauge or crack comparator and document its location, length, width, and crack spacing.

Anchor Blocks

Anchor blocks (or blisters) 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 or if there are voids adjacent to the post-tensioning tendons. 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 9.11.34).

Deviation Blocks

Also known as "deviation saddles", deviation blocks allow longitudinal post-tensioning tendons to change direction or angle within the box girder (see Figures 9.11.35 and 9.11.47). Deviation blocks allow free longitudinal movement through the ducts, while still acting as holding points for the longitudinal post-tensioning tendons. Additionally, deviation blocks may be used with temporary post-tensioning tendons to maintain alignment of the tendon (see Figure 9.11.36). As with anchor blocks, carefully examine deviation blocks since these are points of very high stress concentrations. Locating and identifying areas of delamination, spalling and cracking is essential due to the structural importance of this component.

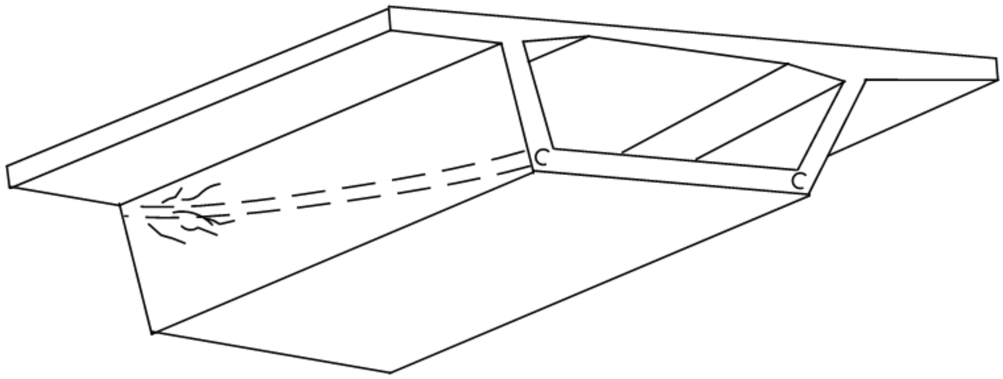

Figure 9.11.34 Web Splitting near an Anchorage Block

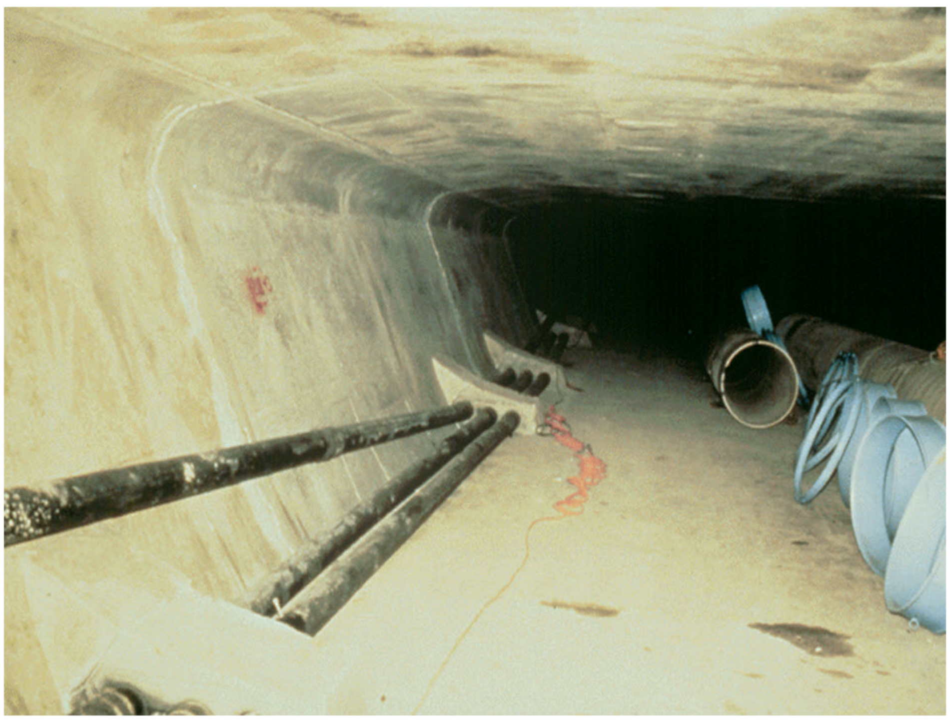

Figure 9.11.35 Deviation Block Used as a Hold-Down Point for External Post-Tensioning

Figure 9.11.36 Temporary Deviation Blocks Used to Maintain Tendon Alignment During Construction

Internal Diaphragms

Internal diaphragms at the piers and abutments serve to stiffen the box section and to distribute the large bearing reaction loads. Tendon anchorages located within the diaphragm will also contribute to additional post-tensioning loads. This region of the structure is very highly stressed and, therefore, prone to crack development. The internal diaphragms require close examination during inspection (see Figure 9.11.48).

Secondary Members

If there are external diaphragms between the girders, check for delaminations, spalls and cracking. Deficiencies on end diaphragms may indicate differential substructure settlement while deficiencies in the intermediate diaphragms may indicate excessive deflection in the girders.

Areas Exposed to Drainage

Examine the box girder for any delaminations, spalling, or scaling which may lead to exposure of reinforcing steel. Give special attention to areas such as joints, scuppers and curb lines exposed to drainage.

Drain Holes

Drain holes are typically provided at the low point of each box girder "cell" (see Figure 9.11.37). They function to allow water to drain from inside the box girder. To prevent the entrance of unwanted wildlife, screens are often placed on the inside of the box girder. Inspect these devices for missing screens or clogging due to debris.

Figure 9.11.37 Concrete Box Girder Drain Hole with Screen

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 tendons, the length of exposed tendons, number of severed strands, the extent of, as well as the loss of, concrete section. The loss of concrete due to such a collision is not always serious, unless the bond between the concrete and steel reinforcement is affected.

Inspection of the roadway surface for delaminations, cracking, spalling, and deformation; the presence of these deficiencies can increase the impact effect of traffic. This may be of greater significance if the top flange does not have an added wearing surface.

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.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Post-Tensioned Grout Pockets

Check the condition of the lateral post-tensioning grout pockets and visible ends of the post-tensioning rods. Cracked grout or rust stains may indicate a failure of the post-tensioning rod or loss of monolithic action.

Camber

Check the camber of the box girders. Loss of positive camber indicates loss of prestress in the tendons.

Miscellaneous Areas

Cracks Caused by Torsion and Shear **-** This type of cracking will occur in both the flanges 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 girder due to the additive effects of all forces (see Figure 9.11.38).

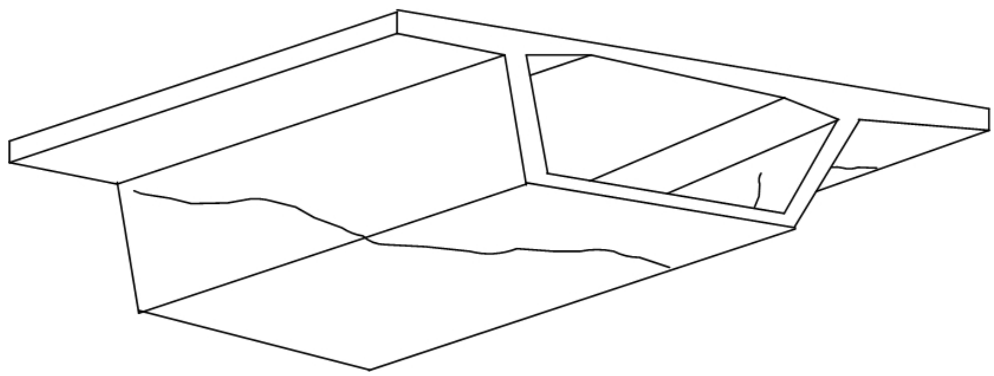

Figure 9.11.38 Box Girder Cracks Induced by Torsion and Shear

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 flanges of the box and longitudinal near changes in cross section thickness (see Figures 9.11.39 and 9.11.40).

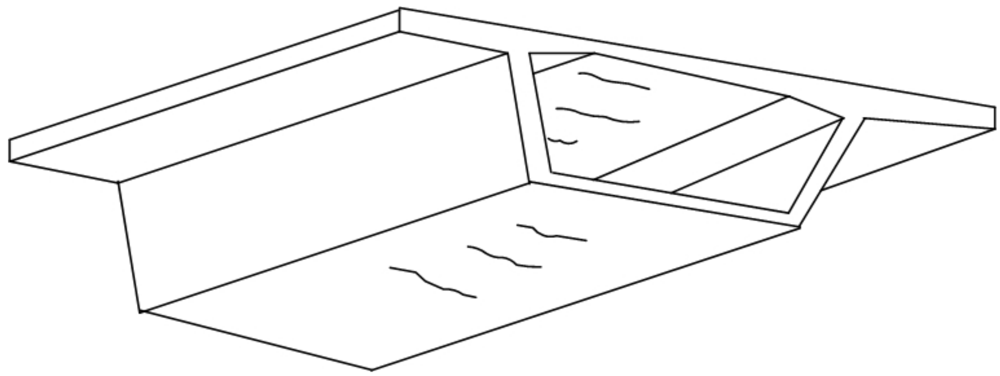

Figure 9.11.39 Thermally Induced Transverse Cracks in Box Girder Flanges

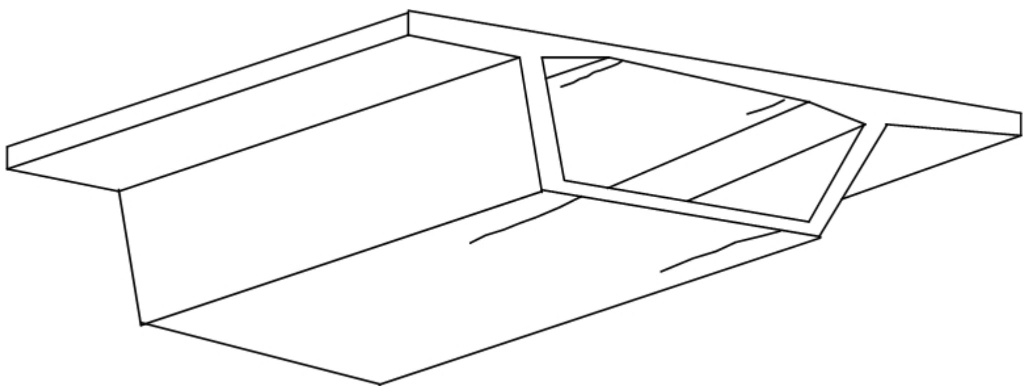

Figure 9.11.40 Thermally Induced Longitudinal Cracks at Change in Box Girder Cross Section

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 Figures 9.11.34 and 9.11.41). This cracking may be the result of a bent tendon, a misaligned tendon with insufficient concrete cover or voids around the tendons. Shrinkage of concrete adjacent to large tendons has also caused this type of cracking.

In older post-tensioned bridges, problems due to insufficient grout placement in the conduit around the tendon have been reported. These voided areas can fill with water and the water will accelerate corrosion of the post-tensioning strands. In colder climates, the water may freeze and burst the conduit and surrounding concrete. Radiography and other nondestructive testing methods have been used successfully to locate these voids. These methods are also used to determine if the voids are present during construction of present-day bridges. If voids are found during construction, additional grout is added to eliminate these voids. States such as Florida have revised their polices for grouting materials and methods to eliminate these problems in bridges currently being constructed.

Figure 9.11.41 Post-tensioning Tendon Duct

Unintentional load path **-** Inspect older cast-in-place box girder interiors 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 9.11.42). Loads from the deck may be directly transferred to the bottom flange. This was not the intent of the original design.

Figure 9.11.42 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. Establish permanent survey points at each substructure and at each mid-span. Likewise, several points need to be set at each of these locations in the transverse direction across the deck (see Figure 9.11.43). During the inspection:

- Inspect the girder for the proper camber by sighting along the fascia of the bottom flange.
- On curved box girders, check for irregularities in the superelevation of the flanges, which could indicate torsional distress.

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 components 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.

Investigate unusual noises, such as banging and screeching, which may be a sign of structural distress.

Observe and record data from any monitoring instrumentation (e.g., strain gauges, displacement meters, or transducers) that has been installed on or within the bridge.

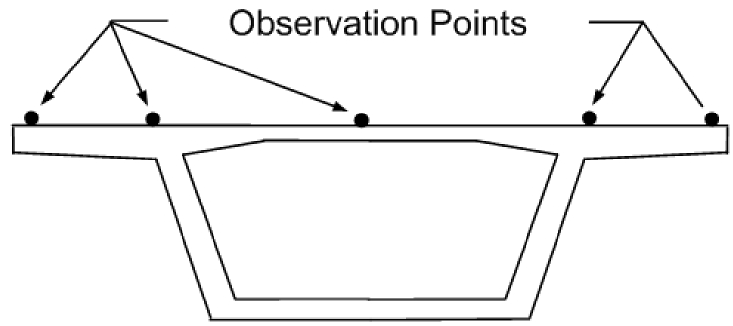

Figure 9.11.43 Location of Observation Points Across the Top Flange

Locations - Segmental Box Girder

In addition to the inspection locations and methods for concrete box girders, there are several special components that are unique to segmental bridges. The bridge inspector needs to be familiar with these special components.

Inspecting a segmental box girder bridge is similar to the methods mentioned previously. This is described in Topic 6.2.8 and this format is consistent with similar topics. for concrete box girders, and includes the following specific methods:

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 9.11.44).

Figure 9.11.44 Segmental Box Girder Bearings at Intermediate Pier

Shear and Tension Zones

Inspect both the interior and the exterior surfaces of the box girder. The inspection methods 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 9.11.29 to 9.11.34 and Figures 9.11.38 to 9.11.40.

Anchor Blocks

Segmental construction relies on the tremendous post-tensioning forces to hold the individual segments together. 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 flange or web on which the anchor block is located will require attention concerning the potential for transverse cracking in the vicinity of the anchor (see Figure 9.11.45).

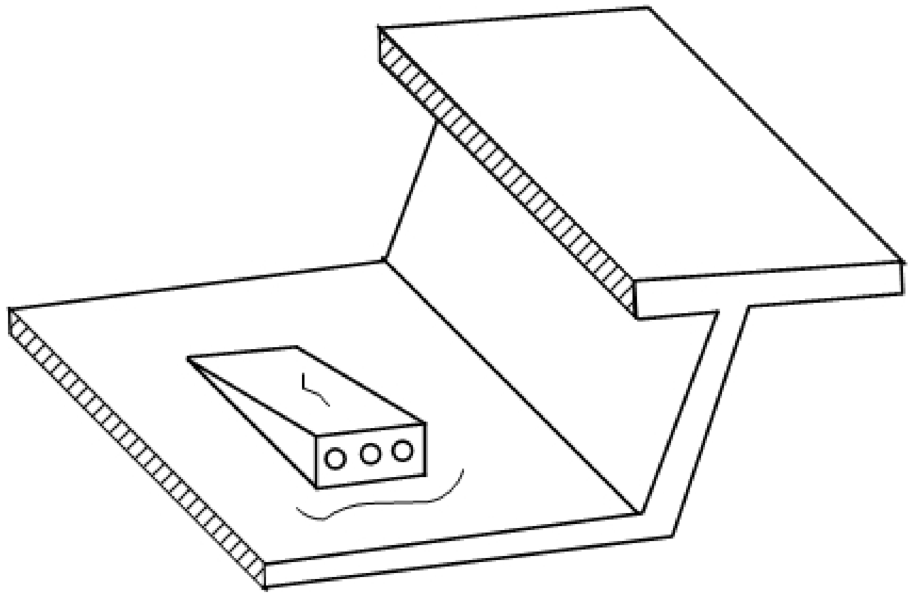

Figure 9.11.45 Segmental Box Girder Cracks Adjacent to Anchorage Block

Deviation Blocks

As with cast-in-place concrete box girder bridges, segmental construction often relies on draped or "harped" strand patterns to counter negative moment near intermediate supports. Deviation blocks (or saddles) are used to change the direction or angle of longitudinal post-tensioning tendons while allowing free longitudinal movement through the duct (see Figures 9.11.35 and 9.11.47). Additionally, deviation blocks may be used with temporary post-tensioning tendons to maintain alignment of the tendon (see Figure 9.11.36). Inspection of deviation blocks for segmental box girder bridges is the same as for concrete box girder bridges.

Secondary Members

Internal diaphragms are located at abutments and piers. Closely examine these members including the tendon anchorages located within them. The diaphragms stiffen the box section and distribute large bearing reaction loads. Examine this high stress region closely for cracks.

If there are external diaphragms between the box girders, check for cracking and spalling. Deficiencies on end diaphragms may indicate differential substructure settlement while deficiencies in the intermediate diaphragms may indicate excessive deflection in the girders.

Joints

Inspect joints for crushing and movement of the shear keys (see Figures 9.11.15 and 9.11.46). The presence of open or loose joints 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 9.11.47).

Additionally, the effects of creep and tendon relaxation are somewhat higher in these regions.

Closely examine the joints between the segments for any signs of leakage or infiltration.

Figure 9.11.46 Close-up View of Box Girder Joint

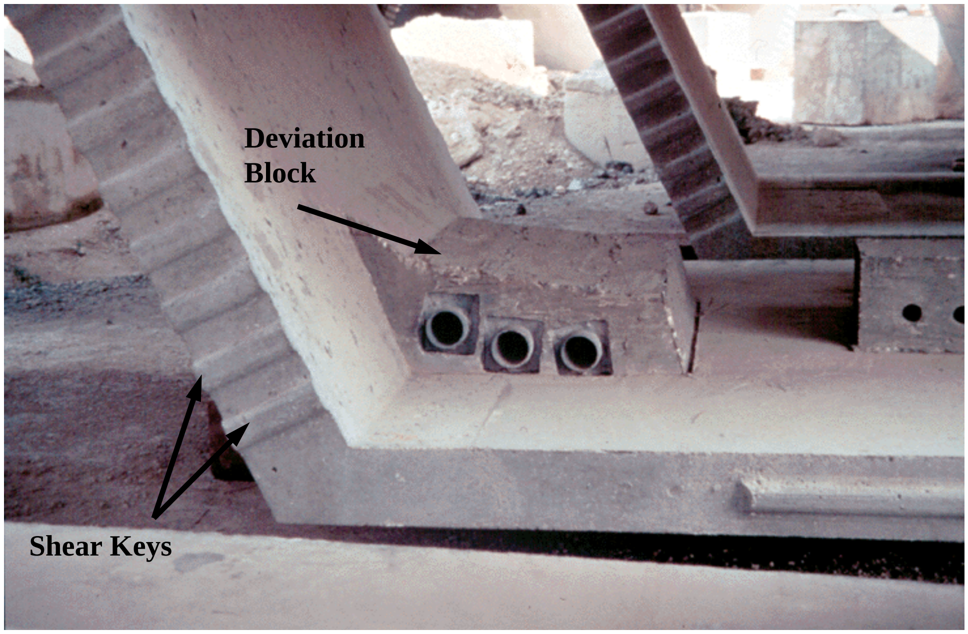

Figure 9.11.47 View of Box Girder Joint (Shear Keys) and Deviation Block

Internal Diaphragms

Internal diaphragms at the piers and abutments serve to stiffen the box section and to distribute the large bearing reaction loads. Tendon anchorages located within the diaphragm will also contribute to additional post-tensioning loads. This region of the structure is very highly stressed and, therefore, prone to crack development. The internal diaphragms require close examination during inspection (see Figure 9.11.48).

Figure 9.11.48 Box Girder Interior Diaphragm and Post-Tensioning Ducts

Areas Exposed to Drainage

Inspection of areas exposed to drainage is the same as those for concrete box girder bridges.

Closely examine the joints between the segments for any signs of leakage or infiltration.

Drain Holes

Inspection of drain holes is the same as those for concrete box girder bridges.

Areas Exposed to Traffic

Inspection of areas exposed to 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.

Other Areas Exposed to External Damage

- Waterborne debris or ice
- Fire
- Equipment hits
- Displacement of superstructure due to floods, storm surges, etc.

Post-Tensioned Grout Pockets

Inspection of post-tensioned grout pockets is the same as those for concrete box girder bridges.

Camber

Inspection for camber and vertical alignment is the same as those for concrete box girder bridges.

Miscellaneous Areas

Cracks caused by torsion and shear are the same as those for concrete box girder bridges.

Cracking within post-tensioning tendon areas (lines) are the same as for concrete box girder bridges.

Radial cracking is the same as for concrete box girder bridges.

Thermal Effects - The effects of temperature and the appropriate inspection methods to accommodate for it is the same as those for concrete box girder bridges. Additionally, these cracks can also occur at component changes in thickness such as that between a web and a flange. In this case the cracking will occur at the juncture between these two elements.

9.11.5 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 FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Items 58 and 59) for additional details about NBI component condition rating guidelines.

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

Element Level Condition State Assessment

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

NBE No.	**Description**
Decks/Slabs	
15	Prestressed/Reinforced Concrete Top Flange* * Note that this element designation is used regardless of the type of riding surface
Superstructure	
104	Prestressed Concrete Closed Web/Box Girder

BME No.	**Description**
Wearing Surfaces and Protection Systems	
520	Deck/Slab Protection Systems
521	Concrete Protective Coating

The unit quantity for the top flange (deck), protection systems and protective coating is square feet. The total area is distributed among the four available condition states depending on the extent and severity of the deficiency (see Figure 9.11.49). The unit quantity for the box girder is feet, and the total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the AASHTO *Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of concrete box girder superstructures:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
362	Superstructure Traffic Impact (load capacity)

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

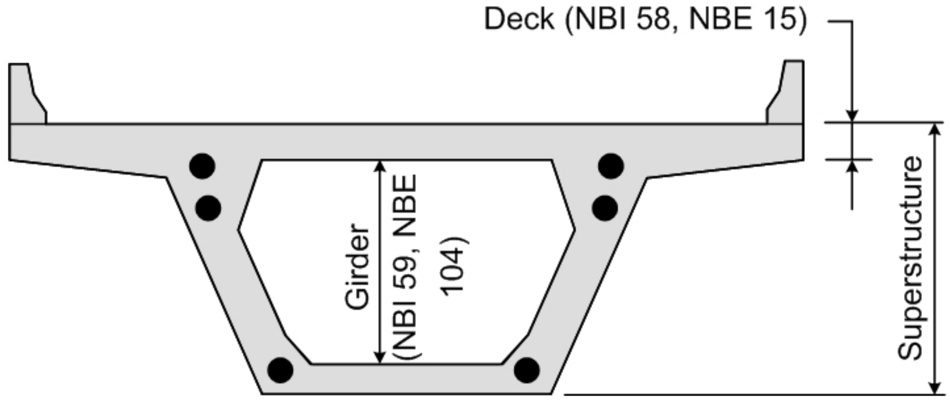

Figure 9.11.49 Components/Elements for Evaluation

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Chapter 10
Inspection and Evaluation of Steel Superstructures

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Chapter 10
Inspection and Evaluation of Steel Superstructures

Topic 10.1 Rolled Steel Multi-Beams and Fabricated Steel Multi-Girders

10.1.1 Introduction

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

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

Rolled beams are generally "compact" sections that satisfy ratios for the flange and web thicknesses to prevent buckling. Rolled beams come in a number of different sizes with each size having specific dimensions for the width and thickness for both the flange and web. These dimensions are standard and can be found in a number of publications, such as the Steel Construction Manual published by the American Institute of Steel Construction, Inc. Also, rolled beams may have bearing stiffeners but typically do not include intermediate stiffeners.

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

10.1.2 Design Characteristics

Rolled Multi-Beam

The steel rolled multi-beam bridge is a configuration of three or more parallel rolled beams with a deck placed on top of the beams. The most common use of this superstructure type is for simple spans, with span lengths from 30 to 50 feet (see Figure 10.1.1). Continuous span designs have also been used, some of which incorporate pin-and-hanger connections (see Figure 10.1.2). Rolled beams are manufactured in structural rolling mills from one piece of steel (i.e., the flanges and web are manufactured as an integral unit). Rolled beams in the past were generally available no deeper than 36 inches in depth but are now available from some mills as deep as 44 inches.

Figure 10.1.1 Simple Span Rolled Multi-Beam Bridge

Figure 10.1.2 Continuous Span Rolled Multi-Beam Bridge with Pin & Hanger

In the past, a common method of economically increasing the capacity of a rolled multi-beam bridge was to weld partial length cover plates to the flanges (see Figure 10.1.3). The cover plates increased a beam's bending strength. This practice also creates a problematic detail in the tension flange. The cover plates are attached by riveting or welding. Fatigue cracking has been found to occur in the beam flanges at the ends of partial length cover plates.

Figure 10.1.3 Rolled Multi-Beam Bridge with a Cover Plate

Fabricated Multi-Girder

The steel fabricated multi-girder bridge is similar to the rolled multi-beam bridge in appearance. However, fabricated girders are often larger than those that could be provided by the rolling mills. Older fabricated multi-girders are riveted or bolted built-up members consisting of angles and plates (see Figure 10.1.4). In a riveted or bolted built-up member, the angles are considered part of the flange. Today's fabricated multi-girders are usually welded plate members (see Figure 10.1.5).

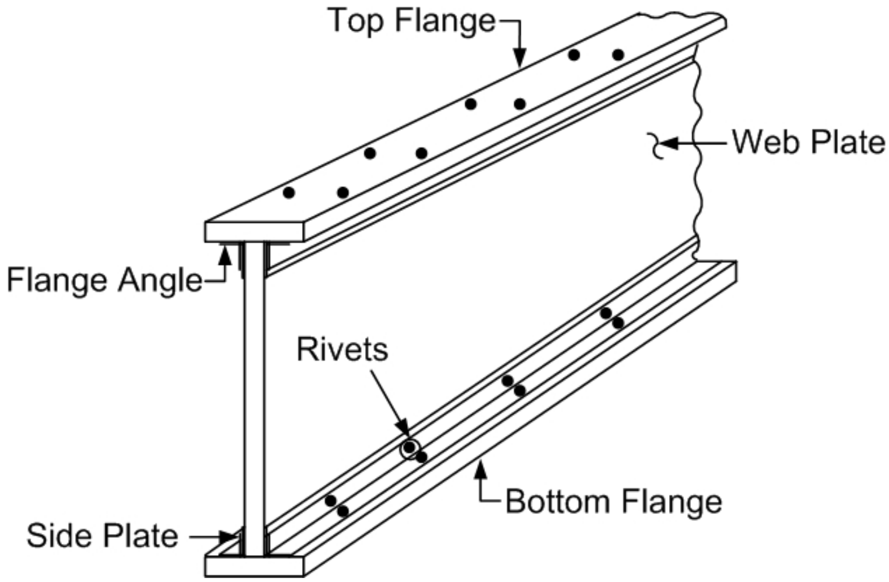

Figure 10.1.4 Built-up Riveted Plate Girder

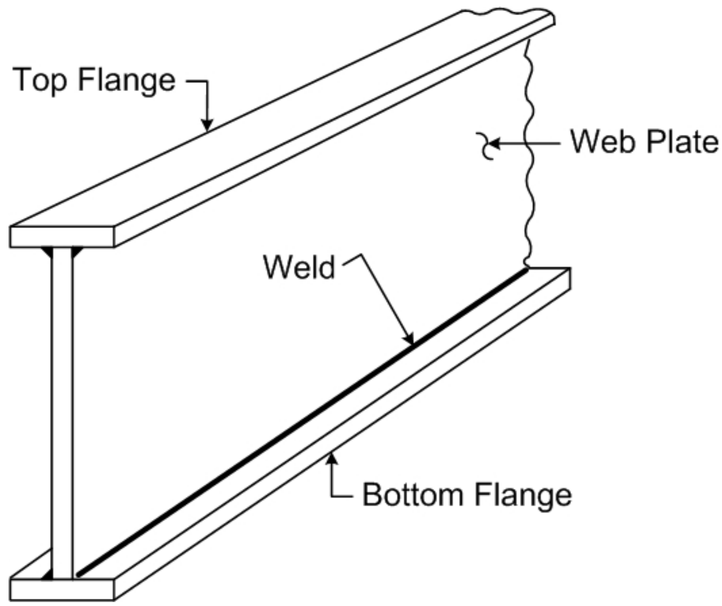

Figure 10.1.5 Welded Plate Girder

This bridge type can be found in single span (see Figure 10.1.6), multiple span, and continuous span designs (see Figure 10.1.7), and it is widely used when curved bridges are required (see Figure 10.1.8). Continuous welded multi-girders have been built for spans of over 500 feet. Pin-and-hanger connections are found in older multi-girder construction (see Figure 10.1.9). This type of connection is not used in modern bridge design practices due to excessive corrosion within the pin-and-hanger assembly which may lead to failure of the connection and girders. Pin-and-hanger assemblies are discussed in detail in Topic 10.7.

Figure 10.1.6 Single Span Fabricated Multi-girder Bridge

Figure 10.1.7 Continuous Span Fabricated Multi-girder Bridge

Figure 10.1.8 Curved Fabricated Multi-girder Bridge

Fabricated multi-girder bridges have three or more primary load paths (girders). Two-girder bridge systems are presented in Topic 10.2.

Figure 10.1.9 Fabricated Multi-girder Bridge with Pin & Hanger Connection

Sometimes, both types of superstructure, rolled steel beams and fabricated steel girders can be used on the same bridge (see Figure 10.1.10). The shorter approach spans are rolled beams while the longer main span utilizes fabricated girders.

Figure 10.1.10 Combination Rolled Beams and Fabricated Girders

Haunched Girder Design

In continuous girder designs, additional girder strength is required in negative moment regions. This is accomplished through a method called haunching. Haunching is the increasing of the web depth for a specified portion of the girder. The regions above intermediate supports (i.e. piers and bents) have negative moments larger than the adjacent positive moments. Typically, the girder depth used at the positive moment regions is not sufficient enough to resist these moments, so the web depth is increased. (See Topic 5.1, Bridge Mechanics) However, instead of increasing the depth for the full length of the girder, the girder is haunched at the intermediate supports.

Three methods have been used to haunch girders.

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

To haunch a rolled beam, the bottom flange is separated from the web and an insert plate of the required depth is welded in place (see Figure 10.1.11).

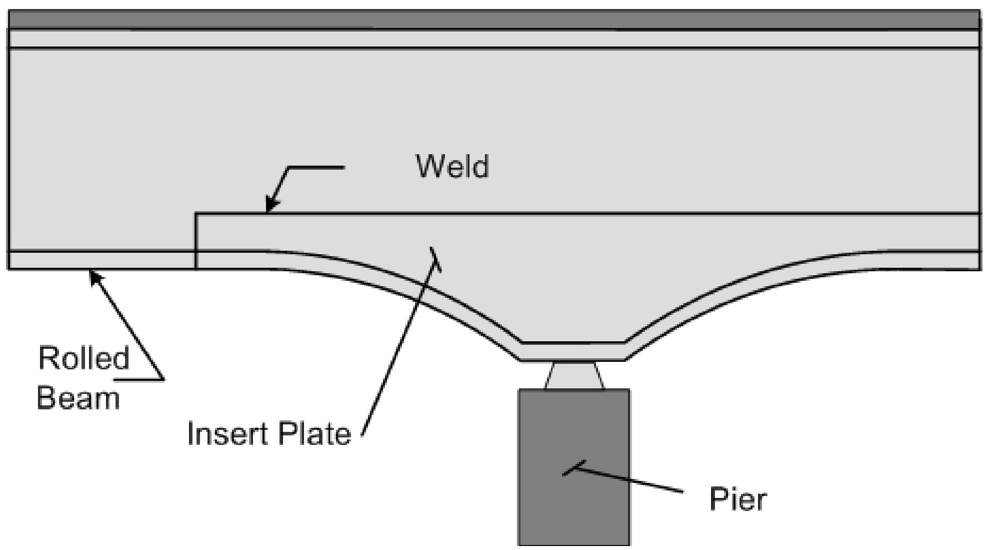

Figure 10.1.11 Web Insert Plate for Multi-beam

A fabricated variable depth girder is the method used today (see Figure 10.1.12). The web plate is simply fabricated to the required depth. The top and bottom flange plates are then welded to the web plate.

Figure 10.1.12 Fabricated Variable Depth Girder Bridge

Function of Stiffeners

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

Primary and Secondary Members

Primary members are designed to resist primary live loads from trucks and dead loads.

Secondary members do not resist primary live loads.

The primary members of a rolled multi-beam bridge are the rolled beams, and the secondary members are the diaphragms (see Figure 10.1.13). Intermediate and end diaphragms are provided to stabilize the beams during construction and to help distribute the live load more evenly to the rolled beams. Diaphragms may or may not be present on multi-beam bridges.

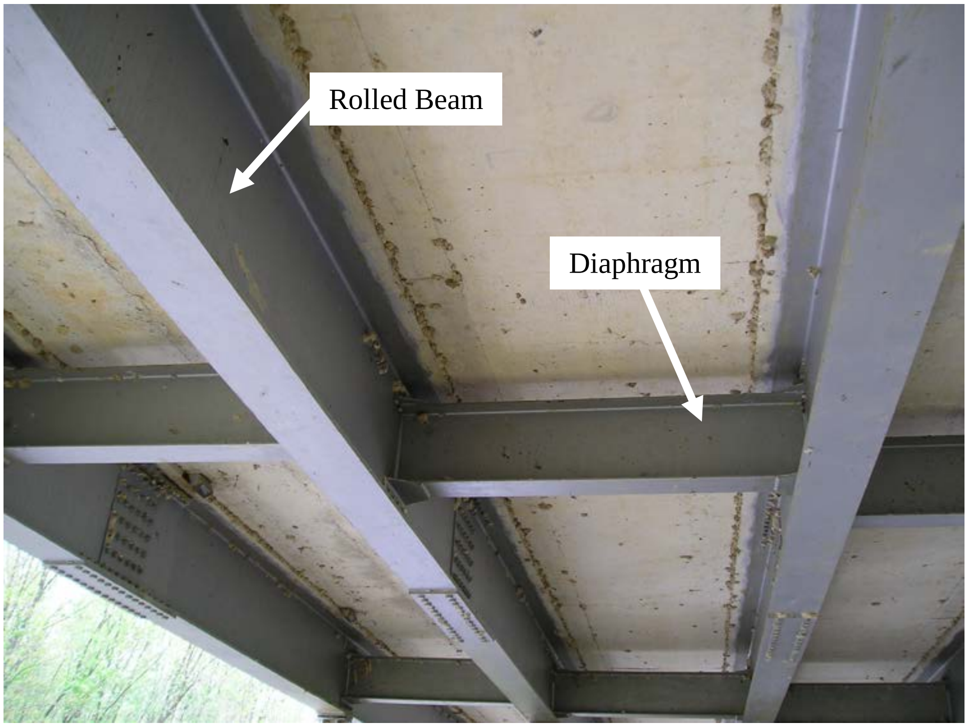

Figure 10.1.13 Rolled Beam (Primary Member) with Diaphragm (Secondary Member)

The primary members of a fabricated multi-girder bridge are the fabricated girders, as well as the diaphragms on a curved bridge. In the case of a curved structure, the diaphragms are designed to withstand the torsional loading attributed to curved structures and therefore, are also considered primary members (see Figure 10.1.14).

On straight multi-girder bridges, diaphragms are considered secondary members. Similar to rolled beam bridges, diaphragms are provided to stabilize the girders during construction and to help distribute secondary live load (see Figure 10.1.15). Diaphragms can be rolled shapes (e.g., I-beams and channels) or they can be cross-frames constructed from angles, tee shapes, and plates. They are usually attached to transverse web stiffeners which are normally referred to as connection plates. On older bridges, secondary members also include lateral bracing. Current design specifications discourage the use of lateral bracing. This is due to connections for lateral bracing being fatigue-prone.

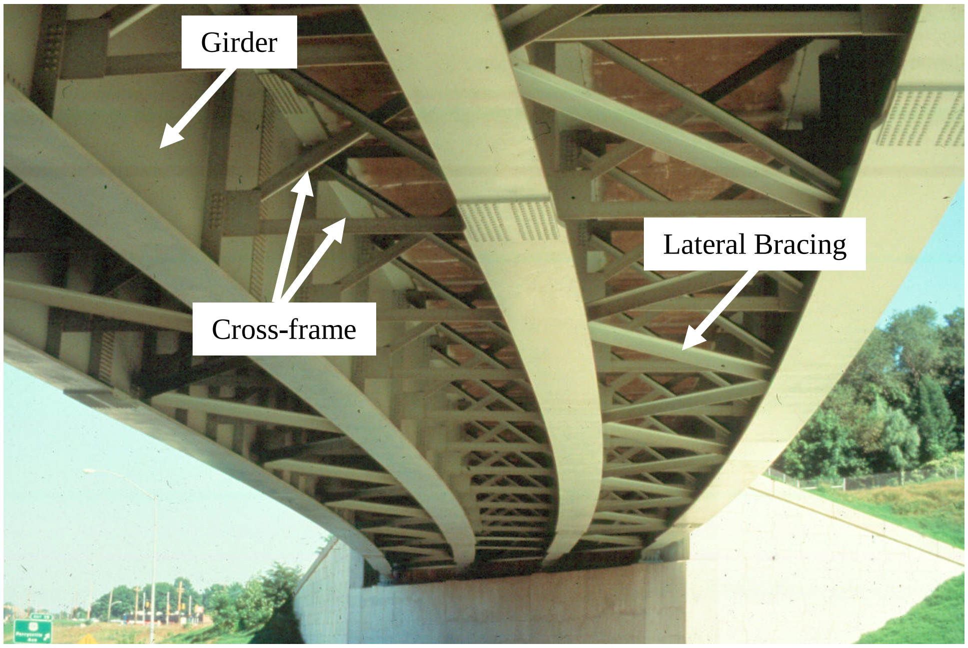

Figure 10.1.14 Curved Multi-Girder Bridge

Figure 10.1.15 Straight Multi-Girder Bridge

Fracture Critical Areas

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

10.1.3 Overview of Common Deficiencies

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

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

See Topic 6.3 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 6.4 for Fatigue and Fracture in Steel Bridges.

10.1.4 Inspection Methods and Locations

Inspection methods to determine steel deterioration are described in detail in Topic 6.3.7.

Methods

Visual

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

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

Physical

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

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

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

Advanced Inspection Methods

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

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

Other methods, described in Topic 15.3.3, include:

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

Locations

Bearing Areas

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

Examine the bearings at each support for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. See Topic 11.1 for a detailed description on the inspection of bridge bearings.

Shear Zones

Examine the web areas near the supports for any section loss or buckling (see Figure 10.1.16). Shear stresses are greatest near the supports. Therefore, the condition of the web is more critical near the supports than at mid-span. Also investigate the web for buckling due to overloads. If girders have been haunched by the use of insert plates, check the weld between the web and the insert plate.

Figure 10.1.16 Corroded Shear Zone on a Rolled Multi-beam Bridge

Flexure Zones

The flexure zone of each beam/girder includes the entire length between the supports (see Figures 10.1.17 and 10.1.18). Investigate the tension and compression flanges for corrosion, loss of section, cracks, dings, and gouges. Check the flanges in high stress areas for bending or flexure- related damage. Examine the compression flange for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. On continuous spans, the beam/girder over the intermediate supports has high flexural stresses due to negative moment (see Figures 10.1.19 and 10.1.20). If welded cover plates are present, check carefully at the ends of the cover plates for cracks due to fatigue.

Figure 10.1.17 Flexural Zone on a Multi-Span Simple Span Rolled Multi-Beam Bridge

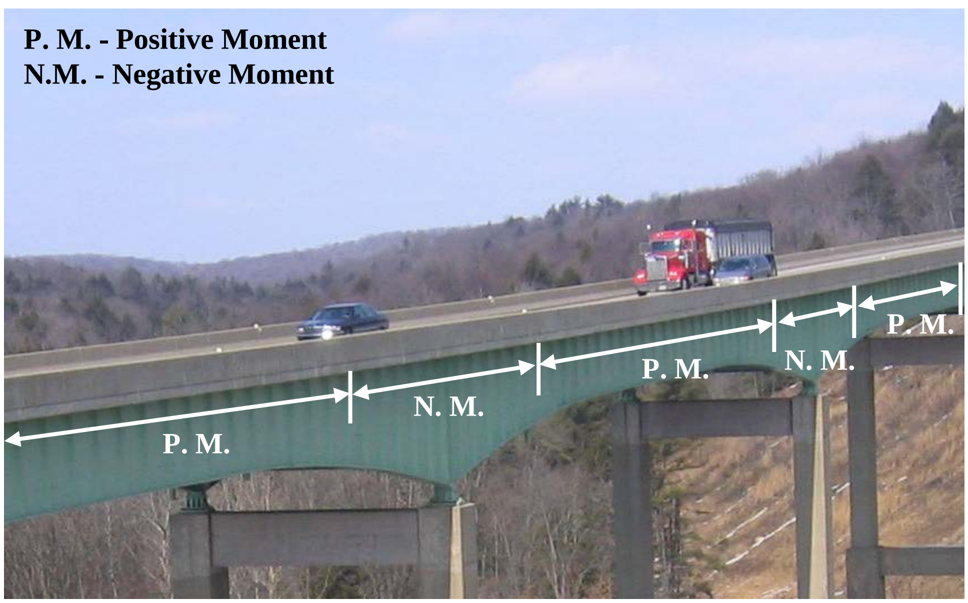

Figure 10.1.18 Flexural Zone on a Fabricated Continuous Span Multi-Girder Bridge

Figure 10.1.19 Negative Moment Region on a Continuous Span Rolled Multi-Beam Bridge

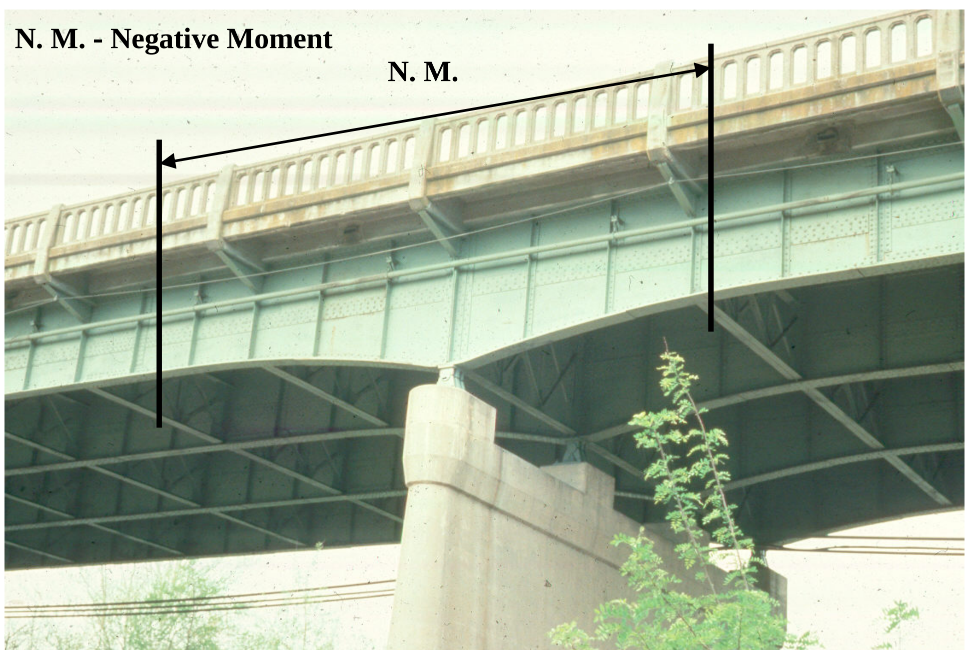

Figure 10.1.20 Negative Moment Region on a Continuous Span Fabricated Multi-Girder Bridge

Secondary Members

Examine the diaphragm and bracing connections for loose fasteners or cracked welds (see Figures 10.1.21 and 10.1.22). This problem is most common on skewed bridges, and it has also been observed on bridges with a high frequency of combination truck loads. Check horizontal connection plates, which can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Check for distorted members. Distorted secondary members may be an indication the primary members may be overstressed or the substructure may be experiencing differential settlement.

Figure 10.1.21 End Diaphragm

Figure 10.1.22 Intermediate Diaphragm

Areas Exposed to Drainage

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

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

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

Areas Exposed to Traffic

Check underneath the bridge for collision damage to the main girders and bracing if the bridge crosses over a highway, railway, or navigable channel. Document any cracks, section loss, or distortion found (see Figures 10.1.23 and 10.1.24).

Figure 10.1.23 Collision Damage on a Rolled Multi-Beam Bridge

Figure 10.1.24 Collision Damage on a Fabricated Multi-Girder Bridge

Problematic Details

Problematic details checked for deficiencies and deterioration include:

- Triaxial constraint
- Intersecting welds
- Cover plates
- Cantilevered suspended spans
- Insert plates
- Field welds: patch and splice plates
- Intermittent welds
- Out-of-plane bending
- Pin-and-hanger assemblies
- Back-up bars
- Mechanical fasteners and tack welds

Note that out-of-plane bending may occur at the following areas, which are presented further in Topic 6.4.8:

- Girder web connections for diaphragms and floorbeams, including connections plates at the top flange, connection plates at the bottom flange, and connection plates at the bottom flange for skewed bridges
- Staggered floorbeams or lateral gusset plate locations (for skewed bridges)
- Lateral bracing gussets and diaphragm connection plates

In addition to common problematic details, beams also utilize the following:

- Stiffeners (transverse and longitudinal)
- Groove welded butt splices
- Lateral bracing gusset plates
- Web-to-flange welds
- Misc. connections (railing and utilities)
- Flanges cut short
- Coped flanges
- Blocked flanges

See Topic 6.4.8 for additional information on problematic details.

Fracture Critical Members

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

10.1.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

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

NBE No.	Description
Superstructure	
107	Steel Girder/Beam
161	Pin, Pin-and-Hanger Assembly, or both

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

The unit quantity for the girder/beam is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for the pin-and-hanger assembly (if applicable) is each, with each pin-and-hanger element placed in one of the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coating is square feet, and the total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of steel multi-beam and multi-girder superstructures:

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

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

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Chapter 10

Inspection and Evaluation of Steel Superstructures

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Topic 10.2 Steel Two-Girder Systems

10.2.1 Introduction

The steel two-girder bridge, like the fabricated multi-girder bridge, can use either riveted or welded construction. The difference is that it has only two girders. Two-girder bridges can also have features similar to those of fabricated multi-girder bridges, such as web insert plates, transverse web stiffeners, and longitudinal web stiffeners (see Figure 10.2.1).

However, unlike the fabricated multi-girder bridge, the two-girder bridge has a floor system of smaller longitudinal stringers and transverse floorbeams. The floor system supports the deck while the girders support the floor system.

Two-girder bridges are typically straight bridges arranged in simple and/or continuous span configurations, but may be curved structures. Pin-and-hanger assemblies may be associated with two-girder bridges. Two-girder bridges are classified as either deck girder or through girder systems.

In a deck girder system, the deck is supported by the floor system and top flanges of the two girders (see Figure 10.2.1). In a through girder system, the deck is supported by the floor system between the two girders and the two girders extend above the deck. (see Figure 10.2.2).

Figure 10.2.1 General View of a Deck Girder Bridge

While few through girders are constructed today, they were commonly used prior to the early 1950's. Since many through girder bridges were constructed in the 1940’s and 1950’s, they are commonly riveted. Their most common use was where vertical underclearance was a concern, such as over railroads (see Figure 10.2.3).

Figure 10.2.2 Through Girder Bridge

Figure 10.2.3 Through Girder Bridge with Limited Underclearance

A rare type of through girder has three or more girders, with the main girders actually separating the traffic lanes (see Figure 10.2.4). These structures are most likely converted railroad or trolley bridges.

Figure 10.2.4 Through Girder Bridge with Three Girders

10.2.2 Design Characteristics

Floor System Arrangement

Floor systems are similar in deck girder and through girder systems.

The floor system supports the deck. There are two types of floor systems found on two-girder bridges:

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

The girder-floorbeam (GF) system consists of floorbeams connected to the main girders. The floorbeams are considerably smaller than the girders and are perpendicular to traffic. The deck is supported by the floorbeams, which in turn transmit the loads to the main girders. The floorbeams can be either rolled beams, fabricated girders, or fabricated cross frames (see Figure 10.2.5).

Figure 10.2.5 Two-Girder Bridge with Girder-Floorbeam System

The girder-floorbeam-stringer (GFS) system consists of floorbeams connected to the main girders, and longitudinal stringers, parallel to the main girders, connected to the floorbeams (see Figure 10.2.6). The stringers may either connect to the web of the floorbeams or be stacked on top of the floorbeams, in which case they may be continuous or simply supported stringers. Stringers are usually rolled beams and are considerably smaller than the floorbeams. It is also possible to find floorbeams that are stacked on top of the main girders, and the floorbeams may extend or overhang from the girders (see Figure 10.2.7). This stack configuration reduces problematic details that may lead to fatigue cracking.

Figure 10.2.6 Two-Girder Bridge with Girder-Floorbeam-Stringer System

Figure 10.2.7 Two-Girder Bridge GFS System with Stacked Floorbeam and Stringers

Primary and Secondary Members

The primary members of a two-girder bridge are the girders, floorbeams, and stringers, if present. The secondary members are diaphragms and the lateral bracing members, if present. These secondary members usually consist of channels, angles or tee shapes placed diagonally in horizontal planes between the two main girders. The lateral bracing is generally in the plane of the bottom flange. Lateral bracing serves to minimize any differential longitudinal movement between the two girders (see Figure 10.2.8). Not all two-girder bridges will have a lateral bracing system. Diaphragms, if present, are usually placed between stringers.

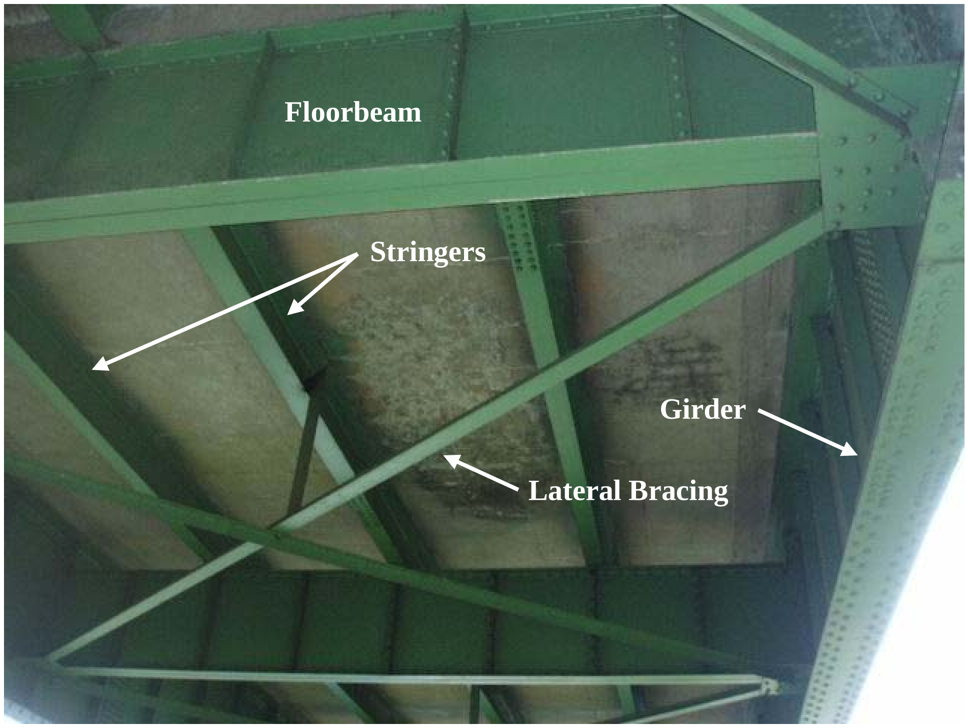

Figure 10.2.8 Underside View of Deck Girder Bridge with Lateral Bracing System

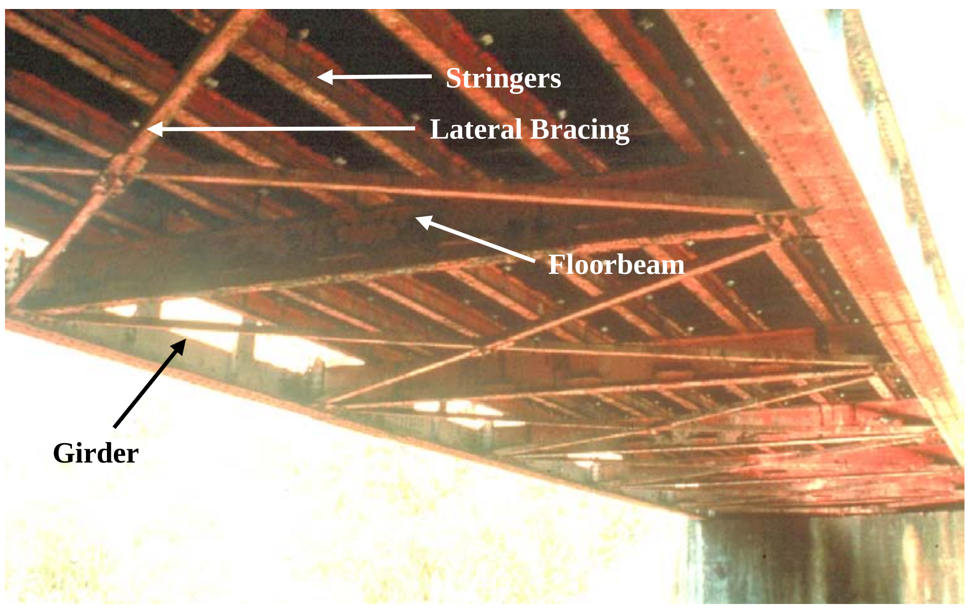

Figure 10.2.9 Underside View of Through Girder Bridge with Lateral Bracing

Fracture Critical Areas

Girders

Two-girder bridges (deck girder and through girder) do not have load path redundancy. Since the girders are constructed of steel, are in tension, and would result in a partial or total bridge collapse if the member fails, they are considered fracture critical. Both deck and through girder bridges are therefore classified as fracture critical bridge types.

Pin-and-hanger assemblies in two-girder bridges are fracture critical members (see Figure 10.2.10). Failure of one pin or one hanger will cause collapse of the suspended span since there is no alternate load path (e.g., Mianus River Bridge). Pins are considered "frozen" when corrosion restricts rotation. The pins and hangers experience additional bearing, torsion, bending and shear stresses when the pin-and-hanger assembly is frozen. This is a critical situation when it occurs on a (load path) nonredundant two-girder bridge.

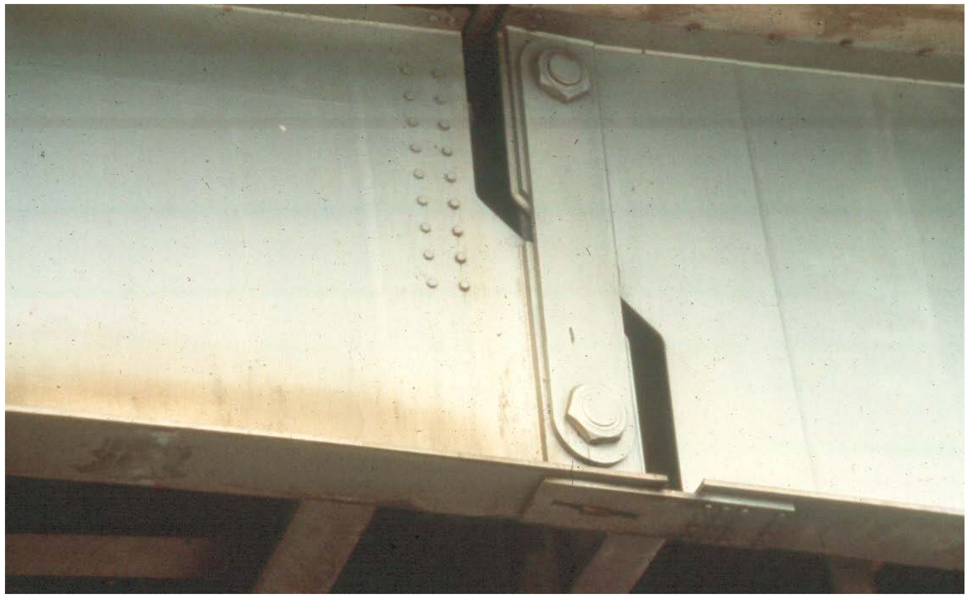

Figure 10.2.10 Two-Girder Bridge with Pin-and-Hanger Assembly

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

Floorbeams

A floorbeam may be fracture critical if it satisfies one or more of the following conditions:

- Flexible or hinged connection to support at the girder/floorbeam connection
- Floorbeam spacing greater than 14'-0"
- No stringers supporting the deck
- Stringers are configured as simple beams

Some states consider any floorbeam to be fracture critical even if none of the above conditions are met.

A three-dimensional finite element structural analysis to determine fracture criticality may be performed to determine the exact consequences to the bridge if a floorbeam or floorbeam connection fails.

10.2.3 Overview of Common Deficiencies

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

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

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

10.2.4 Inspection Methods and Locations

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

Methods

Visual

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

Most deficiencies in steel bridges are first detected by visual inspection. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being inspected, is typically required. More exact visual observations can also be employed using a magnifying unit after cleaning the suspect area. All fracture critical members are required to be inspected within arm's length during each inspection.

Physical

Removal of paint can be done using a wire brush, grinding, or sand blasting, depending on the size of the suspected deficiency. Exercise care when cleaning suspected areas that are cracked. When cleaning steel surfaces, avoid any type of cleaning process that would tend to close discontinuities, such as blasting. The use of degreasing spray before and after removal of the paint may help in revealing the deficiency.

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, examine all other similar locations and details on the bridge.

Advanced Inspection Methods

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

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

Other methods, described in Topic 15.3.3, include:

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

Locations

Bearing Areas

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

Examine the bearings at each support for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. See Topic 11.1 for a detailed presentation on the inspection of bridge bearings.

Shear Zones

Examine the web areas of the girders, floorbeams, and stringers near their supports for section loss or buckling (see Figures 10.2.11 and 10.2.12). This is a critical area, especially if the web is coped or the flange is blocked.

Figure 10.2.11 Shear Zone on a Deck Girder Bridge

Figure 10.2.12 Web Area Near Support on a Through Girder Bridge

Flexure Zones

The flexure zone of each girder includes the entire length between the supports (see Figures 10.2.13 and 10.2.15). Investigate the tension and compression flanges for corrosion, loss of section, cracks, dings, and gouges. Check the flanges in high stress areas for bending or flexure- related damage. Examine the compression flange for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. On continuous spans, the beams over the intermediate supports have high flexural stresses due to negative moment. Check flange splice welds and longitudinal stiffener splice welds in tension areas (see Figure 10.2.14).

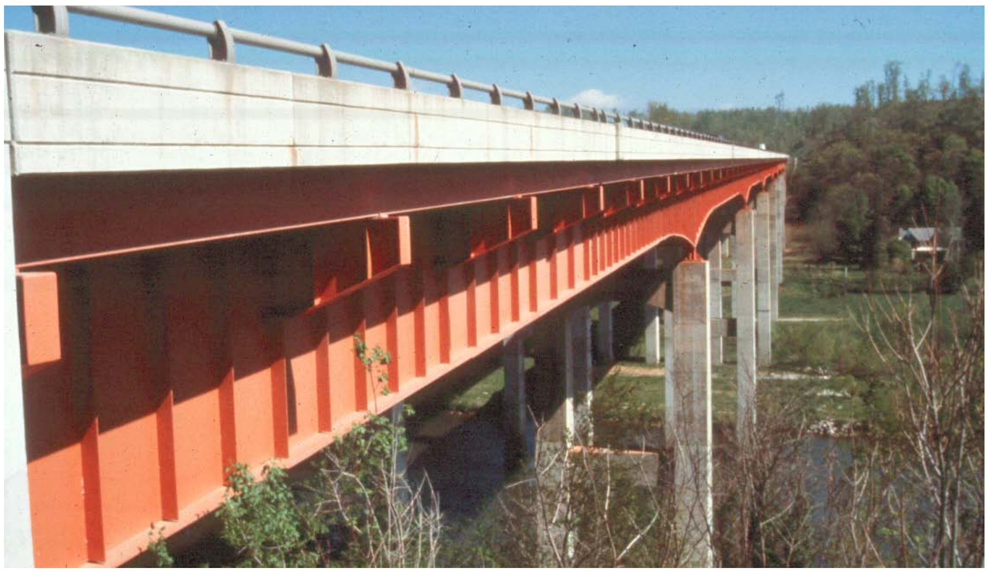

Figure 10.2.13 Flexural Zone on a Two-Girder Bridge

Figure 10.2.14 Longitudinal Stiffener in Tension Zone on a Two-Girder Bridge

Figure 10.2.15 Flexural Zone on a Through Girder Bridge

Secondary Members

Examine the diaphragms, if present, and the connection areas of the lateral bracing for cracked welds, fatigue cracks, and loose fasteners. Inspect the bracing members for any distortion or corrosion (see Figures 10.2.16 and 10.2.17). Distorted or cracked secondary members may be an indication the primary members are overstressed or the substructure may be experiencing differential settlement.

Figure 10.2.16 Lateral Bracing Connection on a Deck Girder Bridge

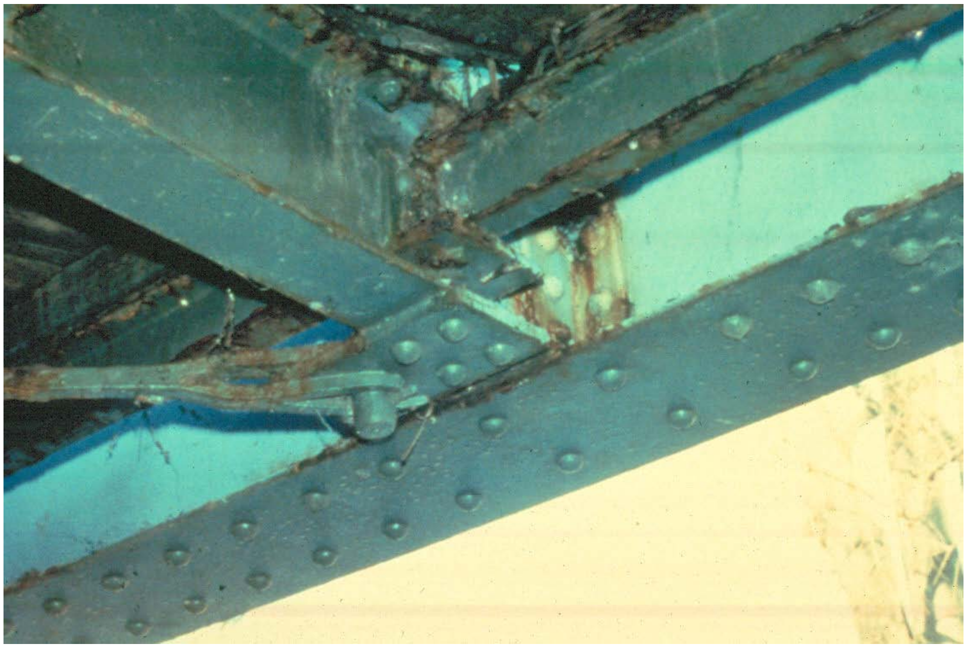

Figure 10.2.17 Lateral Bracing Connection on a Through Girder Bridge

Areas Exposed to Drainage

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

On two-girder bridges check:

- Along the bottom flanges of the girders
- Pockets created by girder-floorbeams and floorbeam-stringer connections
- Lateral bracing gusset plates
- Areas exposed to drainage runoff
- Along the girder webs at the curb line (through girder system)

Areas Exposed to Traffic

Check underneath the bridge for collision damage to the main girders and bracing if the bridge crosses over a highway, railway, or navigable channel. Document any cracks, section loss, or distortion found (see Figures 10.2.18 and 10.2.19). On a through girder bridge, investigate the main girders along the curb lines and at the ends for collision damage.

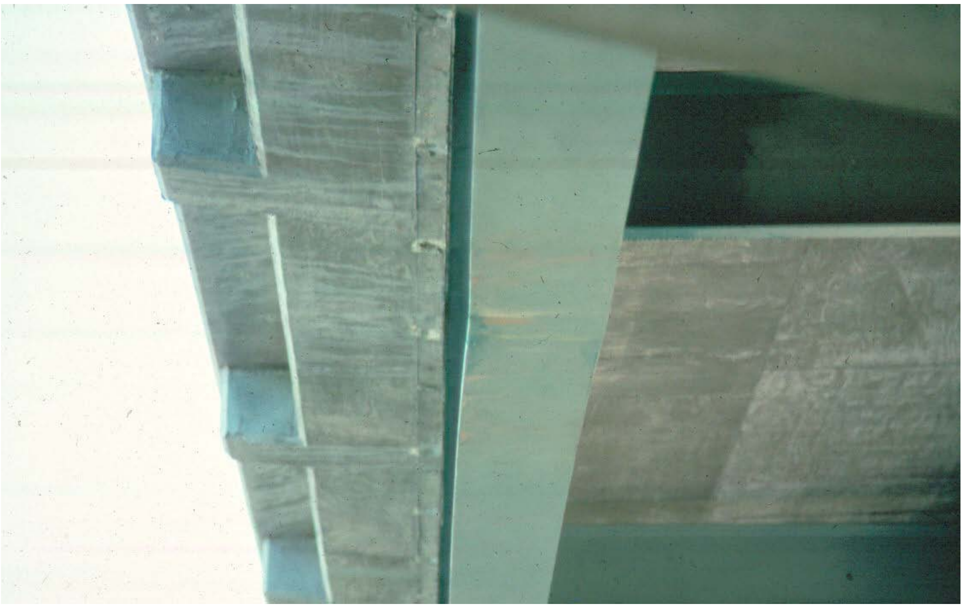

Figure 10.2.18 Collision Damage to a Deck Girder Bridge

Figure 10.2.19 Collision Damage to a Through Girder Bridge

Problematic Details

Problematic details checked for deficiencies and deterioration include:

- Triaxial constraint
- Intersecting welds
- Cover plates
- Cantilevered suspended spans
- Insert plates
- Field welds: patch and splice plates
- Intermittent welds
- Out-of-plane bending
- Pin-and-hanger assemblies
- Back-up bars
- Mechanical fasteners and tack welds

Note that out-of-plane bending may occur at the following areas, which are presented further in Topic 6.4.8:

- Girder web connections for diaphragms and floorbeams, including connections plates at the top flange, connection plates at the bottom flange, and connection plates at the bottom flange for skewed bridges
- Lateral bracing gussets and diaphragm connection plates
- Diaphragm connections to gusset plates
- Cantilevered floorbeams

In addition to the common problematic details, girders also utilize the following:

- Stiffeners (transverse/longitudinal)
- Groove welded butt splices
- Lateral bracing gusset plates
- Web-to-flange welds
- Miscellaneous connections (railing/utilities)

See Topic 6.4.8 for additional information on problematic details.

Fracture Critical Members

Since two-girder bridges have no load path redundancy and are fracture critical, it is important to inspect the main girders thoroughly. Floorbeams may also be fracture critical if they meet the requirements specified in Topic 10.2.2. Bridge specific written inspection procedures should be developed for each bridge with fracture critical members. Document and measure any defects such as cracks, section loss and out-of plane distortions. Review bridge specific inspection procedures and all previous reports before performing the inspection to note any areas of particular concern. Check all reported deficiencies to ensure no further development has occurred.

10.2.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

In an element level condition state assessment of a steel two-girder system, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Superstructure	
107	Steel Girder/Beam
113	Steel Stringer
152	Steel Floorbeam
161	Pin, Pin-and-Hanger Assembly, or both

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

The unit quantity for the girder, stringer (if applicable), and floorbeam is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for the pin-and-hanger assembly (if applicable) is each, with each pin-and-hanger element placed in one of the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coating is square feet, and the total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions. For pin-and-hanger assemblies, see Topic 10.7.

The following Defect Flags are applicable in the evaluation of the steel two-girder systems:

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

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

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Inspection and Evaluation of Steel Superstructures

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Topic 10.3 Steel Box Beams and Girders

10.3.1 Introduction

A box girder bridge is supported by one or more steel box girder members. Steel box members are typically connected by welding. The rectangular or trapezoidal cross section of the box girder consists of two or more web plates connected to a single bottom flange plate. Although concrete box beams and concrete box girders have distinguishable differences including cross-section dimensions and shape, the terms "box beam" and "box girder" may be used interchangeably when discussing steel superstructures.

Box girder bridges are used in simple spans of 75 feet or more (see Figure 10.3.1) and in continuous spans of 100 feet or more. They are frequently used for curved bridges due to their high degree of torsional rigidity (see Figure 10.3.2).

Figure 10.3.1 Simple Span Box Girder Bridge

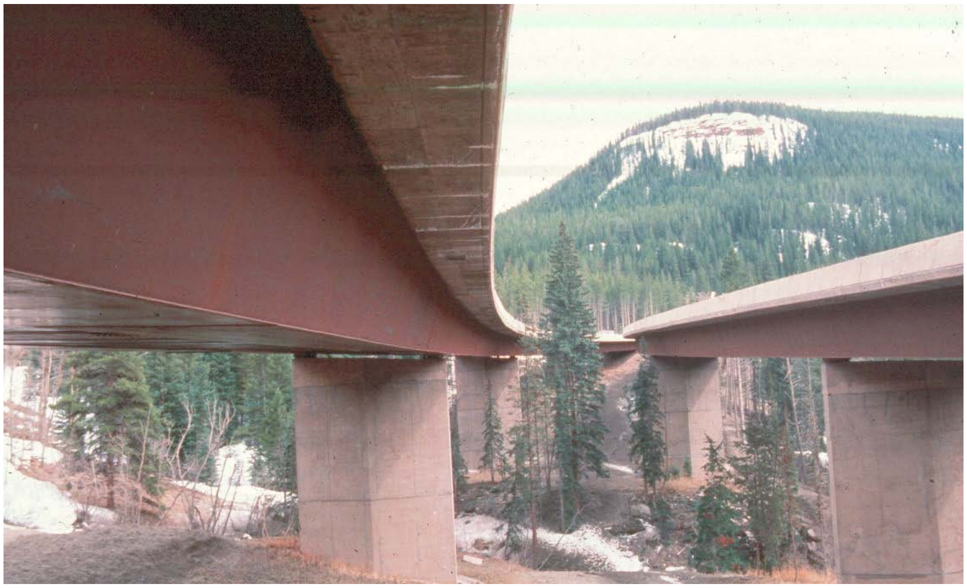


Figure 10.3.2 Curved Box Girder Bridge

10.3.2
Design Characteristics

Configuration

A box girder bridge can use a single box configuration (see Figure 10.3.3) or have multiple (spread) boxes in its cross section (see Figure 10.3.4). Several factors such as deck width, span length, terrain and even aesthetics can all play a role in determining which configuration is used.

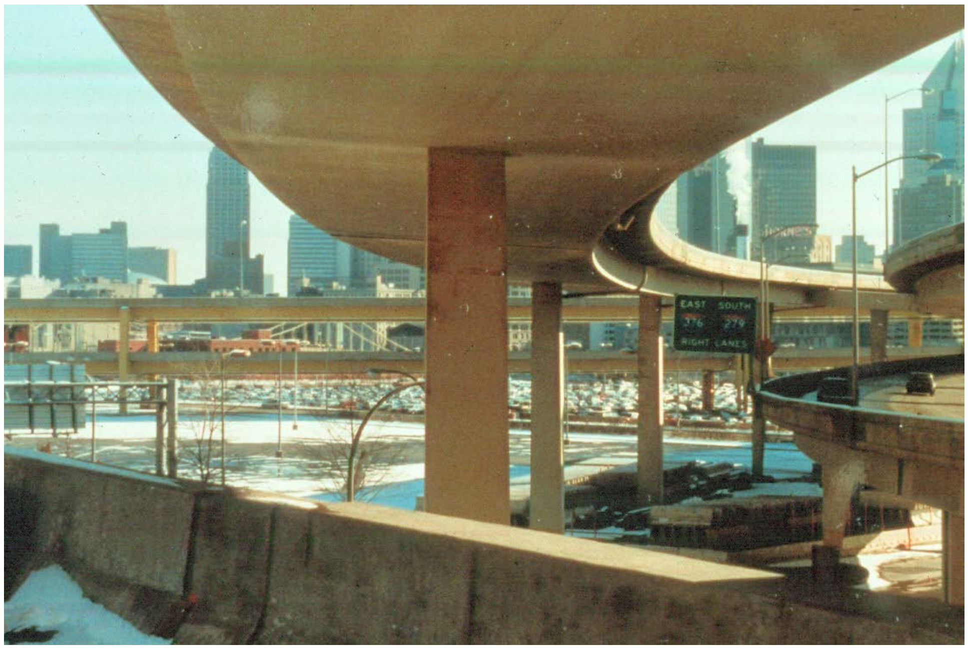


Figure 10.3.3 Box Girders With Multiple Interior Webs

Figure 10.3.4 Spread Box Girders

Primary and Secondary Members

The primary members of a box girder bridge are the box girders (including stiffeners and internal diaphragms) and, on a curved bridge, the external diaphragms. On a straight bridge, the external diaphragms are secondary members. Diaphragms can be solid plates, rolled shapes (e.g., I-beams and channels), or cross frames constructed with angles, tee shapes, channels and plates (see Figure 10.3.5).

Figure 10.3.5 Diaphragms – K Bracing Internal Transverse Stiffeners

Function of an Internal Stiffener

The webs and bottom flange of large box shapes are stiffened in areas of compressive stress. This is accomplished in part by stiffeners located inside the box member. The stiffeners are designed to help the box girder resist buckling due to torsional and shear forces. The stiffeners limit the unsupported length of the web and bottom flange, which result in increased stability of the box girder. Box girders may also incorporate both diaphragm and top flange lateral bracing systems. External diaphragms may be used between box girders (see Figure 10.3.6). Box girders typically have an opening or access door to allow the bridge inspector to examine the inside of the box (see Figure 10.3.7). Box girders may be considered to be confined spaces.

Figure 10.3.6 External Diaphragm

Figure 10.3.7 Box Girder Access Door

Fracture Critical Areas

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

Deck Interaction

The top flange may consist of individual plates welded to the top of each web plate. If the top flange plates incorporate shear connectors, the superstructure is composite with the concrete deck. A composite deck is one in which the deck and the superstructure work together to carry the live load (see Figure 10.3.8). Alternatively, the top flange may consist of a single plate extending the width of the box. This configuration is classified as an orthotropic steel plate deck (see Figure 10.3.9). A wearing surface is then placed on the top flange as the riding surface.

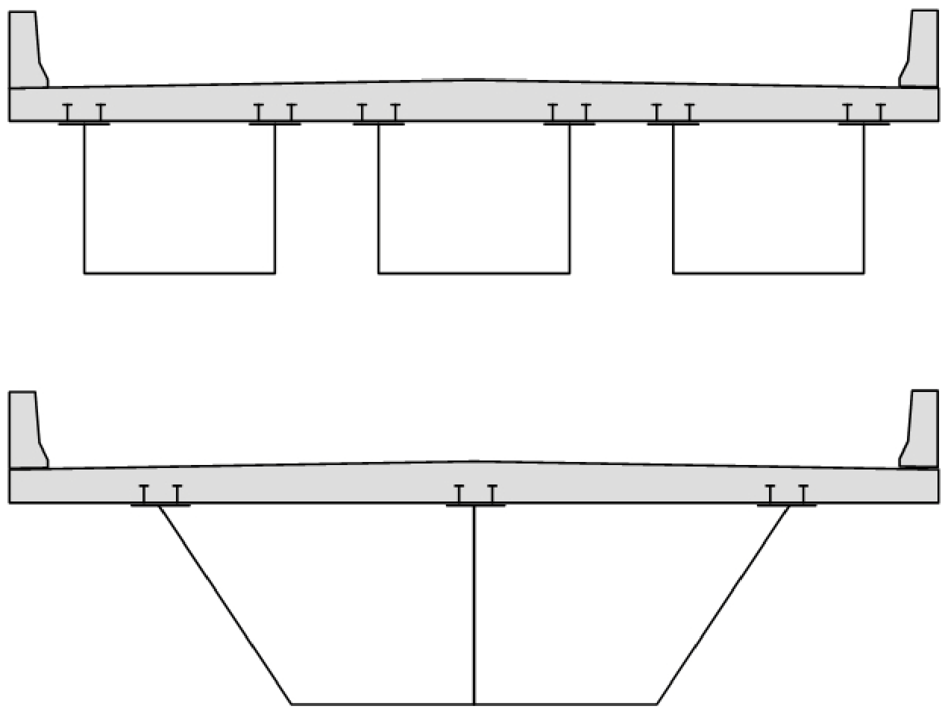


Figure 10.3.8 Box Girder Cross Section with Composite Deck

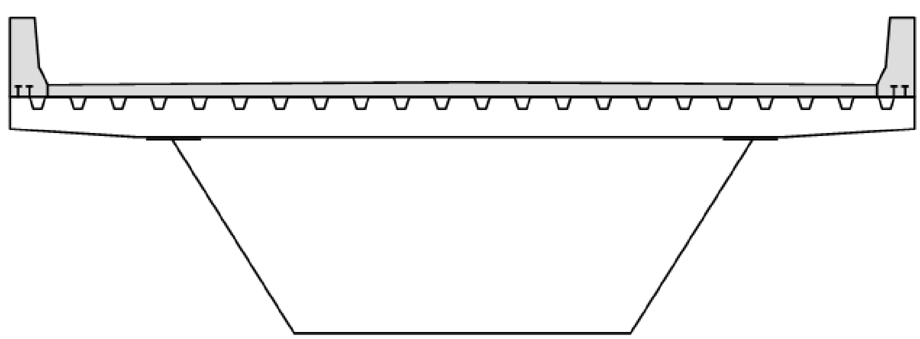


Figure 10.3.9 Box Girder Cross Section (at Floorbeam) with Orthotropic Steel Plate Deck

10.3.3 Overview of Common Deficiencies

Common deficiencies that occur on steel box girder bridges are:

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

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

10.3.4 Inspection Methods and Locations

Inspect box girders on both the interior and the exterior. When examining the interior, exercise caution at all times. Major concerns involved with inspecting a confined space include lack of sufficient oxygen, the presence of toxic or explosive gases, unusual temperatures and poor ventilation. Also, the distance between access hatches frequently exceeds the limit that rescue crews can reach in the event of an emergency (refer to Topic 2.2, Safety Fundamentals for Bridge Inspectors, for a more detailed description of these and other safety concerns).

Inspection methods to determine other causes of steel deterioration are detailed in Topic 6.3.8.

Methods

Visual

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

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

Physical

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

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, examine all other similar locations and details on the bridge.

Advanced Inspection Methods

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

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

Other methods, described in Topic 15.3.3, include:

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

Locations

Bearing Areas

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

Examine the bearings at each support for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. See Topic 11.1 for a detailed presentation on the inspection of bridge bearings.

Shear Zones

Examine the web areas near substructure supports for cracks, section loss and buckling (see Figure 10.3.10). Be sure to include intermediate supports provided by piers (see Figure 10.3.11).

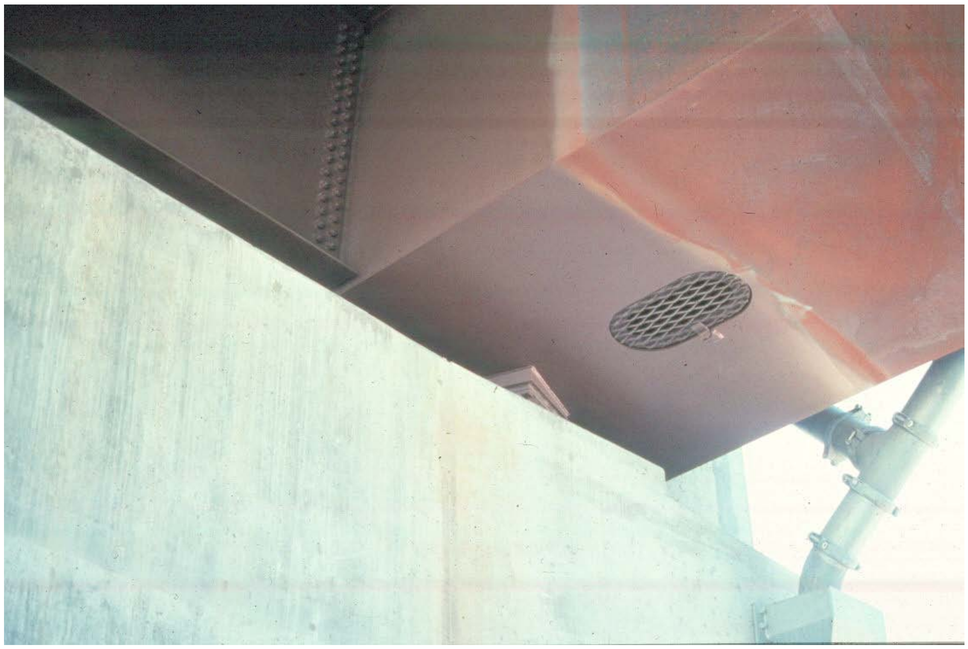


Figure 10.3.10 Box Girder Shear Zone

Figure 10.3.11 Continuous Box Girders

Flexure Zones

The flexure zone of each box girder includes the entire length between the supports (see Figure 10.3.11). Investigate the tension and compression flanges for corrosion, loss of section, cracks, dings, and gouges. Check the flanges in high stress areas for bending or flexure-related damage. Examine the compression flange for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. On continuous spans, the box girders over the intermediate supports have high flexural stresses due to negative moment. If welded cover plates are present, check carefully at the ends of the cover plates for cracks.

Secondary Members

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

Areas Exposed to Drainage

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

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

Areas Exposed to Traffic

For box girders over a highway, railway, or navigable channel, check the box girder for signs of collision damage. Document any loss of section, cracking, scrape marks or distortion.

Problematic Details

Problematic details checked for deficiencies and deterioration include:

- Triaxial constraint
- Intersecting welds
- Cover plates
- Cantilevered suspended spans
- Insert plates
- Field welds: patch and splice plates
- Intermittent welds

- Out-of-plane bending
- Pin-and-hanger assemblies
- Back-up bars
- Mechanical fasteners and tack welds

Note that out-of-plane bending may occur at the following areas, which are presented further in Topic 6.4.8:

- Girder web connections for diaphragms and floorbeams, including connections plates at the top flange, connection plates at the bottom flange, and connection plates at the bottom flange for skewed bridges
- Lateral bracing gussets and diaphragm connection plates
- Diaphragm connections to gusset plates

In addition to the common problematic details, box girders also utilize the following:

- Stiffeners (transverse/longitudinal)
- Groove welded butt splices
- Lateral bracing gusset plates
- Web-to-flange welds
- Miscellaneous connections (railing/utilities)

See Topic 6.4.8 for additional information on problematic details.

Fracture Critical Members

The redundant nature of a box girder bridge depends primarily on the number of box girders in the span. If two or less box girders are used, the structure is considered nonredundant and the box girders are fracture critical members (see Figure 10.3.12). Bridge specific written inspection procedures should be developed for each bridge with fracture critical members. Review bridge specific inspection procedures and all previous reports before performing the inspection to note any areas of particular concern. Check all reported deficiencies to ensure no further development has occurred.

If three or more box girders are used, the structure is generally considered redundant (see Figure 10.3.13). However, if the spacing of the box girders is large, the structure may not be redundant.

Figure 10.3.12 Non-Redundant Box Girder Bridges

Figure 10.3.13 Redundant Box Girder Bridge

10.3.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

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

NBE No.	**Description**
Superstructure	
102	Steel Closed Web/Box Girder

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

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

The following Smart Flags are applicable in the evaluation of steel box girder superstructures:

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

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

Table of Contents

Chapter 10
Inspection and Evaluation of Steel Superstructures

Topic 10.4 Steel Trusses

10.4.1 Introduction

Metal truss bridges have been built since the early 1800's. They can be thought of as a deep girder with the web cut out. They are also the only bridge structure made up of triangles (see Figure 10.4.1). The original metal trusses were made of wrought iron, then cast iron, then steel. When trusses were first being built of metal, material costs were very high and labor costs were low. Because trusses were made up of many short pieces, it was cost effective to build the members in the shop and assemble them at the site. Today the higher costs of labor and the lower costs of material have limited the use of trusses to major river crossings.

10.4.2 Design Characteristics

The superstructure of a truss bridge usually consists of two parallel trusses (see Figure 10.4.2). The trusses are the main load-carrying members on the bridge. There are three types of trusses, grouped according to their position relative to the bridge deck (see Figure 10.4.2).

Figure 10.4.1 Simple Span Truss

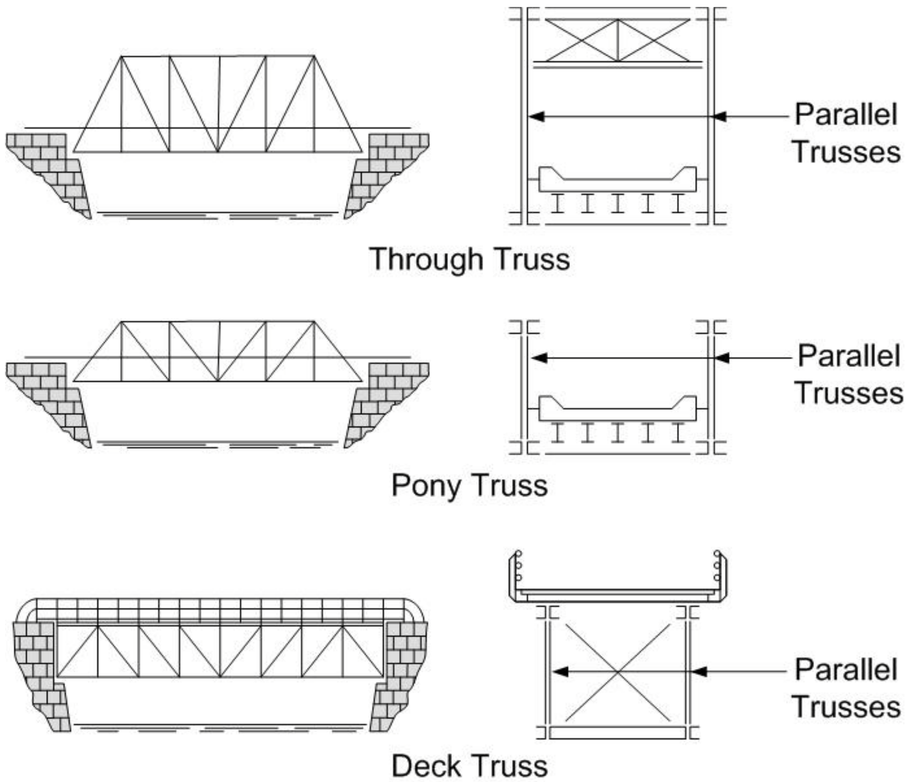


Figure 10.4.2 Through-Pony-Deck Truss Comparisons

Through Truss

On a through truss, the roadway is placed between the parallel trusses. (see Figure 10.4.3). Through trusses are constructed when underclearance is limited.

Figure 10.4.3 Through Truss

Pony Truss

A pony or "half-through" truss has no overhead bracing members connecting the two trusses (see Figure 10.4.4). The vertical height of the pony truss is much less than the height of a through truss. Today, pony trusses are seldom built, having been replaced by the multi-beam bridge.

Figure 10.4.4 Pony Truss

Deck Truss

On a deck truss, the roadway is placed on top of the parallel trusses (see Figure 10.4.5). Deck trusses have unlimited horizontal clearances and can readily be widened. For these reasons, they are preferred over through trusses when under-clearance is not a concern.

Figure 10.4.5 Deck Truss

Other Truss Applications

Trusses are generally considered to be main members. However, they are also used as floor systems in arches and as stiffening trusses in suspension bridges and arch bridges (see Figures 10.4.6 and 10.4.7). Trusses are also commonly used for movable bridge spans because they are lightweight and have higher overall stiffness (see Figure 10.4.8). Even towers are sometimes braced with web members, as a truss.

Figure 10.4.6 Suspension Bridge with Stiffening Truss

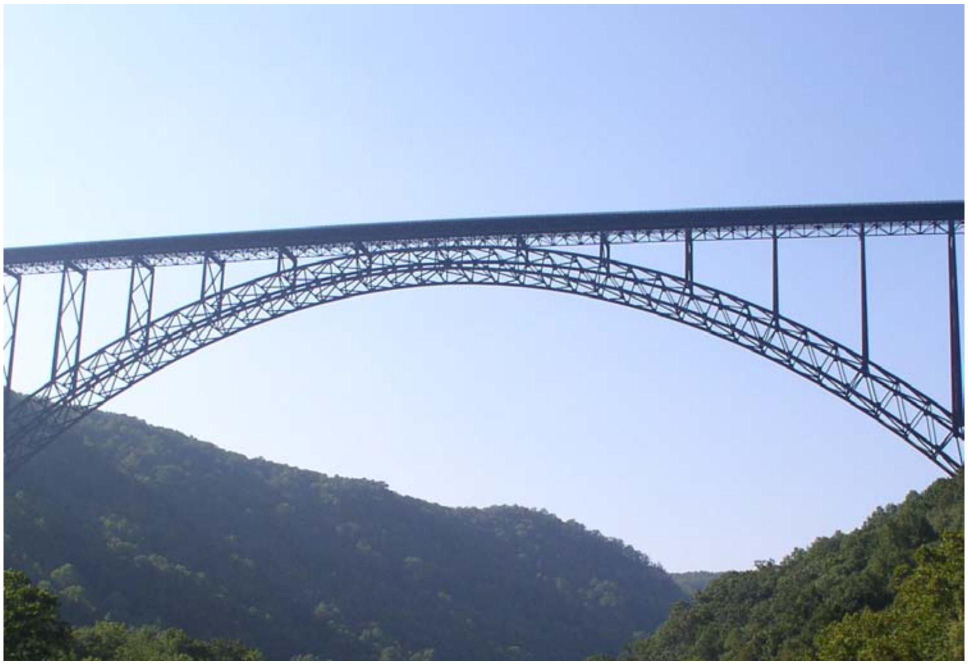


Figure 10.4.7 Deck Arch Bridge with Stiffening Truss

Figure 10.4.8 Vertical Lift Bridge

Design Geometry

Bridge engineers have used a variety of arrangements in the design of trusses. Many of the designs were patented by and named after their inventor. One characteristic that bridge trusses have in common is that the arrangement of the truss members forms triangles (see Figure 10.4.9).

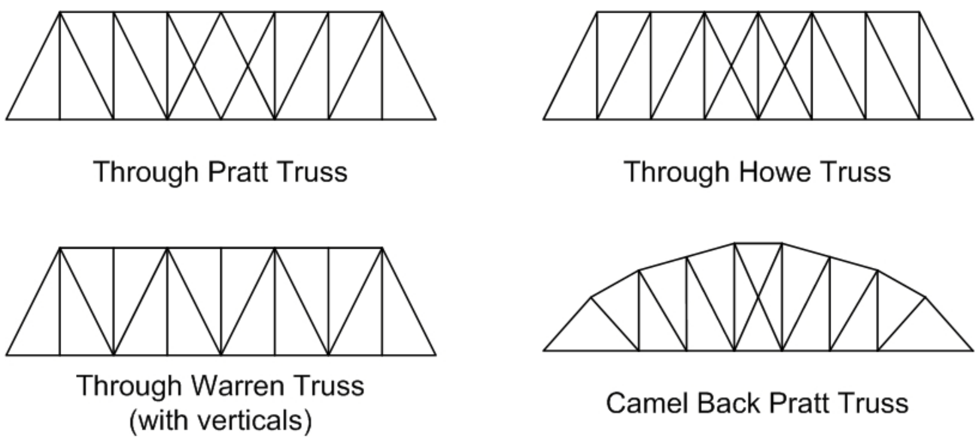


Figure 10.4.9 Various Truss Designs

Trusses have been constructed for short to very long spans, using simple, multiple and continuous designs (see Figure 10.4.10 to Figure 10.4.14). Cantilevered trusses often incorporate a "suspended" or "drop-in" span between two cantilevered spans (see Figures 10.4.15 and 10.4.16). The suspended span behaves as a simple span and is connected to cantilevered spans with pins or pin-and-hanger connections (see Figure 10.4.17). The back span on a cantilever truss is called the anchor span.

Figure 10.4.10 Single (Simple) Span Camel Back Pratt Truss

Figure 10.4.11 Single (Simple) Span Through Truss

Figure 10.4.12 Multiple Span Pony Truss

Figure 10.4.13 Multiple Span Through Truss

Figure 10.4.14 Continuous Through Truss

Figure 10.4.15 Cantilever Deck Truss

Figure 10.4.16 Cantilever Through Truss

Figure 10.4.17 Pin-and-Hanger Assembly for Cantilevered Truss

As stated earlier, a truss can be thought of as a very deep girder with portions of the web cut out. Truss members are divided in to three groups:

- Top or upper chord members
- Bottom or lower chord members
- Web members (diagonals and verticals)

See Figure 10.4.18 for truss members, floor systems, and various bracing configurations.

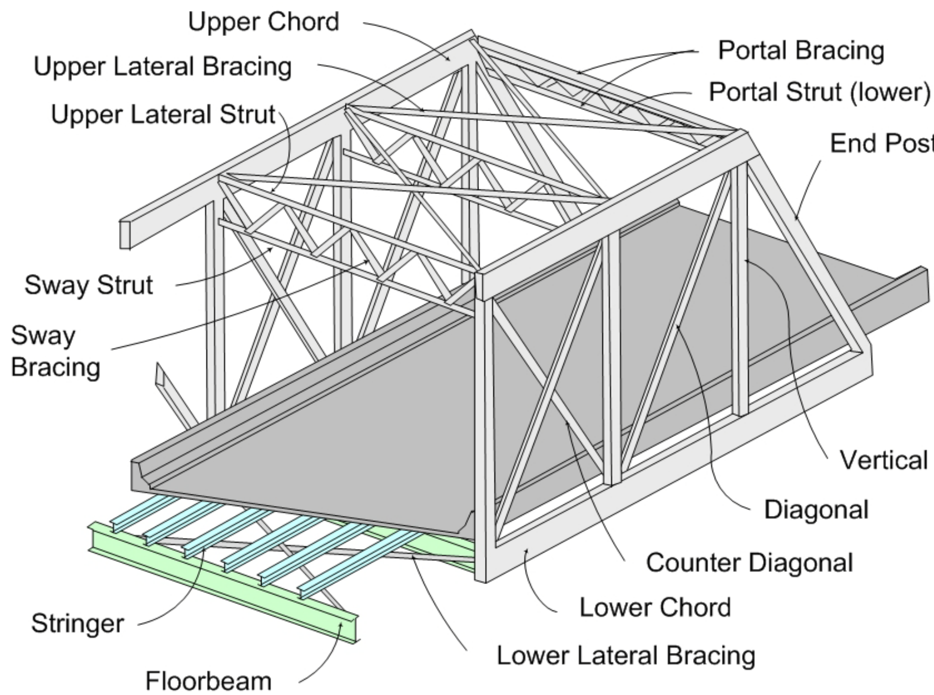


Figure 10.4.18 Truss Members, Floor Systems and Bracing

Truss members are fabricated from eyebars and rolled shapes (see Figure 10.4.19).

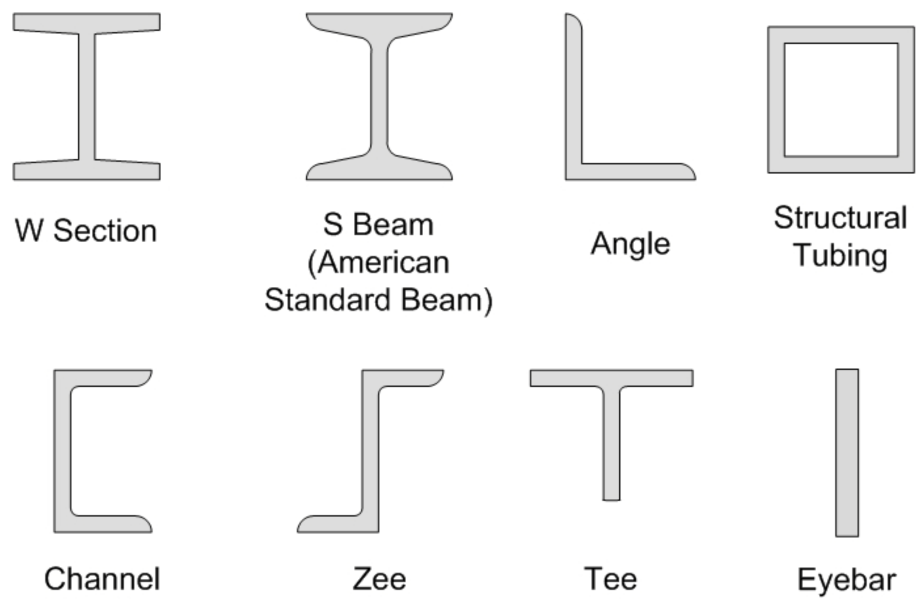


Figure 10.4.19 Rolled Steel Shapes

Trusses also utilize built-up sections. Built-up sections are fabricated by either bolting, riveting, or welding rolled shapes together. Built-up sections can also be custom designed to be efficient for expected design loads (see Figure 10.4.20). They are desirable for members that carry compression because they can be configured to resist buckling. Box sections are popular for modern trusses because they provide a “clean” look and are easier to maintain.

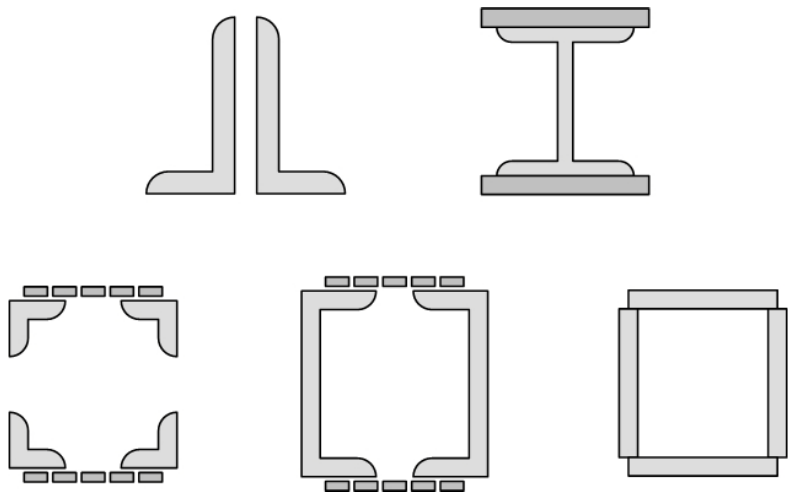


Figure 10.4.20 Built-Up Sections

Chord Members

Trusses, like beams and girders, support their loads by resisting bending. As the truss bends, the chord members behave like flanges of a beam and carry axial tension or compression forces (see Figure 10.4.21). On a simple span truss, the bottom chord is always in tension, while the top chord is always in compression. The diagonally sloped end post is a chord member. Top chords are also known as upper chords (U), and bottom chords are referred to as lower chords (L).

As truss bridge spans increase, cantilever and continuous designs are used, creating negative moment regions. Therefore, over an intermediate support, the top chord of a truss, like the top flange on a girder, is in tension (see Figure 10.4.21). It is common to find varying depth trusses on large complex structures, with the greatest depth at the supports where the moments are the largest (see Figure 10.4.14).

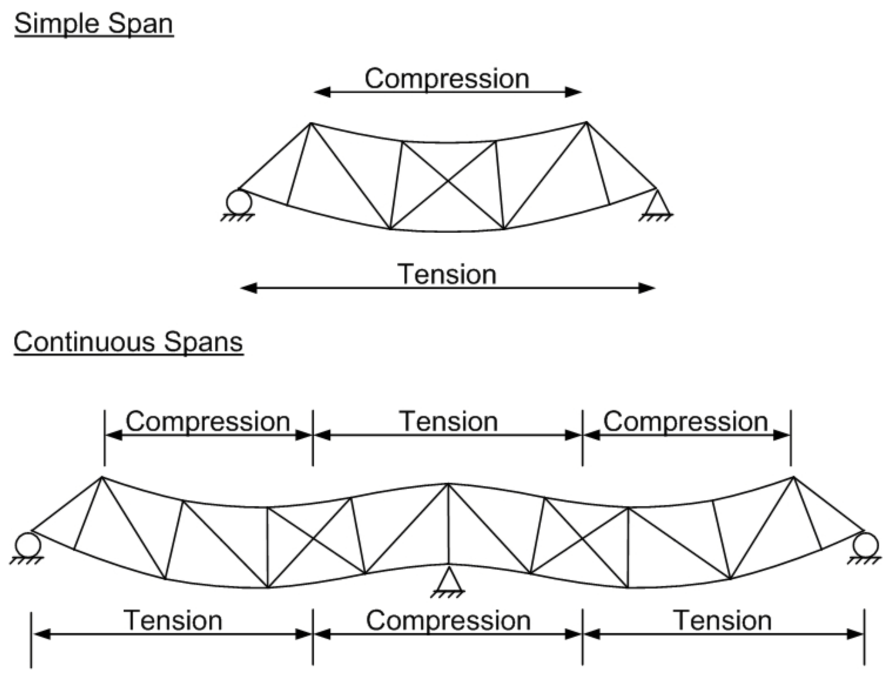


Figure 10.4.21 Axial Loads in Truss Chord Members

Web Members

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

Diagonals

For simple spans, an easy method to determine when a truss diagonal is in tension or compression is to use the "imaginary cable - imaginary arch" rule (see Figure 10.4.22). Diagonals that are symmetrical about midspan and point upward toward midspan, like an arch, are in compression. Diagonals that are symmetrical about midspan and point downward toward midspan, like a cable, are in tension. This rule applies only to simple span trusses.

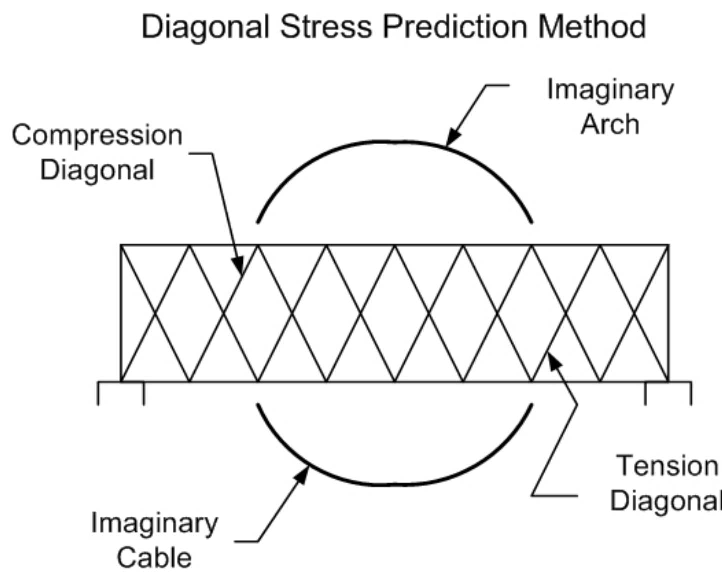


Figure 10.4.22 “Imaginary Cable – Imaginary Arch”

On older simple span trusses, the cross-section of the member can be used to determine which members are in tension and which are in compression. The design of a 25-foot member subjected to a tensile load requires a much smaller cross-member than a 25-foot member subjected to a compressive load of the same magnitude. On older pin-connected trusses, compression members are always the larger built-up members as compared to the tension members, which were often eyebar members. The Pratt truss, with its diagonals in tension, quickly replaced the Howe truss, whose diagonals are in compression. The Pratt truss is lighter and therefore less expensive to erect.

For trusses, counters are tension-resisting diagonals installed in the same panel in which the force reversal occurs. They are oriented opposite from each other, creating an "X" pattern. Counters are stressed only under live loads. On older bridges on which counters are bar shaped, they are typically capable of being moved by hand during an inspection. Counters are found on many older trusses but rarely on newer trusses.

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

As trusses become longer and, more importantly, as live loads become larger, the forces in some diagonals on a bridge continually change from tension to compression and back again. This situation occurs near the inflection points of continuous trusses. The inflection points in a continuous truss are similar to a continuous girder. The inflection points are located at the transition between positive and negative moments. Adjacent to the inflection joints, an unsymmetrical live load can cause large enough forces to overcome the symmetric dead load forces in the diagonals.

See Figure 10.4.18 of a sample truss schematic showing diagonals in a simply supported truss.

Verticals

There is also an easy method to determine when a vertical member is in tension or compression for a simply supported truss. Verticals that have one diagonal at each end are opposite to the force of the diagonals (see Figure 10.4.23). Verticals that have two diagonals at the same end are similar to the force in the diagonal closest to midspan (see Figure 10.4.24). Verticals that have counters on both ends are in compression (see Figure 10.4.25).

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

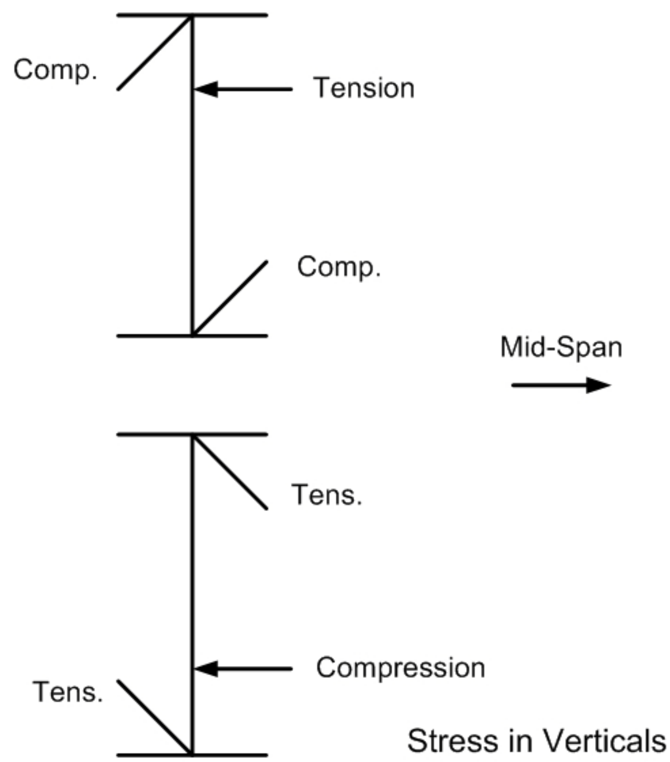


Figure 10.4.23 Vertical Member Stress Prediction Method

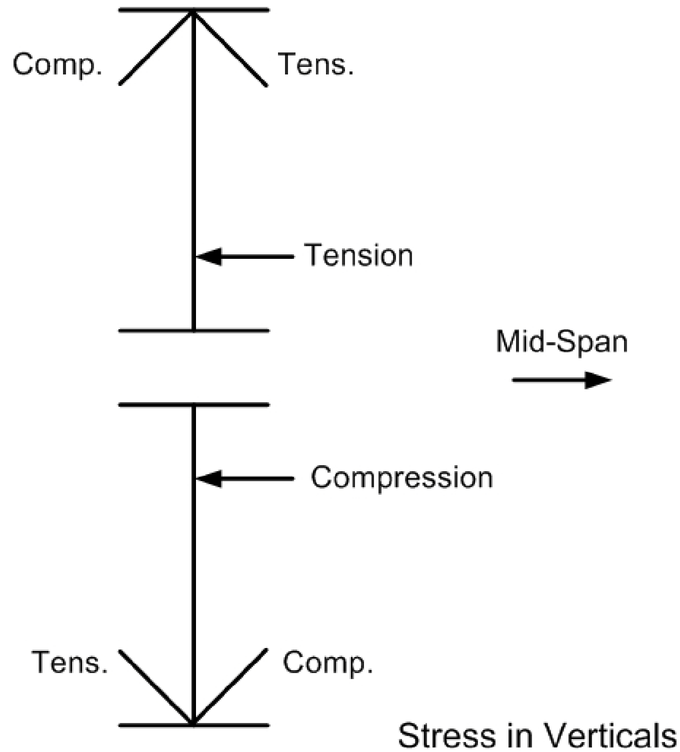


Figure 10.4.24 Vertical Member Stress Prediction Method

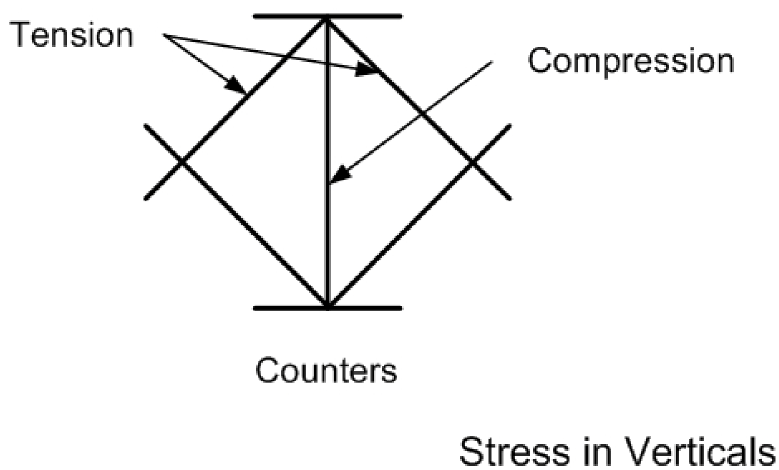


Figure 10.4.25 Vertical Member Stress Prediction Method

See Figure 10.4.18 of a sample truss schematic showing verticals in a simply supported truss.

Panel Points and Panels

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

The connections are typically made using gusset plates and are made by riveting, bolting, welding or a combination of these methods. Connections using both rivets and bolts were popular on bridges constructed in the late 1950's and early 1960's, as high strength bolts began to replace rivets. Rivets were used during shop fabrication while bolts were used to complete the connection in the field (see Figure 10.4.26).

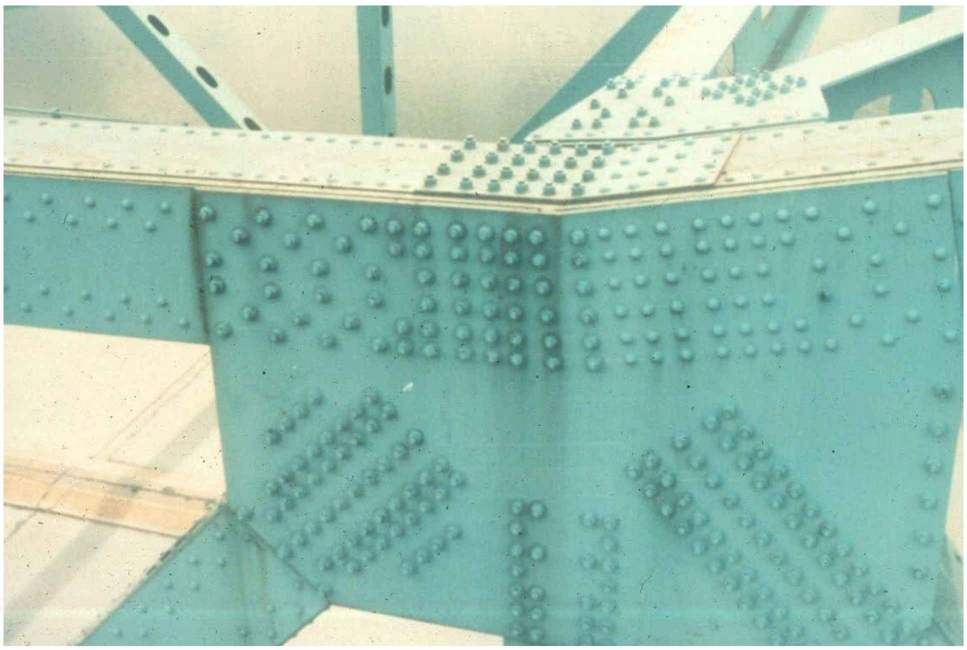


Figure 10.4.26 Truss Panel Point using Shop Rivets and Field Bolts

See Topic 10.8 for details and inspection methods of gusset plates.

Old trusses used pins at panel point connections (see Figure 10.4.27). Truss members may also be spliced, sometimes at locations other than the panel points.

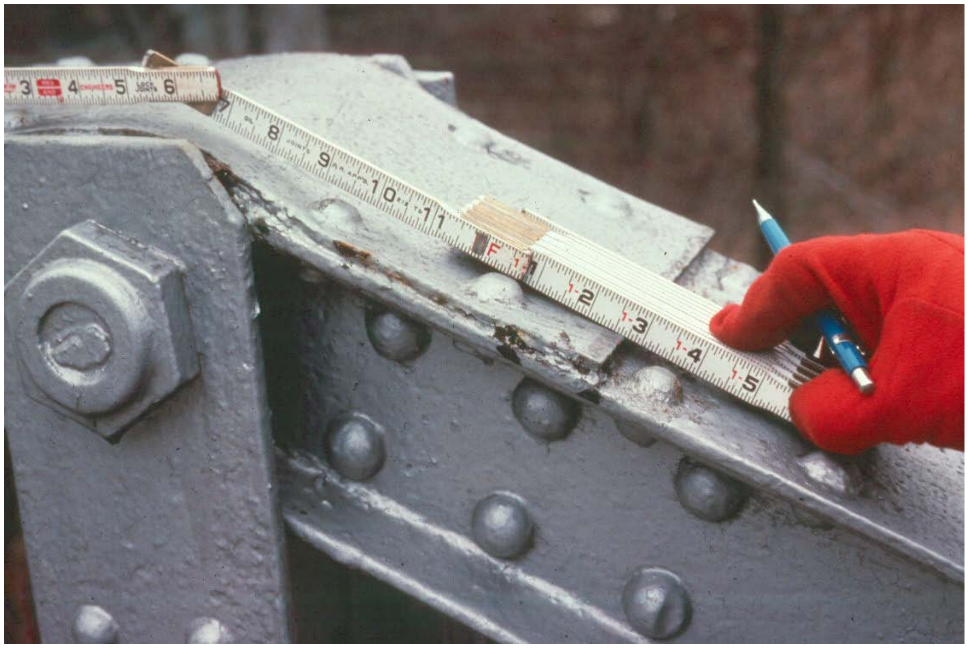


Figure 10.4.27 Pin Connected Truss

Either the letter U, for upper chord, or the letter L, for lower chord, or the letter M, for middle chord designates a panel point. Additionally, the panel points are numbered from bearing to bearing, beginning with 0 (zero). Most trusses begin with panel point L_0. Some deck trusses may begin with U_0. Upper and lower panel points of the same number are always in a vertical line with each other (e.g., U_7 is directly above L_7) (see Figures 10.4.28 and 10.4.29).

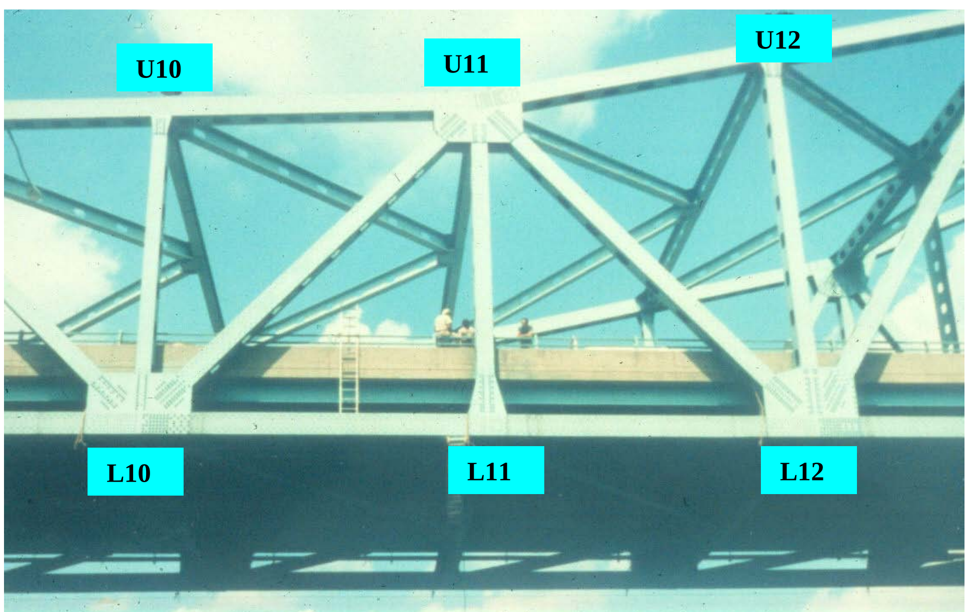


Figure 10.4.28 Truss Panel Point Numbering System

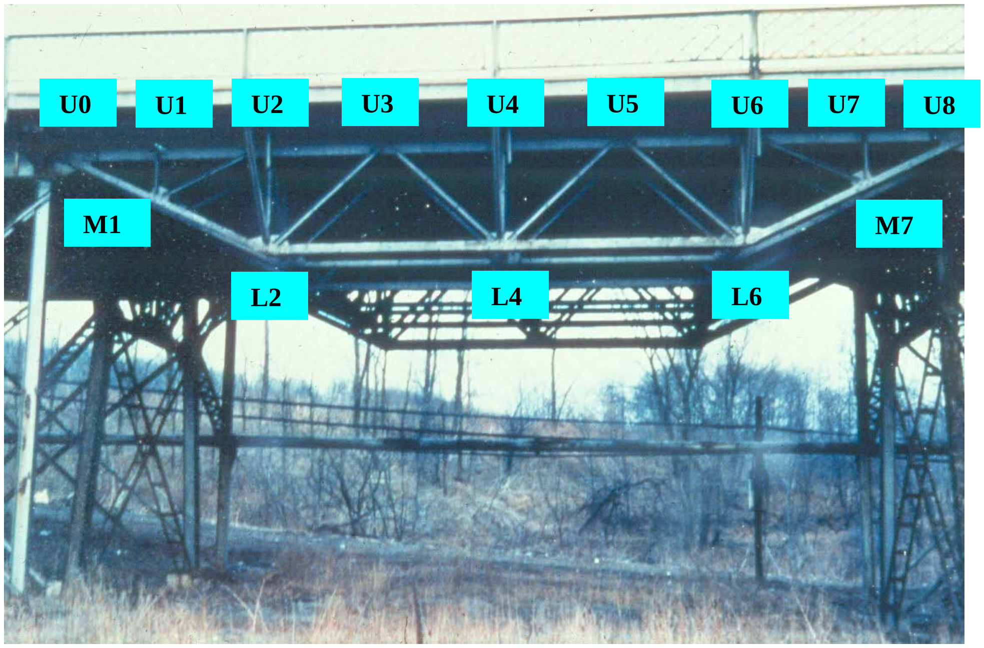


Figure 10.4.29 Deck Truss

A panel is the space, or distance, between panel points. Truss panels are typically 20 to 25 feet long and range 16 to 32 feet deep. The panel length is a design compromise between cost and weight, with the longer panels requiring heavier floor systems.

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

To obtain a lighter floor system, designers subdivided the panels. The midpoint of each diagonal was braced with a downwardly inclined sub-diagonal in the opposite direction and with a sub-vertical down to the lower chord. Subpanel points are designated with the letter M. Sometimes, the "half" number of the adjoining panels is used for these diagonal midpoints (e.g., $M_{7\ 1/2}$). The method of subdividing the truss created a secondary truss system within the main truss to support additional floorbeams. Baltimore and Pennsylvania trusses, patented in the 1870’s, use this method. The K truss, a more recent design, accomplishes the same purpose (see Figure 10.4.30).

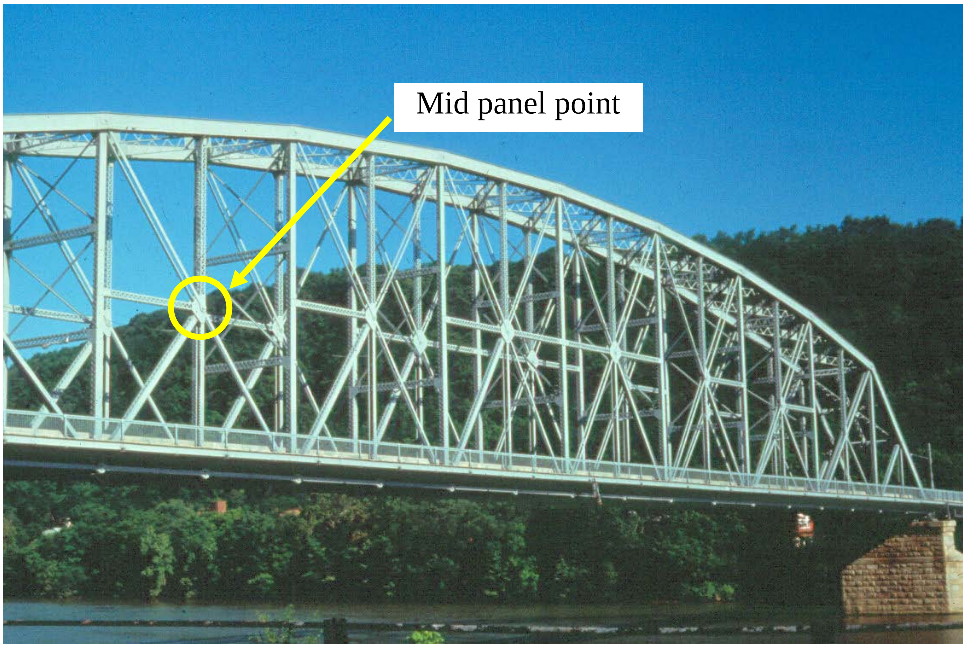


Figure 10.4.30 A Pennsylvania Truss, a Subdivided Pratt Truss with a Camel Back Top Chord

Floor System Arrangement

Most trusses have a floor system arrangement consisting of stringers and floorbeams similar to the two girder systems (see Figure 10.4.31). Floor systems support the deck and are supported by the trusses. Floor systems (floorbeams and stringers) are subjected to bending, shear and out-of-plane bending stresses. Trusses have floorbeams at each panel and sub panel point along the truss. Designate floorbeams by their panel point number. Some floor systems only contain floorbeams and no stringers (see Figure 10.4.32).

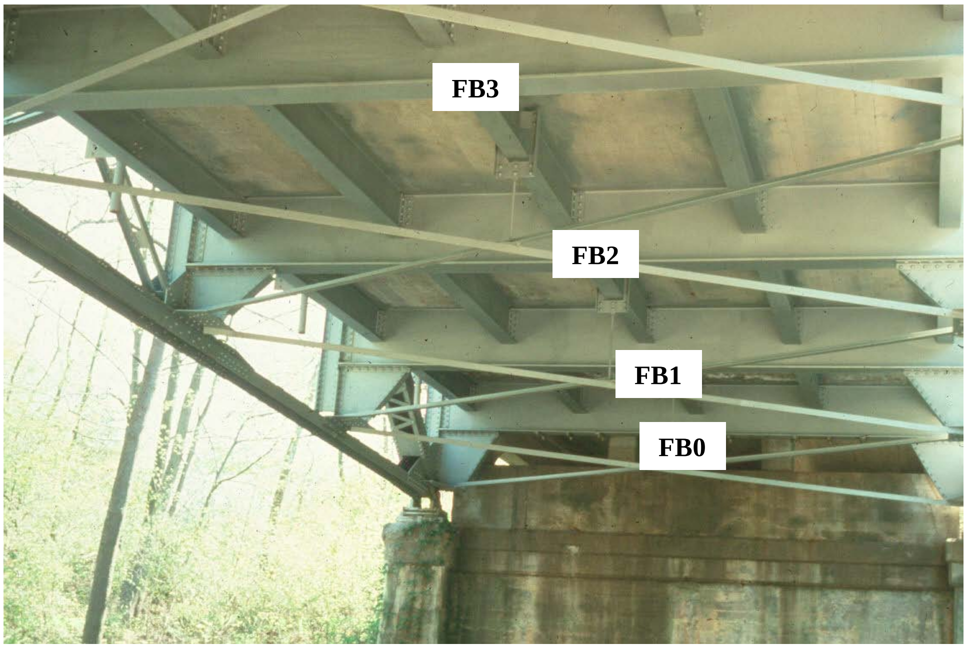


Figure 10.4.31 Floorbeam Stringer Floor System

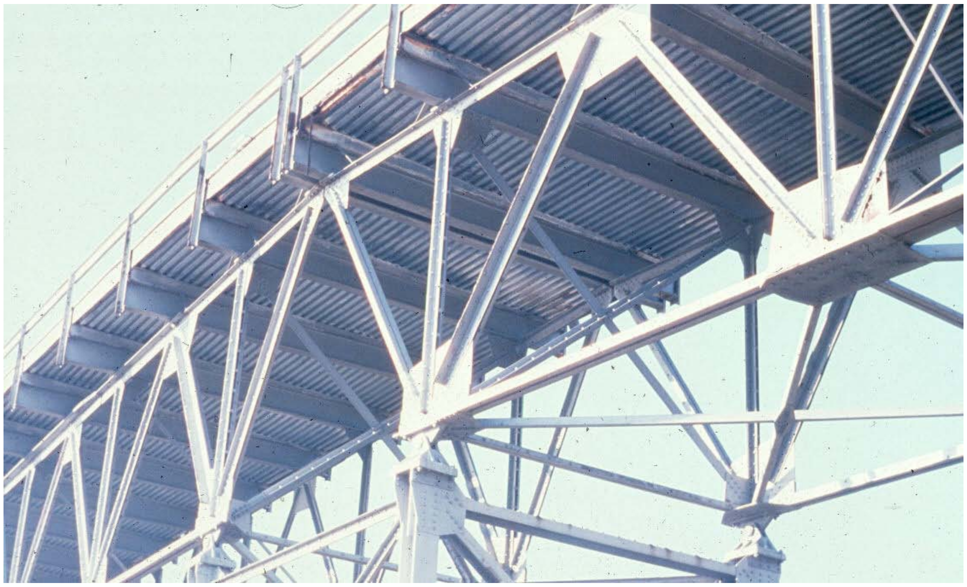


Figure 10.4.32 Floorbeam Floor System

See Figure 10.4.18 of a sample truss schematic showing a truss floor system consisting of floorbeams and stringers.

Lateral Bracing

Upper and lower lateral bracing is in a horizontal plane and functions to keep the two trusses longitudinally in line with each other and are considered secondary members. Most trusses have upper and lower chord lateral bracing, with the exception of pony trusses, which do not have upper lateral bracing. The bracing is typically constructed from built-up or rolled shapes and is connected diagonally to the chords and floorbeams at each panel point using gusset plates (see Figure 10.4.18, Figure 10.4.33, Figure 10.4.34 and Figure 10.4.35). Lateral bracing is subjected to tensile stresses caused by longitudinal or transverse loadings.

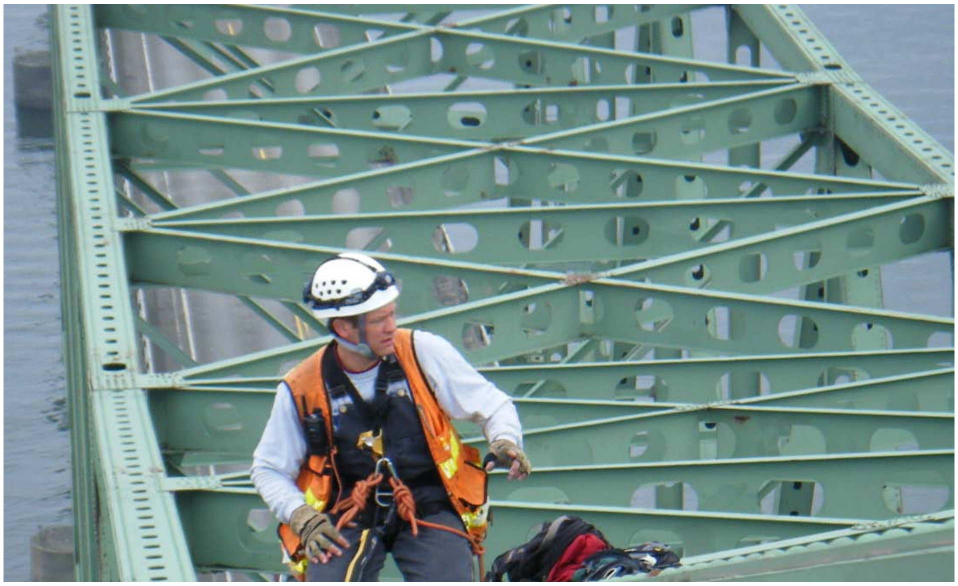


Figure 10.4.33 Inspection of Upper Lateral Bracing

Figure 10.4.34 Lower Lateral Bracing

Figure 10.4.35 Lateral Bracing Gusset Plate

Sway and Portal Bracing Sway bracing is in a vertical plane and functions to keep the two trusses parallel and are considered secondary members. The bracing is typically constructed from built-up or rolled shapes. The sway bracing at the end diagonal is called portal bracing and is much heavier than the other sway bracing. Sway bracing on old through trusses often limits the vertical clearance, and it therefore often suffers collision damage. Large pony trusses also have sway bracing in the form of a transverse diagonal brace from top chord to bottom chord (see Figures 10.4.18, 10.4.36, 10.4.37, 10.4.38 and 10.4.39). Sway and portal bracing are subjected to compressive stress caused by transverse, horizontal loads. They also help resist buckling of axial compression in truss chords.

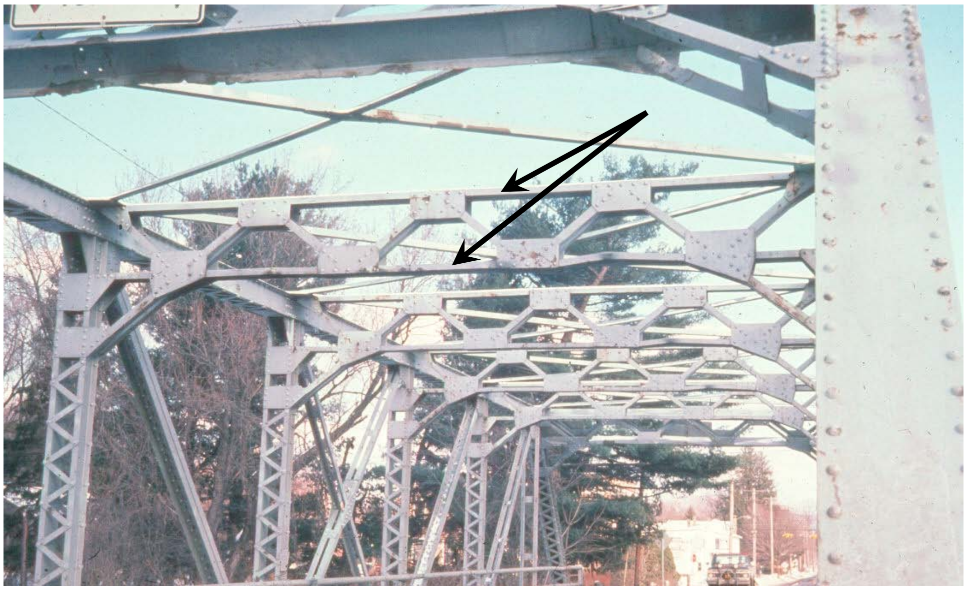


Figure 10.4.36 Sway Bracing

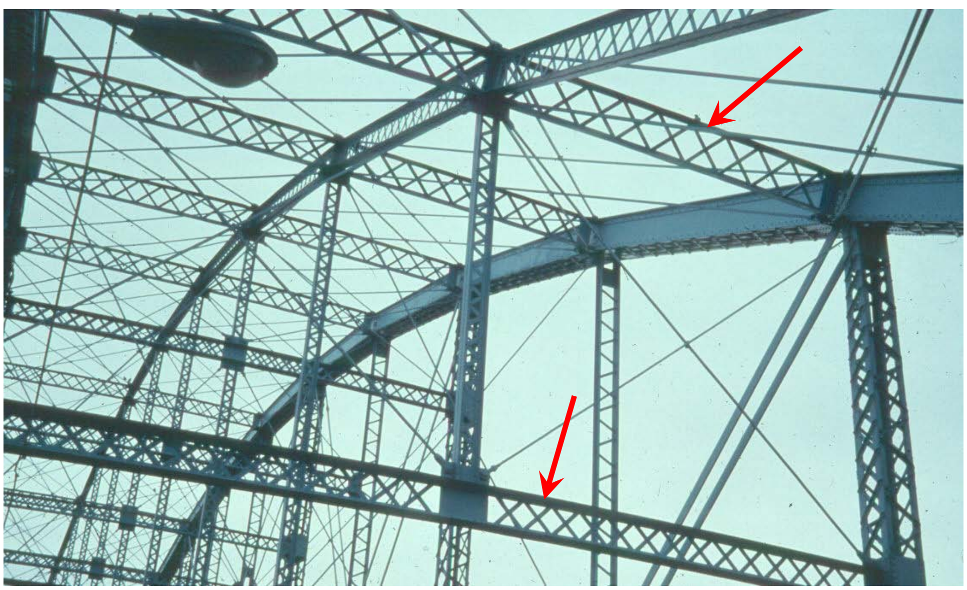


Figure 10.4.37 Sway Bracing

Figure 10.4.38 Portal Bracing with Attached Load Posting Sign

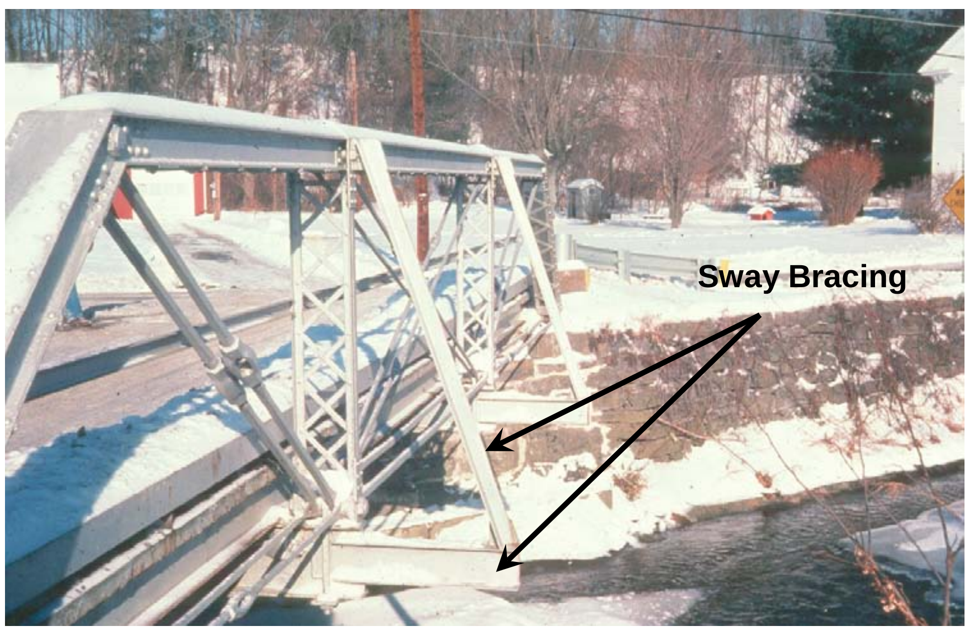


Figure 10.4.39 Pony Truss "Sway Bracing"

Primary and Secondary Members

Primary members carry permanent (dead) loads and transient (live) loads. Primary members are:

- Trusses (chords, verticals and diagonals)
- Floorbeams
- Stringers

These members are to the primary members that are evaluated during and load rating and control the load-carrying capacity of the bridge.

Secondary members resist horizontal and longitudinal loads and consist of:

- Portal bracing
- Lateral bracing
- Sway bracing

Secondary members do not contribute to the primary live load-carrying capacity of the bridge. Rather, they function only to keep the primary members properly aligned and resist secondary live loads.

Trusses, floor systems and bracing are shown on Figure 10.4.18.

Fracture Critical Members

Fracture Critical Member

A fracture critical member (FCM) is a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.

Truss

Steel trusses are considered fracture critical since they are constructed with fracture critical members. Common fracture critical members include chords, verticals, and diagonals that experience tension and do not have load path redundancy.

Trusses are considered fracture critical since they are constructed of steel, have tension members and a failure of a tension member may have cause a portion or the entire bridge to collapse (no load path redundancy). If a truss chord, vertical or diagonal, is a tension or stress reversal member, consider it fracture critical until a detailed structural analysis is performed.

Floorbeams

Steel truss bridges have floorbeams that are considered fracture critical members if one or more of the following conditions exist:

- Flexible or hinged connection to support at the floorbeam connection
- Floorbeam spacing greater than 14'-0"
- No stringers supporting the deck
- Stringers are configured as simple beams

Some states consider any floorbeam to be fracture critical regardless of the above conditions.

A three-dimensional finite element structural analysis for fracture criticality may be performed to determine the exact consequences to the bridge if a floorbeam or floorbeam connection fails.

10.4.3 Overview of Common Deficiencies

Common deficiencies that occur on steel truss bridges are:

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

See Topic 6.3 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 6.4 for Fatigue and Fracture in Steel Bridges.

10.4.4 Inspection Methods and Locations

Inspection methods to determine causes of steel deterioration are discussed in detail in Topic 6.3.

Methods

Visual

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

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

Physical

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

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

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

Advanced Inspection Methods

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

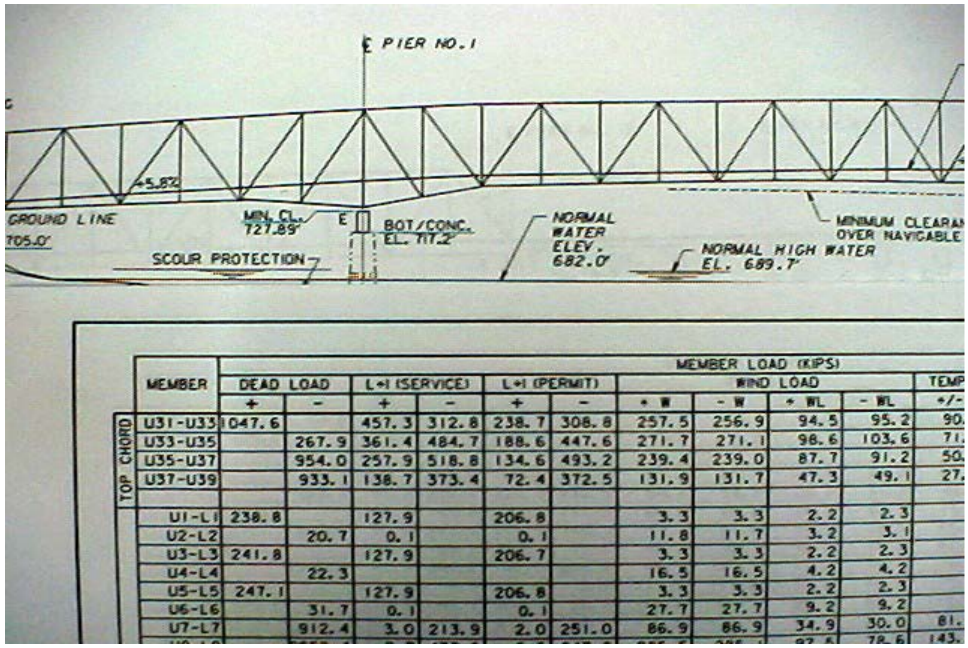
- Acoustic emissions testing
- Corrosion sensors
- Smart coatings
- Dye penetrant
- Magnetic particle
- Radiographic testing
- Computed Tomography
- Robotic inspection
- Ultrasonic testing
- Eddy current
- Strain evaluation
- Electrochemical fatigue sensor (EFS)

Other methods, described in Topic 15.3.3, include:

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

Locations

A truss consists of members, which are primarily under axial loading only. Furthermore, many truss members are designed for force reversal. If a review of the bridge's design drawings indicates that a member is subjected to tension and compression, inspect the member as a tension member subjected to cracking or elongation or as a compression member subjected to buckling (see Figure 10.4.40). Floor systems experience shear forces, tension and compression stresses caused by bending moments.



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

Figure 10.4.40 Truss Design Drawings: Member Load Table

Bearing Areas

Examine the web areas of the stringers, floorbeams and truss members over their supports for cracks, section loss and buckling. If web stiffeners are present at the supports, inspect them for cracks, section loss and buckling.

Examine the bearings at each support for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. See Topic 11.1 for a detailed presentation on the inspection of bearings.

Shear Zones

Examine the web areas near the supports for any section loss or buckling. Shear stresses are greatest near the supports. Therefore, the condition of the web is more critical near the supports than at mid-span. Also investigate the web for buckling due to overloads.

Tension Members

For truss members subjected to tensile loads, give special attention to the following locations:

- Check for section loss (corrosion) and cracks (see Figure 10.4.41).
- For box-shaped chord members, check inside for debris and corrosion, cracks or section loss (see Figure 10.4.42).
- Examine eyebar heads for cracks in the eyes and in the forge zone (see Figure 10.4.43).

- Check loop rods for cracking where the loop is formed (see Figure 10.4.44).
- Where multiple eyebars make one member, check that the tension is evenly distributed and that each eyebar within the member is parallel and evenly spaced to the adjacent eyebar. (see Figure 10.4.45).
- Check eyebars or loop rods where attachments are welded to them, especially if such attachments connect the eyebars together (see Figure 10.4.46).
- Determine whether the spacers on the pins are holding the eyebars and loop rods in their proper positions.
- Look for repairs, especially welded repairs, if they have been applied to steel tension members. Base metal cracks can develop at these locations (see Figure 10.4.46).
- Check the alignment of the members; make sure they are straight and not bowed, as this could be a sign of pier movement, collision damage or unintentional force reversal (see Figure 10.4.47).
- A member may be experiencing loads that were not intended during design, such as a member designed for tension that is now in compression. An example of this is a buckled bottom chord member in a simply supported truss (see Figures 10.4.47 and 10.4.48). Look for causes of the unintended loading and also look at adjacent members, which may be overstressed.
- Check the counters for excessive wear and abnormal rubbing where the counters cross.
- Check the tension in threaded members. Pull transversely (by hand) to check the relative tension. Proper tension allows the counter to move slightly. If improper tension is found, do not adjust the turnbuckle. Instead, promptly notify the designated point of contact for the bridge owner.

Figure 10.4.41 Corrosion and Section Loss on Truss Bottom Chord

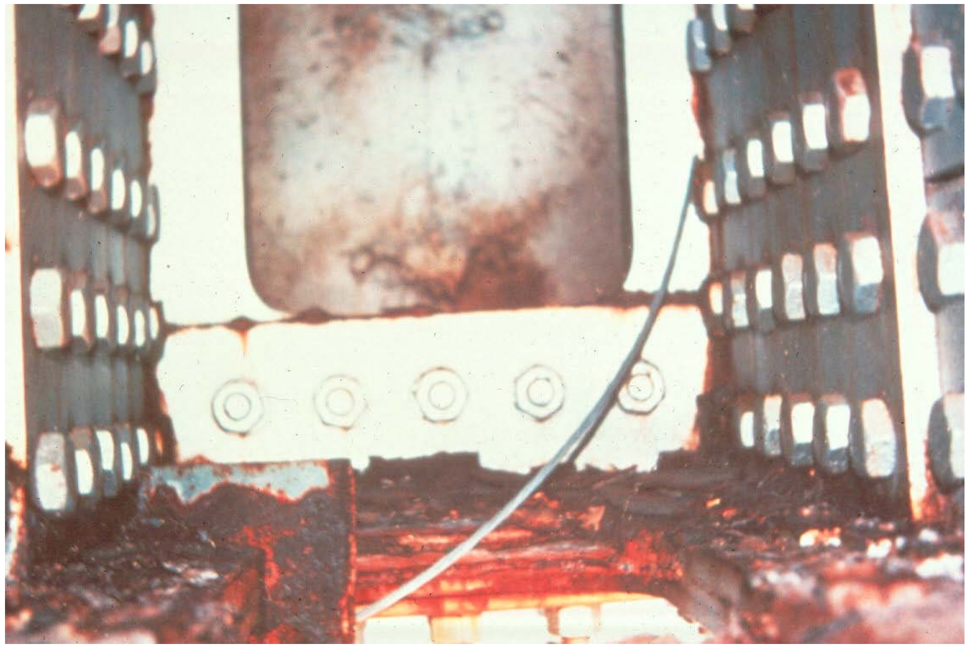


Figure 10.4.42 Inside of Box Chord Member

Figure 10.4.43 Cracked Forge Zone on an Eyebar

Figure 10.4.44 Cracked Forge Zone on a Loop Rod

Figure 10.4.45 Bottom Chord with Eyebars

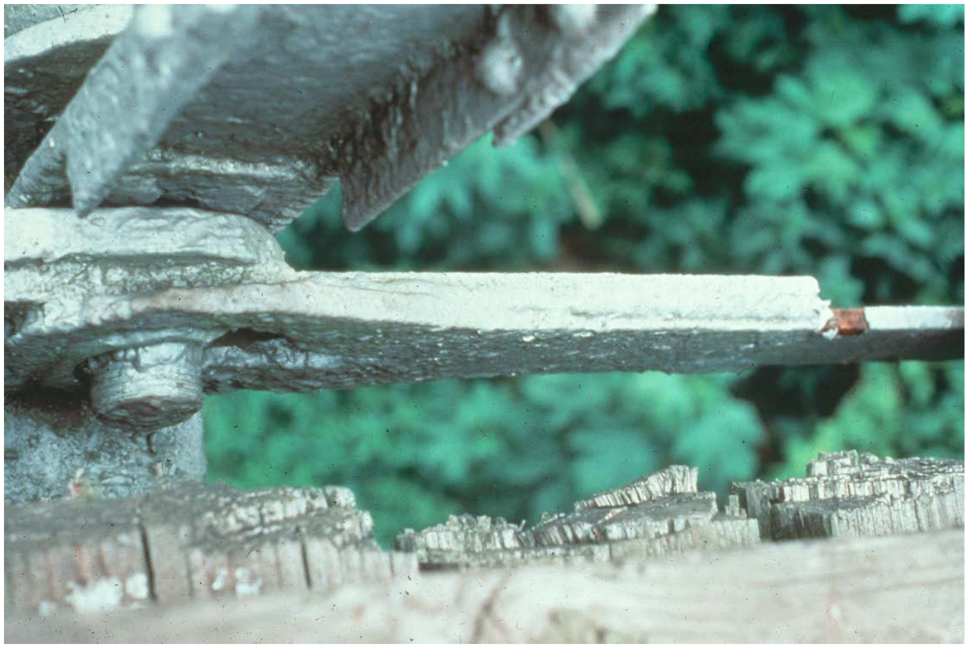


Figure 10.4.46 Welded Repair to Loop Rod

Figure 10.4.47 Bowed Bottom Chord Eyebar Member

Figure 10.4.48 Buckled Bottom Chord Member, Due to Abutment Movement

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

Compression Members

For truss members subjected to compressive loads, give special attention to the following locations:

- End posts, verticals and diagonals, which are vulnerable to collision damage from passing vehicles. Buckled, torn, or misaligned members may severely reduce the load carrying capacity of the member (see Figure 10.4.49).
- Check for local buckling, an indication of overstress (see Figure 10.4.50).
- Wrinkles or waves in the flanges, webs or cover plate are common forms of buckling.

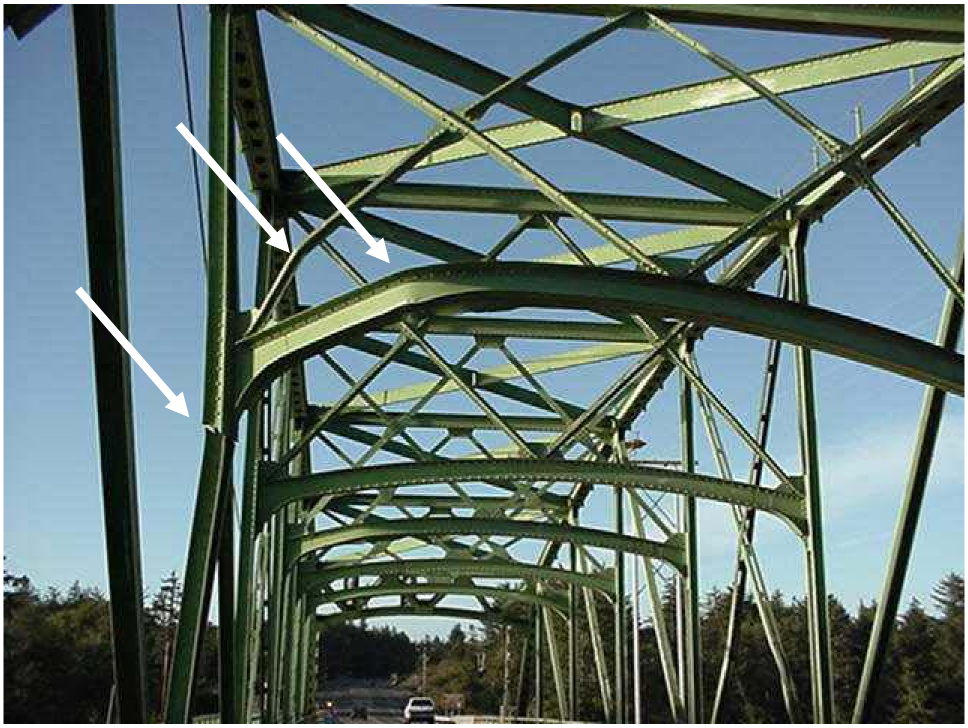


Figure 10.4.49 Collision Damage to Truss Members Due to Overheight Vehicle

Figure 10.4.50 Buckled End Post

Gusset Plates

For most truss bridges, gusset plates are the primary method of connecting individual truss members (chords, diagonals, verticals) together. Gusset plates may also be used to connect bracing members together. Unlike ordinary truss members, gusset plates experience axial tension and compression forces and shear forces simultaneously and subsequently require thorough inspection.

For steel truss gusset plates, give special attention to the following locations:

- Look for out-of-plane distortions particularly at areas susceptible to pack rust and along areas of unbraced length.
- Check for coating system failures, as these allow for advancement of corrosion and section loss (see Figure 10.4.51).
- Check for corrosion and section loss, comparing current measurements to original drawings.
- Check for cracking around tack welds, bolt and rivet holes, and other welds.
- Examine the gusset plate for loose or broken bolts and rivets, which can be detected through hammer sounding.
- Closely examine gusset plate retrofits, particularly welded retrofits as they can be problematic.

Refer to Topic 10.8.4 for detailed information on inspection locations and methods for gusset plates.

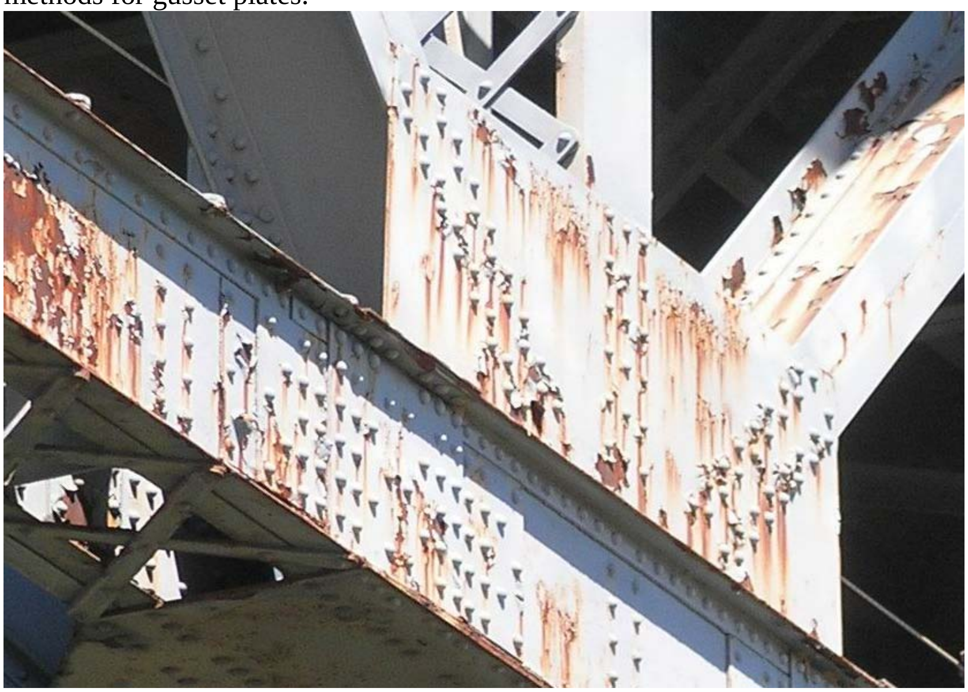


Figure 10.4.51 Gusset Plate Connection with Coating System Failure

Floor System

The floor system on a truss contains floorbeams and possibly stringers. These members function as beams and are subjected to bending, shear and out-of-plane bending stresses. Distortion induced fatigue cracks have also developed in the webs of many floorbeams at connections to truss bridge lower chord panel points when the stringers are placed above the floorbeams. The webs of these floorbeams at the connections and adjacent to flanges and stiffeners are inspected for signs of buckling.

For steel truss floor systems, give special attention to the following locations:

- Check the end connections of floorbeams for corrosion as they are exposed to moisture and deicing chemicals from the roadway (see Figure 10.4.52).
- Check the floorbeams and stringers for corrosion, particularly under open grid decks (see Figure 10.4.53).
- Check floor system member flanges and webs for corrosion and cracks (see Figures 10.4.54 and 10.4.55).
- During the passage of traffic, listen for abnormal noises caused by moving members and loose connections.

Figure 10.4.52 Corroded Floorbeam End and Connection with Deicing Chemical Residue

Figure 10.4.53 Corroded Stringers under an Open Grid Deck

Figure 10.4.54 Corroded End of Stringer

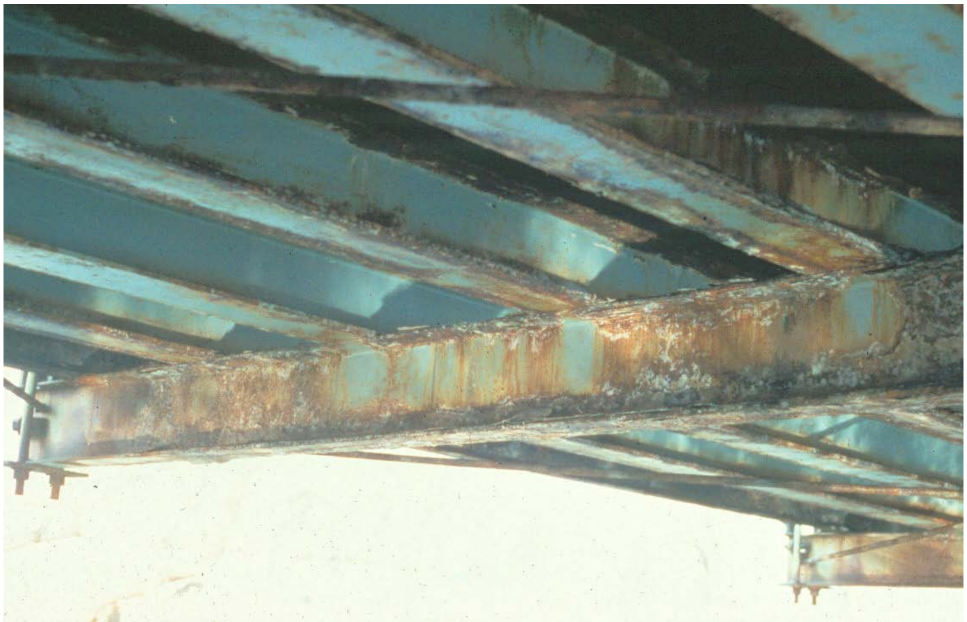


Figure 10.4.55 Corroded Floorbeams and Stringers

Problematic Details

Problematic details checked for deficiencies and deterioration include:

- Triaxial constraint
- Intersecting welds
- Cover plates
- Cantilevered suspended spans
- Insert plates
- Field welds: patch and splice plates
- Intermittent welds
- Out-of-plane bending
- Pin-and-hanger assemblies
- Back-up bars
- Mechanical fasteners and tack welds

Note that out-of-plane bending may occur at the following areas, which are presented further in Topic 6.4.8:

- Lateral bracing gussets and diaphragm connection plates

In addition to common problematic details, trusses also utilize the following:

- Stiffeners (transverse and longitudinal)
- Groove welded butt splices
- Lateral bracing gusset plates
- Web-to-flange welds
- Misc. connections (railing and utilities)

See Topic 6.4.8 for additional information on problematic details.

Secondary Members

Investigate the diaphragms, if present, and the connection areas of the lateral bracing for cracked welds, fatigue cracks, and loose fasteners. Check the lateral bracing gusset plates for corrosion. These horizontal plates typically deteriorate more rapidly than other elements on a truss because they are exposed to, and retain, moisture and deicing salts (see Figure 10.4.35). Inspect the bracing members for any distortion, or corrosion and pack rust (see Figure 10.4.57 and Figure 10.4.58). Distorted or cracked secondary members may be an indication the primary members may be overstressed or the substructure may be experiencing differential settlement.

For steel truss secondary members, check for collision damage at the portals and at knee braces (see Figure 10.4.56).

Figure 10.4.56 Collision Damage to Portal

Figure 10.4.57 Lateral Bracing with Corrosion

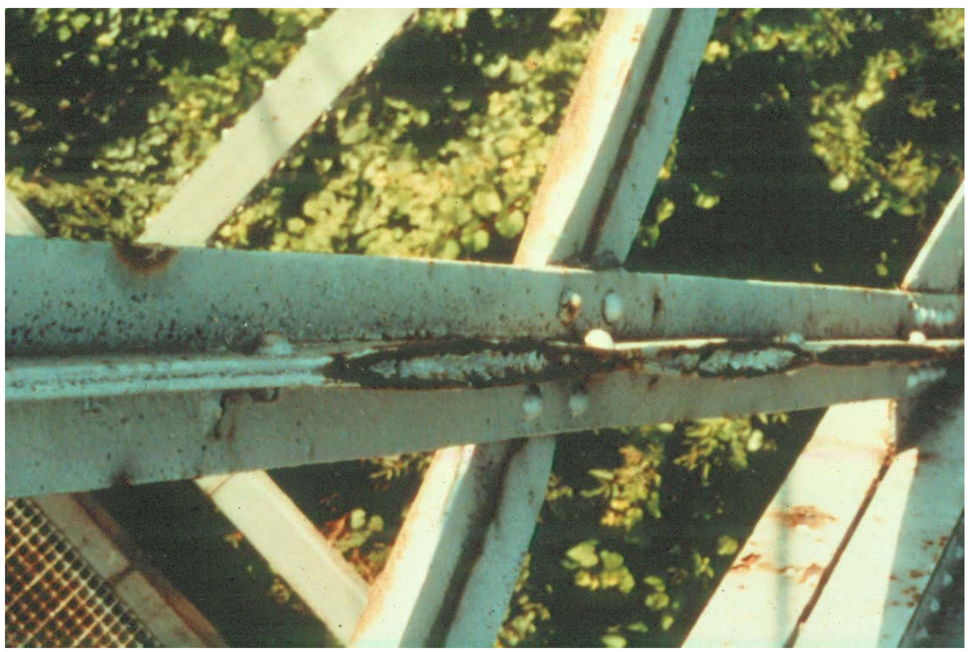


Figure 10.4.58 Sway Bracing with Pack Rust

Areas Exposed to Drainage

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

On steel truss bridges check:

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

Other Elements

Inspect chord members for corrosion, examining horizontal surfaces where moisture can collect. Check for corrosion and general deterioration of the lacing bars, stay plates, and batten plates (see Figure 10.4.59).

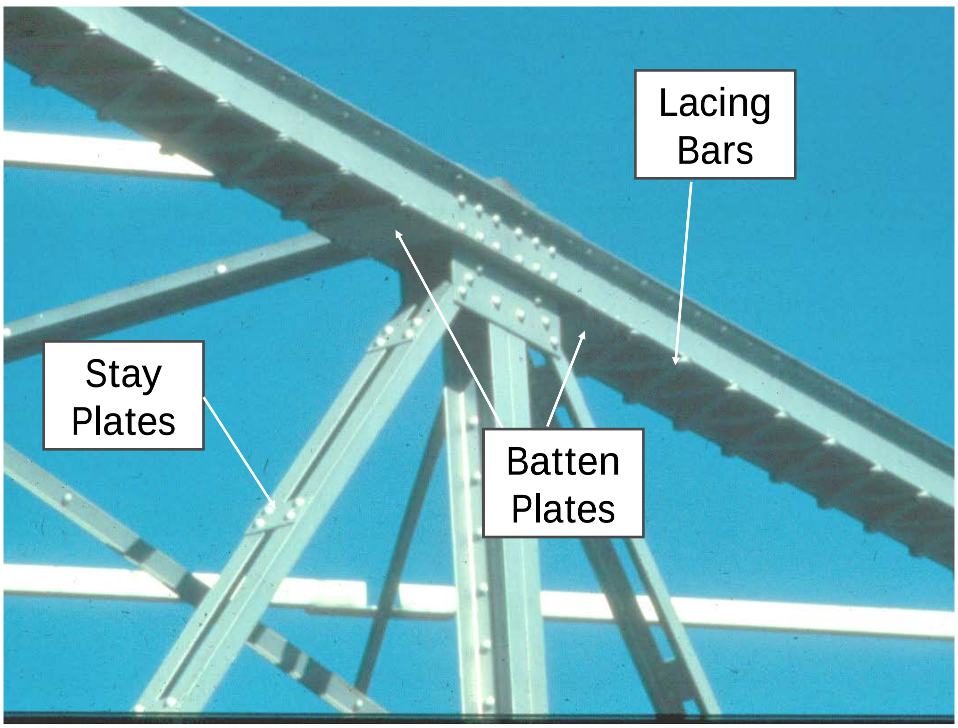


Figure 10.4.59 Other Elements

10.4.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

The component condition rating is only influenced by the condition of the primary load-carrying members.

Deficiencies such as corrosion, section loss, and fatigue cracks impact the superstructure rating. Note the location, dimensions and extent of the deficiencies on inspection forms and include supporting sketches and photos.

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

Element Level Condition State Assessment

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

NBE No.	Description
Superstructure	
113	Steel Stringer
120	Steel Truss
152	Steel Floor Beam
161	Steel Pin, Pin-and-Hanger Assembly, or both
162	Steel Gusset Plate

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

The unit quantity for the trusses, stringers (if applicable), and floorbeams is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. Note that the length of a truss is measured as the distance of each truss panel longitudinal to the roadway. The unit quantity for pins (if applicable) and gusset plates is each, with the total quantity distributed among the four available condition states depending on the severity of the deficiency. The unit quantity for steel protective coating is square feet, and the total area is distributed among the four conditions states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions. For pin-and-hanger assemblies, see Topic 10.7. For gusset plates, see Topic 10.8.

The following Defect Flags are applicable in the evaluation of steel truss superstructures:

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

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

Table of Contents

Chapter 10
Inspection and Evaluation of Steel Superstructures

Topic 10.5 Steel Arches

10.5.1 Introduction

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

Arches are divided into three types: deck, through, and tied (see Figures 10.5.1, 10.5.2, and 10.5.3).

Figure 10.5.1 Deck Arch Bridge

Figure 10.5.2 Through Arch Bridge

Figure 10.5.3 Tied Arch Bridge

10.5.2 Deck Arch Design Characteristics

General

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

Like its concrete counterpart, the steel open spandrel arch is designed to resist a load combination of axial compression and bending moment. The open spandrel steel arch is considered a deck arch since the roadway is above the arches (see Figure 10.5.4). The area between the arches and the roadway is called the spandrel.

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

Figure 10.5.4 Deck Arch

The arch members are called ribs and can be fabricated in to I-shapes, boxes, or truss shapes. The arches are classified as either solid ribbed, braced ribbed, or spandrel braced (see Figures 10.5.5, 10.5.6 and 10.5.7). The members are fabricated using riveted, bolted, or welded connections. Most steel deck arches have two arch rib members, although some structures have three or more ribs (see figure 10.5.7).

Figure 10.5.5 Solid Ribbed Deck Arch

Figure 10.5.6 Braced Rib Deck Arch, New River Gorge, WV

Figure 10.5.7 Spandrel Braced Deck Arch with Six Arch Ribs

An arch with a pin at each end of the arch is called a two-hinged arch (see Figure 10.5.8). If there is also a pin at the crown, or top, of the arch, it is a three-hinged arch. Steel one-hinged and fixed arches may exist, although these are very rare. Foundation conditions, in part, dictate the requirements for hinges. Three-hinged arches, for example, are not significantly affected by small foundation settlements.

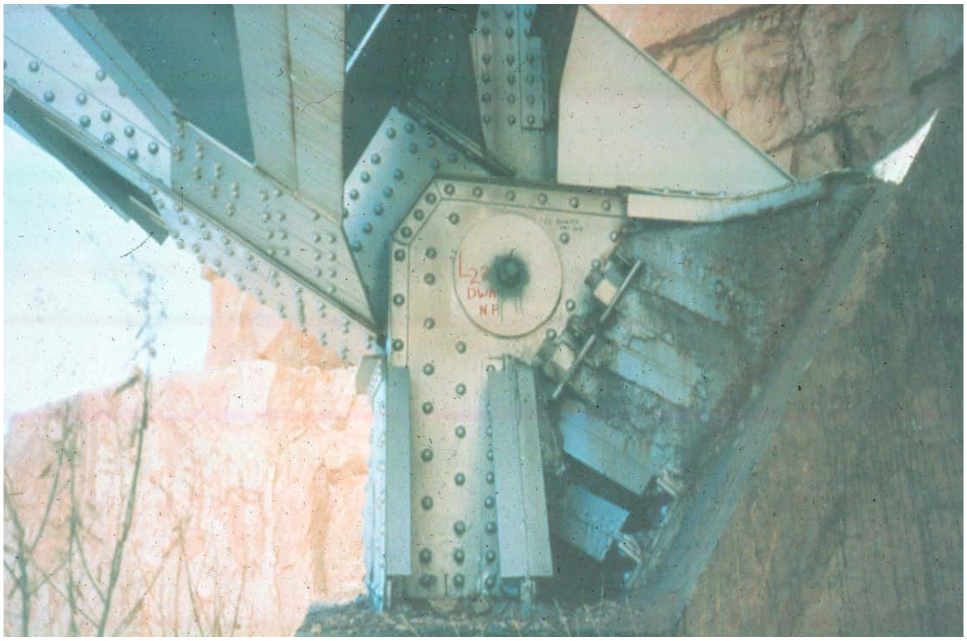


Figure 10.5.8   Hinge Pin at Skewback for Spandrel Braced Deck Arch

Primary and Secondary Members

The primary members of a deck arch bridge consist of the arches or ribs, spandrel columns or bents, spandrel girders and the floor system. The floor system consists of floorbeams and stringers (if present) (see Figure 10.5.9).

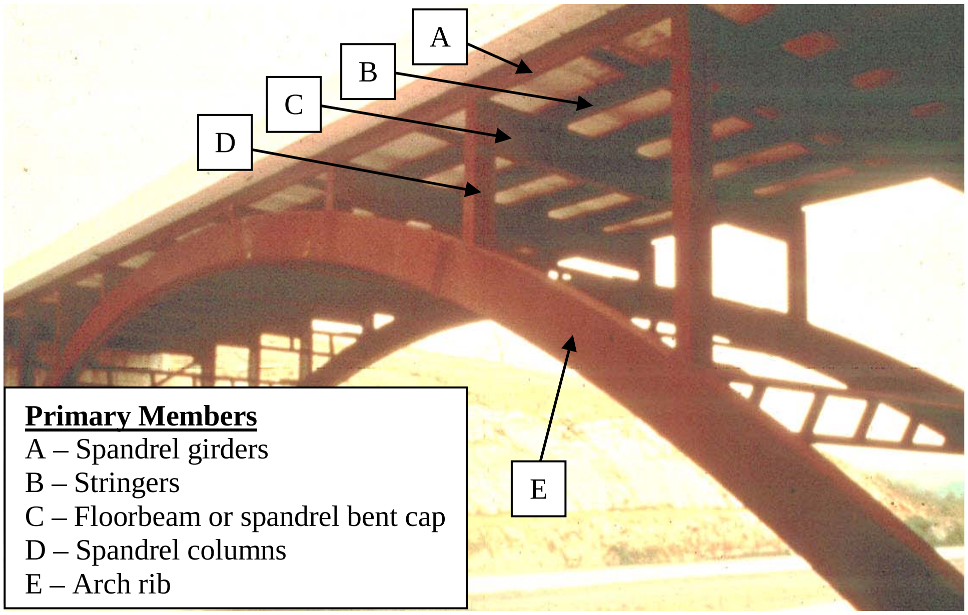


Figure 10.5.9   Solid Ribbed Deck Arch Primary Members

The secondary members of a deck arch bridge consist of the sway bracing and the upper lateral and lower lateral bracing of the arch or floor system (see Figure 10.5.10).

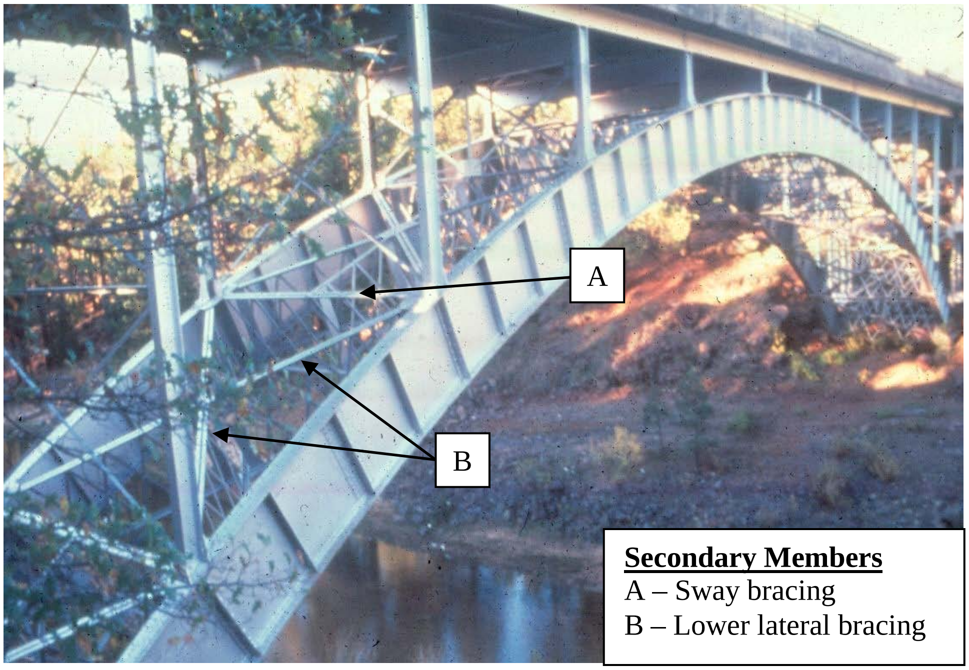


Figure 10.5.10 Solid Ribbed Deck Arch Secondary Members

Load Transfer

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

Fracture Critical Members

Most arches are built with two load paths (arches). However, the arch is not a tension member and is therefore not considered fracture critical.

Some members of the floor system and spandrel bent may be considered fracture critical A floorbeam may be fracture critical if it satisfies one or more of the following conditions:

- Flexible or hinged connection to support the floorbeam connection
- Floorbeam spacing greater than 14'-0"
- No stringers supporting the deck
- Stringers are configured as simple beams

Some states consider any floorbeam to be fracture critical even if all above conditions are met.

10.5.3 Through Arch Design Characteristics

General

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

The steel through arch is constructed with the crown of the arch above the roadway and the arch foundations below the roadway (see Figure 10.5.11). The deck is hung from the arch by wire rope cables or eyebars.

Figure 10.5.11 Elevation View of a Braced Ribbed Through Arch

The arch members are called ribs and are usually fabricated box-type members. Steel through arches are known as either solid ribbed or braced ribbed. The solid ribbed arch, which can be any type of arch, has a single curve defining the arch shape, while the braced ribbed arch has two curves defining the arch shape, braced with truss webbing between the curves. The lower curve is the bottom rib chord, and the upper curve is the top rib chord. The rib chord bracing consists of posts and diagonals. The braced ribbed arch is sometimes referred to as a trussed arch and is more common than the solid ribbed through arch (see Figure 10.5.11).

Primary and Secondary Members

The primary members of a through arch bridge consist of arch ribs (consisting of top and bottom rib chords and rib chord bracing); rib chord bracing, hangers and floor system including floorbeams and stringers (if present) (see Figure 10.5.12).

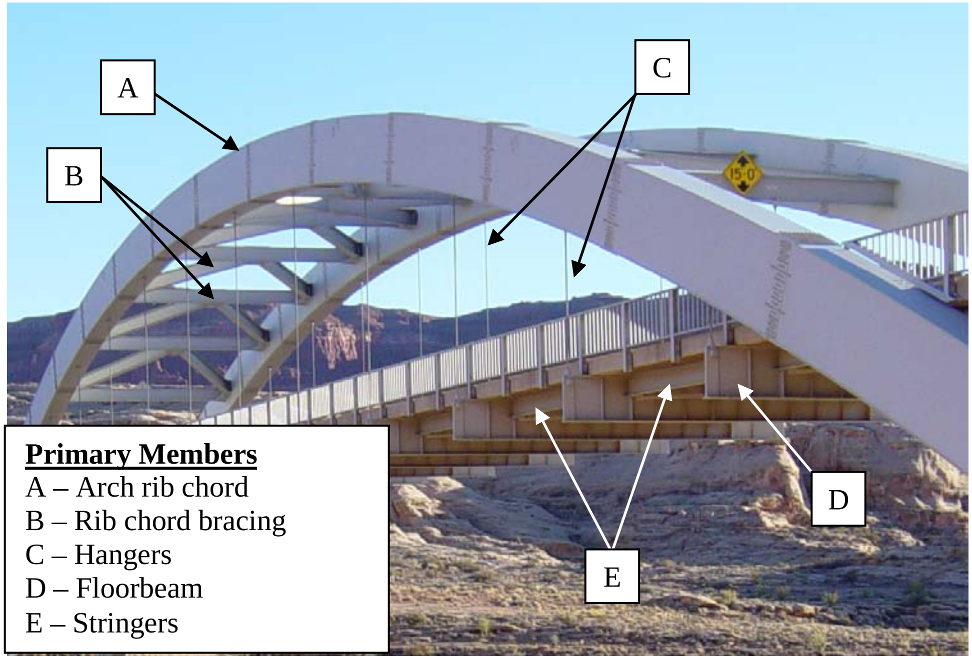


Figure 10.5.12 Through Arch Primary Members

The secondary members of a through arch bridge consist of sway bracing, lateral bracing (top and bottom rib chords and floor system) (see Figure 10.5.13).

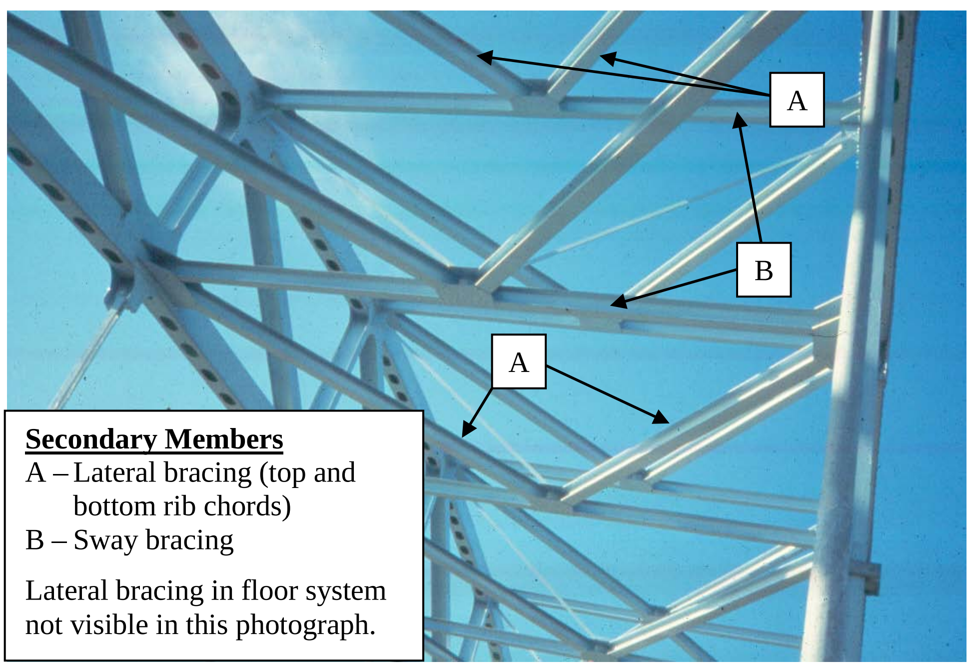


Figure 10.5.13 Through Arch Secondary Members

Load Transfer

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

Fracture Critical Members

The through arch is the main load-carrying member. Since there are typically only two arch ribs, the structure is nonredundant for load path. However, the bridge is not classified as fracture critical because the arches are not tension members. The hangers may be fracture critical, depending on the results of a detailed structural analysis. Some members of the floor system may be fracture critical as described in Topic 10.5.2.

10.5.4 Tied Arch Design Characteristics

General

The tied arch is a variation of the through arch with one significant difference. In a through arch, the horizontal thrust of the arch reactions is transferred to large rock, masonry, or concrete foundations. A tied arch transfers the horizontal reactions through a horizontal tie which connects the ends of the arch together, like the string on an archer's bow (see Figure 10.5.14). The tie is a tension member. If the string of a bow is cut, the bow springs open. Similarly, if the arch tie fails, the arch loses its compression and collapses.

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

Figure 10.5.14 Three-Span Tied Arch Bridge

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

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

Primary and Secondary Members

The primary members of a tied arch bridge consist of arch ribs, tie members, rib bracing truss (if present), hangers, and floor system including floorbeams and stringers (if present) (see Figure 10.5.15).

The secondary members of a tied arch bridge consist of sway bracing, lateral bracing (arch rib, top chord and floor system) (see Figure 10.5.16).

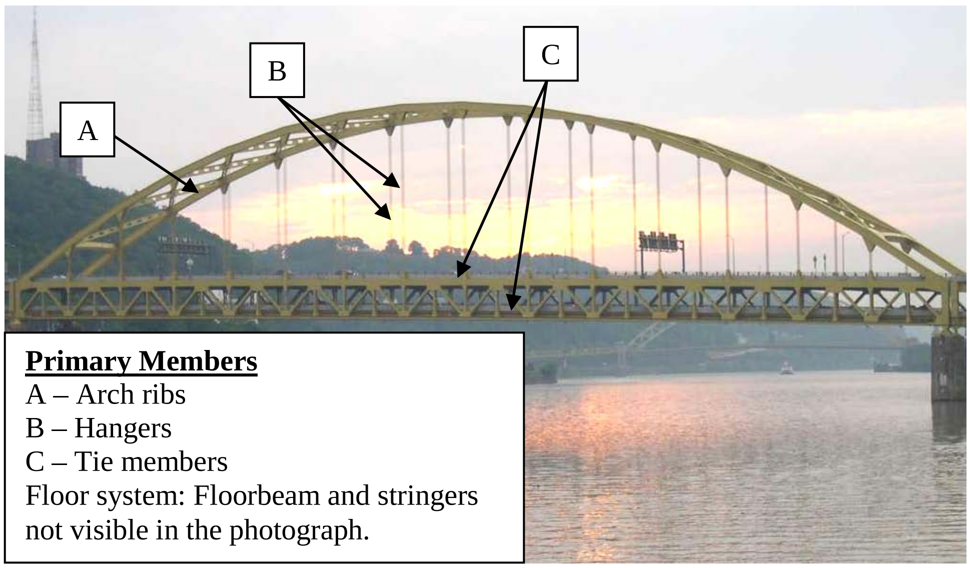


Figure 10.5.15 Tied Arch Primary Members

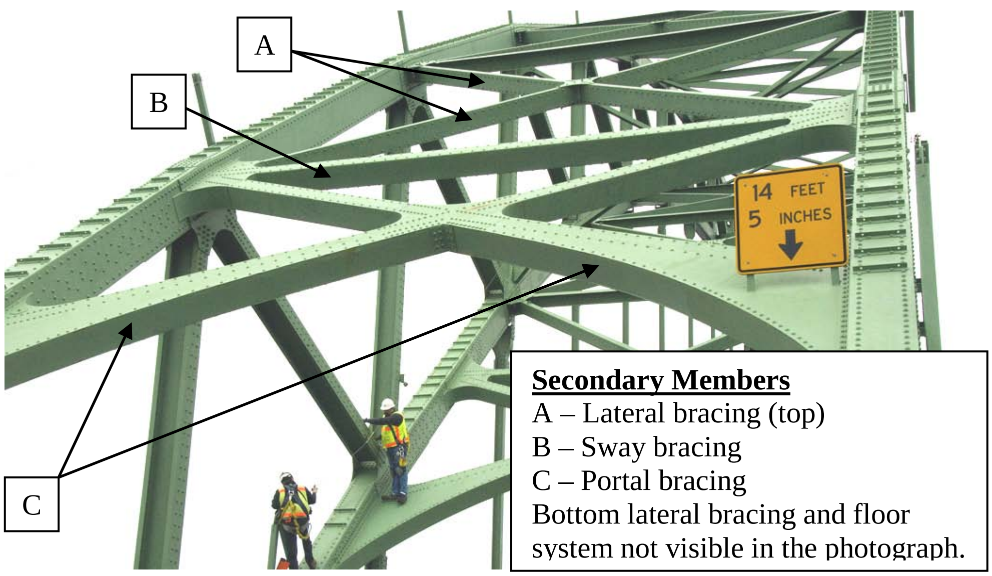


Figure 10.5.16 Tied Arch Secondary Members

Load Transfer

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

Fracture Critical Members

With only two load paths, arches are considered non-redundant structures. The arches are not fracture critical since they are subjected to axial compression. The tie girders, on the other hand, are axial tension members and are considered fracture critical (see Figure 10.5.17). Floor systems are similar to those discussed in Topic 10.5.2 and may be considered fracture critical.

Figure 10.5.17 Tied Arch Bridge with Fracture Critical Eyebar Tie Members

10.5.5 Overview of Common Deficiencies

Common deficiencies that occur on steel arch bridges are:

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

See Topic 6.3 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 6.4 for Fatigue and Fracture in Steel Bridges.

10.5.6 Inspection Methods and Locations

Inspection methods to determine causes of steel deterioration are discussed in detail in Topic 6.3.

Methods

Visual

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

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

Physical

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

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

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

Advanced Inspection Methods

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

- Acoustic emissions testing
- Corrosion sensors
- Smart coatings
- Dye penetrant

- Magnetic particle
- Radiographic testing
- Computed tomography
- Robotic inspection
- Ultrasonic testing
- Eddy current
- Electrochemical fatigue sensor (EFS)

Other methods, described in Topic 15.3.3, include:

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

Locations

When inspecting steel arch members and its floor systems (see Figure 10.5.18), it is important to check the bearing areas, shear zones, and flexure zones as described below.

Bearing Area

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

Examine the bearings on each of the supports for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion. (see Topic 11.1).

Flexure Zones

Examine the entire length of the spandrel beam or girder, floorbeam and stringer between the supports. Investigate the tension and compression flanges for corrosion, loss of section, cracks, dings, scrapes and gouges. Check the flanges in high stress areas for bending or flexure-related damage. Examine the compression flange for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. On continuous spans, the beams or girders over the intermediate supports have high flexural stresses due to negative moment. If welded cover plates are present, check carefully at the ends of the cover plates for cracks due to fatigue.

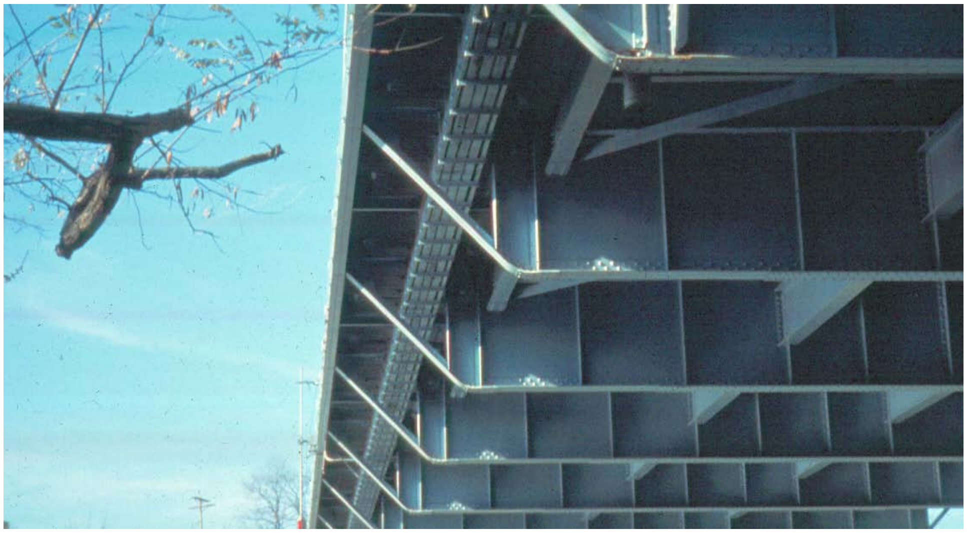


Figure 10.5.18 Floor System on a Through Arch

Arch Members

Arches are designed to primarily resist axial compression. Inspect the alignment of the arch and look for signs of buckling and crippling in the arch ribs (see Figure 10.5.19). Check for general corrosion and deterioration Examine any pins for corrosion and wear. Check the arch rib splice plates and the connections at the hangers or spandrel bents.

For arch members subjected to compression, give special attention to the end posts, web members, which are vulnerable to collision damage from passing vehicles. Buckled, torn, or misaligned members may severely reduce the load carrying capacity of the member. Check for local buckling, an indication of overstress. Wrinkles or waves in the flanges, webs or cover plate are common forms of buckling.

Figure 10.5.19 Through Truss Arch Members

Spandrel Members (Deck Arch)

Examine the end connections of the spandrel bents, spandrel columns and spandrel girders for cracks and loose fasteners. Check the spandrel girders and caps and columns for flexure, section loss, and buckling damage (see Figure 10.5.20).Check the bracing connections where attached to the spandrel members.

Figure 10.5.20 Braced Rib Deck Arch Showing Spandrel Columns

Hangers (Through and Tied Arches)

Hangers are designed to resist axial tensile loads and consist of either eyebars or cables. Check the connections at both ends of the hangers. Look for corrosion, cracks, and broken or misaligned wire strands. Examine the alignment of the hangers; the hangers may be near traffic, so inspect for collision or fire damage (see Figures 10.5.21 and 10.5.22). Check the hangers for any welded attachment; examine the welds between the attachment and the hanger for cracks.

Figure 10.5.21 Hanger Connection on a Through Arch

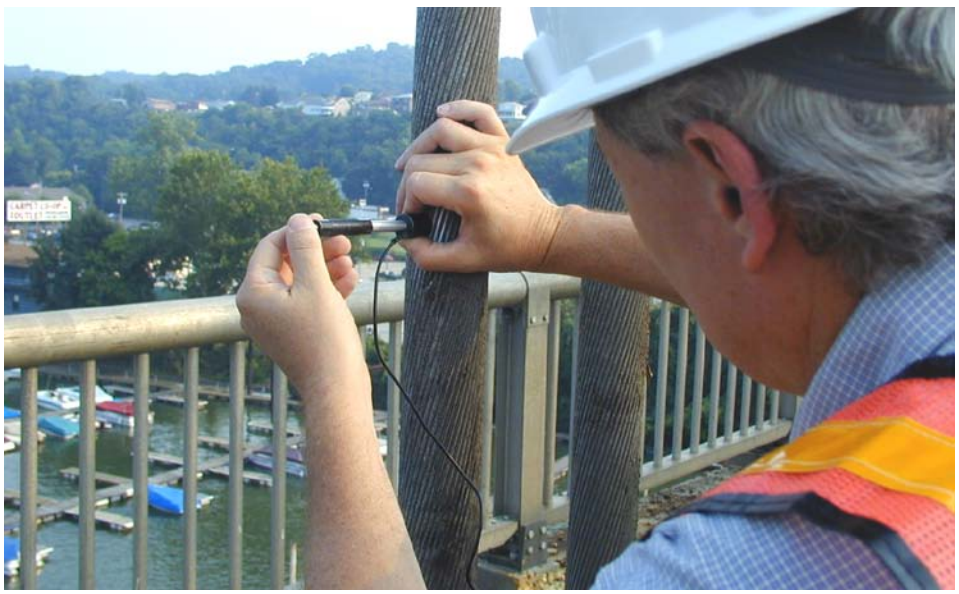


Figure 10.5.22 Performing Baseline Hardness Test on Fire Damaged Arch Cables

Gusset Plates

For arch bridges incorporating a deck, through or tied arch, gusset plates are the primary method of connecting individual arch members and vertical members together. Gusset plates may also be used to connect arch bracing members together. Thorough inspection is required for gusset plates, since these connections experience axial and shearing forces simultaneously.

For steel arches with gusset plates, give special attention to the following locations:

- Look for out-of-plane distortions particularly at areas susceptible to pack rust and along areas of unbraced length.
- Check for coating system failures, as these allow for advancement of corrosion and section loss.
- Check for corrosion and section loss, comparing current measurements to original drawings.
- Check for cracking around tack welds, bolt/rivet holes, and other welds.
- Examine the gusset plate for loose or broken rivets/bolts, which can be detected through hammer sounding.
- Closely examine gusset plate retrofits, particularly welded retrofits as they are very problematic.

Refer to Topic 10.8.4 for detailed information on inspection locations and methods for gusset plates.

Tied Arches and Tie Girders

In tied arch bridges, axial compressive forces from the arch members are transferred into the tie girder. The tie girder resists axial tension. For tension members, inspect the following locations:

- Check for section loss (corrosion) and cracks.
- For box-shaped chord members, check inside for debris and corrosion, cracks or section loss.
- Examine eyebar heads for cracks in the eyes and in the forge zone.
- Check loop rods for cracking where the loop is formed.
- Where multiple eyebars make one member, check the tension is evenly distributed - each eyebar element perfectly parallel and evenly spaced to the adjacent elements.
- Check eyebars or loop rods where attachments are welded to them, especially if such attachments connect the eyebars together.
- Determine whether the spacers on the pins are holding the eyebars and loop rods in their proper positions.
- Look for repairs, especially welded repairs, if they have been applied to steel tension members. Base metal cracks can easily develop at these locations.
- Check the alignment of the members, make sure they are straight and not bowed - this could be a sign of pier movement, collision damage or

unintentional force reversal.

- Check the girders for welded attachments.

Check floorbeam-to-tie member connections for distortion caused by fatigue or horizontal floorbeam displacement in the webs of the floorbeams when the stringers are placed above the floorbeams.

Problematic Details

Problematic details checked for deficiencies and deterioration include:

- Triaxial constraint
- Intersecting welds
- Cover plates
- Cantilevered suspended spans
- Insert plates
- Field welds: patch and splice plates
- Intermittent welds
- Out-of-plane bending
- Pin-and-hanger assemblies
- Back-up bars
- Mechanical fasteners and tack welds

Note that out-of-plane bending may occur at the following areas, which are presented further in Topic 6.4.8:

- Girder web connections for diaphragms and floorbeams, including connections plates at the top flange, connection plates at the bottom flange, and connection plates at the bottom flange for skewed bridges
- Lateral bracing gussets and diaphragm connection plates
- Diaphragm connections to gusset plates
- Cantilevered floorbeams

In addition to the common problematic details, arches also utilize the following:

- Stiffeners (transverse or longitudinal)
- Groove welded butt splices
- Lateral bracing gusset plates
- Web-to-flange welds
- Miscellaneous connections (railing or utilities)

See Topic 6.4.8 for additional information on problematic details.

Secondary Members

Investigate the alignment of the bracing elements (see Figure 10.5.25). Check horizontal connection plates, which can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Examine the diaphragm and bracing connections for loose fasteners or cracked welds. This problem is most common on skewed bridges, and it has also been observed on bridges with a high frequency of combination truck loads. Check for distorted members. Distorted secondary members may be an indication the primary members may be overstressed or the substructure may be experiencing differential settlement. Examine the end connections for cracks, corrosion, and loose fasteners.

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

Figure 10.5.23 Bracing Members in Deck Arch Bridge

Areas Exposed to Traffic

Inspect any areas exposed to traffic for collision damage (see Figure 10.5.26). If collision damage is found, document the location and dimensions and reference with photographs and/or sketches.

Figure 10.5.24 Through Arch Member Exposed to Traffic

Areas Exposed to Drainage

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

On steel arch bridges check:

- Areas exposed to drainage runoff
- Lateral bracing gusset plates
- Inside built-up arch members
- Pockets created by floor system connections
- Tightly packed panel points
- Pin-and-hanger and cable assemblies
- Bottom flanges of arch members and floor system

10.5.7 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

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

NBE No.	Description
Superstructure	
102	Steel Closed Web/Box Girder
107	Steel Girder/Beam
113	Steel Stringer
141	Steel Arch
148	Steel Cable
152	Steel Floor Beam
161	Steel Pin, Pin-and-Hanger Assembly, or both
162	Steel Gusset Plate

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

The unit quantity for the closed web/box girder, girder/beam, stringer, arch, cable, floor beam, and pier cap is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for the pin-and-hanger assembly and gusset plate is each, with each pin-and-hanger element placed in one of the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions. For pin-and-hanger assemblies, see Topic 10.7. For gusset plates, see Topic 10.8.

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

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

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

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Chapter 10

Inspection and Evaluation of Steel Superstructures

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Topic 10.6 Steel Rigid Frames

10.6.1 Introduction

A frame consists of horizontal members rigidly attached to vertical or inclined members without the use of bearings (see Figure 10.6.1). Steel rigid frames are popular today in building construction because of their space-saving characteristics and aesthetics. The same principles that permit the omission of intermediate column supports in buildings are applied to bridge frames. In a steel rigid frame bridge structure, the frame sides or "legs" replace intermediate supports. Because the legs contribute to the structures overall capacity, increased span lengths and material savings can be realized.

Figure 10.6.1 Typical Rigid K-frame Constructed of Two Frames

10.6.2 Design Characteristics

General

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

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

The superstructure of a rigid frame bridge can be constructed of two frames similar to a two-girder bridge (see Figure 10.6.1) or of multiple frames in the same manner as a multi-girder bridge (see Figure 10.6.2). These frames can be thought of as fabricated girders with attached legs.

Figure 10.6.2 Typical Rigid Frame Constructed of Multiple Frames

K – Frames

Most steel rigid frame bridges are multi-span structures and are commonly referred to as "K-frame" or "grasshopper leg" bridges (see Figure 10.6.1). The sloping legs give the rigid frame a "K" shape, when looked at by rotating the frame counterclockwise 90 degrees. K-frames are not economical for very short or very long span bridges. Because of their aesthetically pleasing appearance, sometimes an effort is made to use steel rigid frames whenever possible.

It is possible to think that the legs of the K-frame look very much like piers and consider them part of the substructure. This is not the case because there is no bearing between the legs and the girder portion of the frame (see Figure 10.6.3).

Since there are no bearings between the legs and girder portion of the frame, bending forces are transferred between the girder portion and the legs (see Figure 10.6.3).

Figure 10.6.3 Connection Between Legs and Girder Portion

Delta Frames

In some designs, a triangular frame configuration can be used. For very long spans, two K-frames can be connected together end-to-end (see Figure 10.6.4). Instead of one of the end spans bearing on an abutment, it is connected to the end span of another K-frame. The bottoms of the legs are also connected together and share the same bearing. This type of configuration is known as a delta frame. The leg connections form an inverted triangle with the girder portion of the frame. The Greek letter Delta (∇) is the symbol used for this triangle.

Figure 10.6.4 Delta Frame

Regardless of the frame configuration, the entire portion of the bridge, (legs and girders) constitutes the frame, and is considered the superstructure. The legs of rigid frames are supported by relatively small concrete footings and bearings which are essentially hinges (see Figure 10.6.5).

Figure 10.6.5 Bearings

Stiffeners

Steel rigid frames may have up to three different types of stiffeners (see Figure 10.6.6).

Transverse Stiffeners

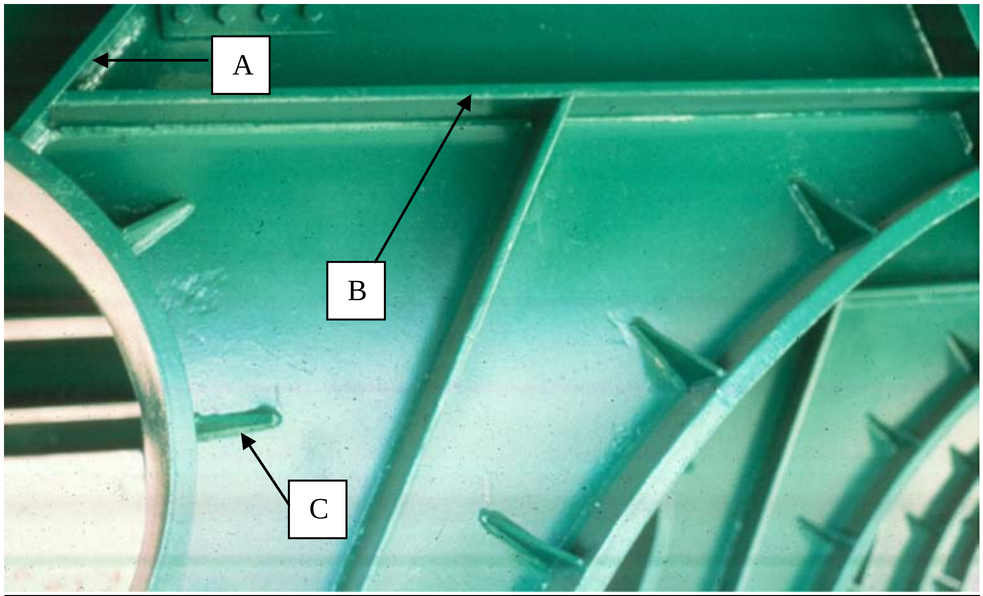
Transverse stiffeners are placed approximately perpendicular to the flanges and welded to the web and flanges of the frame at spacings required by design. Transverse stiffeners are used to prevent buckling in high shear regions.

Longitudinal Stiffeners

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

Radial Stiffeners

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



Stiffeners
A – Transverse
B – Longitudinal
C – Radial

Figure 10.6.6 Transverse, Longitudinal, and Radial Stiffeners on a Frame Knee

Floor System Arrangement

A rigid frame has one of three floor systems:

Multiple Frame System

For a multiple frame system, the deck is supported only by the frames.

Frame-Floorbeam System

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

Frame-Floorbeam-Stringer System

Longitudinal stringers, parallel to the frames, are connected to the floorbeams (see Figure 10.6.7). Floorbeams are connected to the girder portion of the two frames. The stringers are typically rolled sections and are supported by the floorbeams.

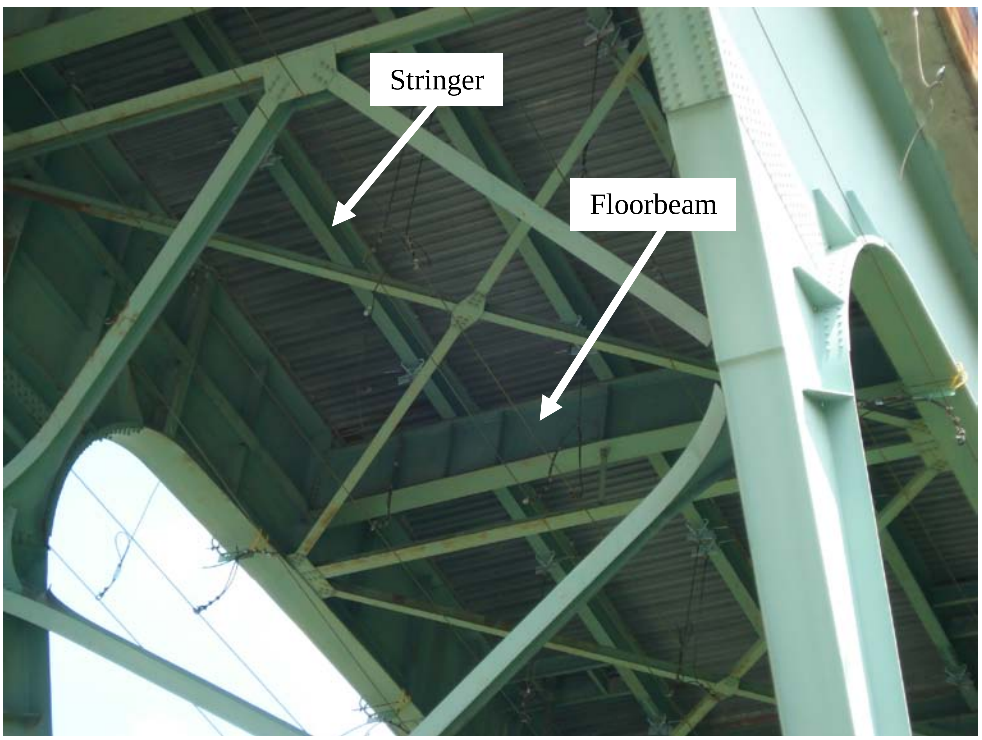


Figure 10.6.7 Two Frame Bridge with Floorbeam-Stringer Floor System

Primary and Secondary Members

For steel rigid frame bridges, the primary members are the frames as a whole, including floorbeams and stringers (if present) (see Figure 10.6.7). However, for ease of discussion, the frame is commonly broken down into the following five elements:

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

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

In a two frame system, lateral bracing members are placed diagonally between the horizontal members of the frames. This bracing restricts any horizontal differential and longitudinal movements between the frames. This bracing is in the plane of the bottom flange of the girder portion of the frame or between the legs of the frame (see Figure 10.6.8).

In a two frame system, sway bracing is placed between the leg portions of the frame (see Figure 10.6.8). In a multiple frame system diaphragms are placed perpendicular between the frames. The sway bracing and diaphragms minimize any transverse movements of the frames (see Figure 10.6.9).

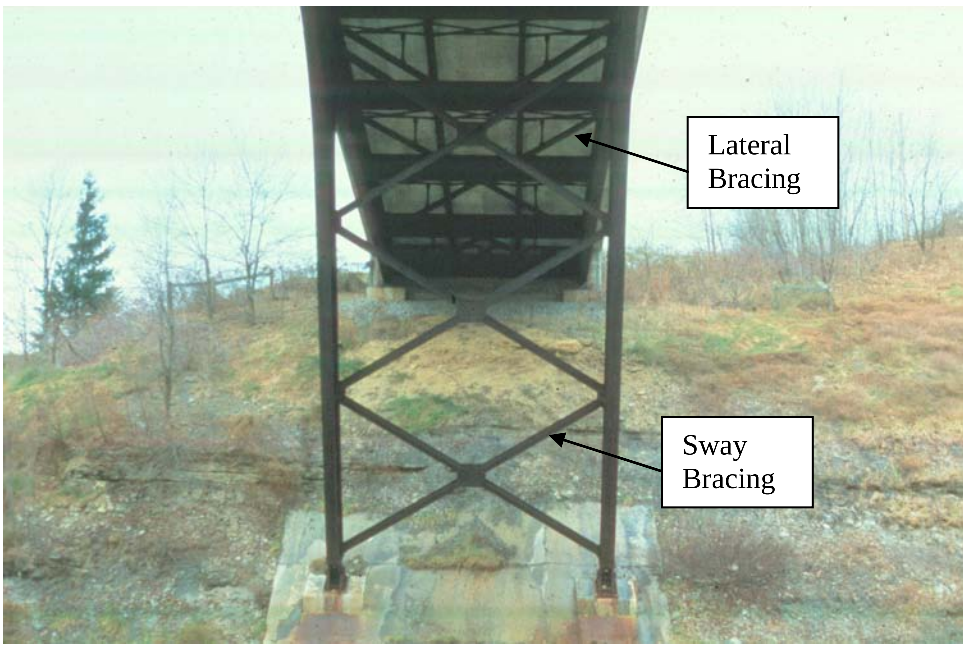


Figure 10.6.8 Lateral and Sway Bracing for the Frame Legs

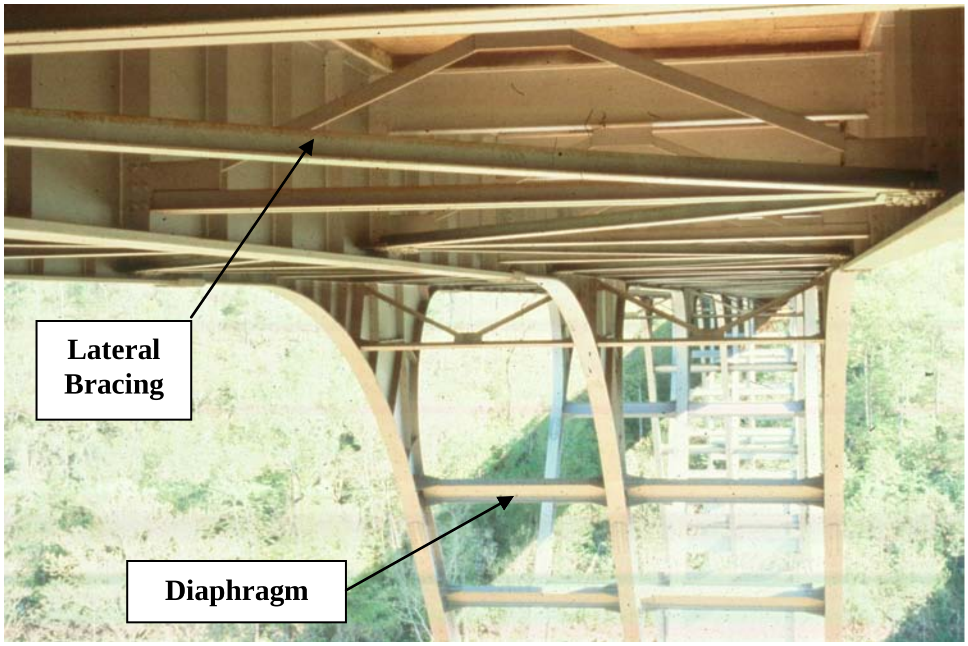


Figure 10.6.9 Lateral Bracing and Diaphragms

Stress Zones

Each element of the frame resists various levels of stress due to moment and shear. Tension zones are similar to those for concrete rigid frames (see Figure 10.6.10).

Stress zones for the floor systems are similar to the two-girder floor systems discussed in Topic 10.2.

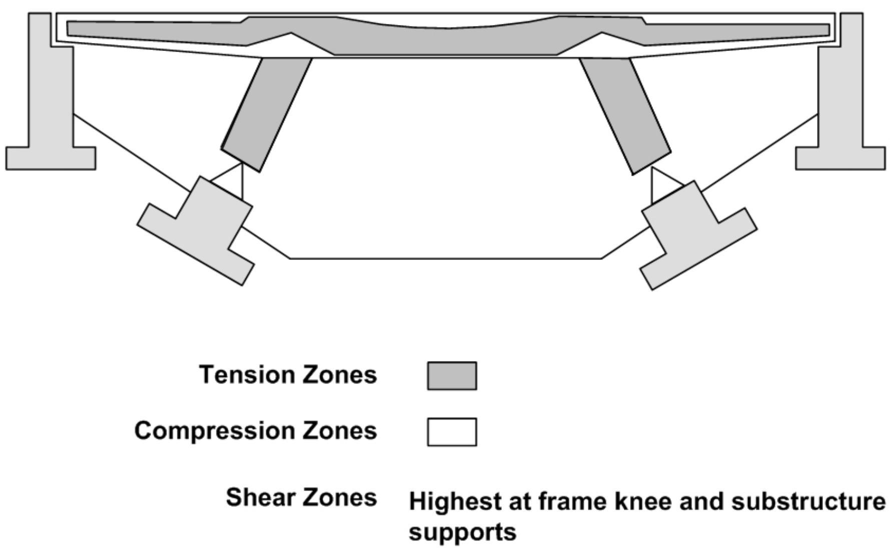


Figure 10.6.10 Stress Zones in a Frame

Fracture Critical Members

A rigid frame consisting of two frames has no load path redundancy. This means that a two frame steel rigid frame is considered a fracture critical bridge type (see Figure 10.6.11). Potential fracture critical members include portions of the frames in tension, as well as floorbeams (if applicable).

Figure 10.6.11 Fracture Critical Structure - No Load Path Redundancy

A rigid frame bridge consisting of three or more frames has load path redundancy and is not considered fracture critical (see Figure 10.6.12).

Figure 10.6.12 Multiple Frame Rigid Frame – Not a Fracture Critical Structure

10.6.3 Overview of Common Deficiencies

Common deficiencies that occur on steel rigid frame bridges include:

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

See Topic 6.3 for a detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel. Refer to Topic 6.4 for Fatigue and Fracture in Steel Bridges.

10.6.4 Inspection Methods and Locations

Inspection methods to determine other causes of steel deterioration are discussed in detail in Topic 6.3.

Methods

Visual

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

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

Physical

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

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

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

Advanced Inspection Methods

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

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

Other methods, described in Topic 15.3.3, include:

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

Locations

Bearing Areas

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

Examine the bearings at each support for corrosion. Check the alignment of each bearing and note any movement. Report any build up of debris surrounding the bearings that may limit the bearing from functioning properly. Check for any bearings that are frozen due to heavy corrosion (see figure 10.6.13). See Topic 11.1 for a detailed presentation on the inspection of bearings.

Figure 10.6.13 Bearing Area of a Two Frame Bridge

Shear Zones

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

Flexure Zones

Check the tension and compression flanges for corrosion, section loss, cracks or buckling. Give special attention to the flanges at the connection between the legs and girder portion of the beam. Bending moment is at its greatest in this area (see Figure 10.6.14).

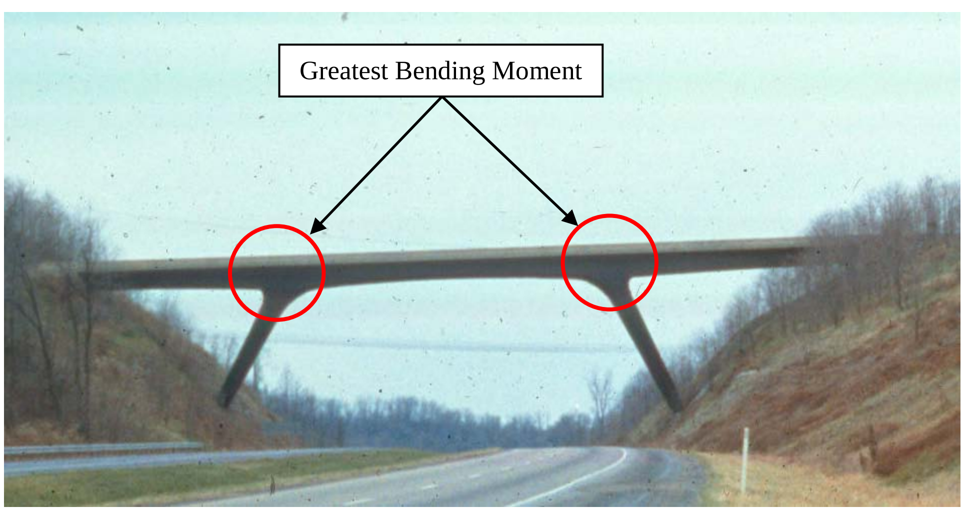


Figure 10.6.14 Flexural Zones (Greatest Bending Moment)

Gusset Plates

Gusset plates may be used to connect frame bracing members together. Thorough inspection is required for gusset plates, since these connections experience axial and shearing forces simultaneously.

For steel frames with gusset plates, give special attention to the following locations:

- Look for out-of-plane distortions particularly at areas susceptible to pack rust and along areas of unbraced length.
- Check for coating system failures, as these allow for advancement of corrosion and section loss.
- Check for corrosion and section loss, comparing current measurements to original drawings.
- Check for cracking around tack welds, bolt/rivet holes, and other welds.
- Examine the gusset plate for loose or broken rivets/bolts, which can be detected through hammer sounding.
- Closely examine gusset plate retrofits, particularly welded retrofits as they are very problematic.

Refer to Topic 10.8.4 for detailed information on inspection locations and methods for gusset plates.

Problematic Details

Problematic details checked for deficiencies and deterioration include:

- Triaxial constraint
- Intersecting welds
- Cover plates
- Cantilevered suspended spans
- Insert plates
- Field welds: patch and splice plates
- Intermittent welds
- Out-of-plane bending
- Pin-and-hanger assemblies
- Back-up bars
- Mechanical fasteners and tack welds

Note that out-of-plane bending may occur at the following areas, which are presented further in Topic 6.4.8:

- Girder web connections for diaphragms and floorbeams, including connections plates at the top flange, connection plates at the bottom flange, and connection plates at the bottom flange for skewed bridges
- Lateral bracing gussets and diaphragm connection plates

- Diaphragm connections to gusset plates
- Cantilevered floorbeams

In addition to the common problematic details, rigid frames also utilize the following:

- Stiffeners (transverse or longitudinal)
- Groove welded butt splices
- Lateral bracing gusset plates
- Web-to-flange welds
- Miscellaneous connections (railing or utilities)

See Topic 6.4.8 for additional information on problematic details.

Secondary Members

Check horizontal connection plates which can trap debris and moisture and are susceptible to a high degree of corrosion and deterioration. Investigate the areas beneath drainpipes and deck joints for corrosion from exposure to roadway drainage. Examine the connection areas of the lateral bracing or diaphragms for cracked welds, fatigue cracks, and loose fasteners. Check for distortion in the secondary members. Distorted secondary members may be an indication the primary members are overstressed or the substructure may be experiencing differential settlement.

Areas Exposed to Drainage

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

On rigid frame bridges check:

- Horizontal surfaces that include top of bottom flange
- Lateral bracing gusset plates
- Pockets created by floor system connections

Areas Exposed to Traffic

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

10.6.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

The element level method does not have specific elements for steel rigid frames. Due to this fact, individual states may choose to create their own elements or use the AASHTO National Bridge Elements (NBEs) or Bridge Management Elements (BMEs) that "best describe" the rigid frame. In an element level condition state assessment of a steel rigid frame bridge, possible AASHTO National Bridge Elements or Bridge Management Elements that relate closest to a rigid frame include:

NBE No.	Description
Superstructure	
107	Steel Girder/Beam
113	Steel Stringer
152	Steel Floor Beam
162	Steel Gusset Plate

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

The unit quantity for the girders/beams, stringers, and floorbeams is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for gusset plates (if applicable) is each, with each gusset plate element placed in one of the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coating is square feet, with the total area distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element*

Inspection for condition state descriptions. For gusset plates, see Topic 10.8.

For states that create their own Agency Developed Elements, use that particular state's bridge inspection manual to determine the appropriate element(s) as well as the correct condition state(s).

The following Defect Flags are applicable in the evaluation of steel rigid frames:

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

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

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Chapter 10
Inspection and Evaluation of Steel Superstructures

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Topic 10.7 Pin-and-Hanger Assemblies

10.7.1 Introduction

Pin-and-hanger assemblies are devices that utilize two pins with connecting hangers in bridges to permit longitudinal expansion movement and rotation (see Figure 10.7.1). If only rotation of the joint is desired, one pin is used (see Figure 10.7.2).

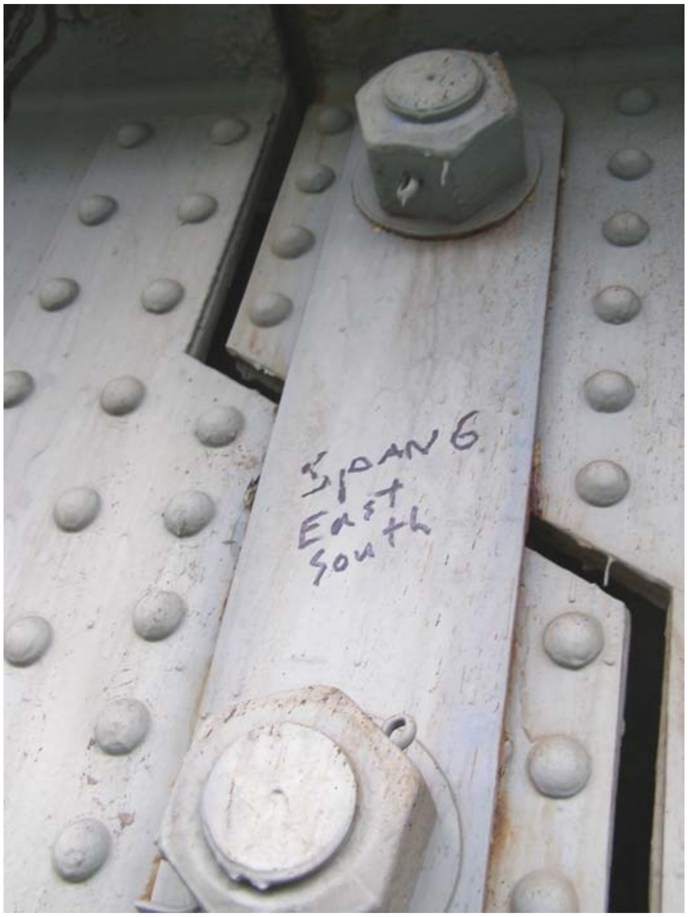

Figure 10.7.1 Typical Pin-and-Hanger Assembly

Pin-and-hanger joints are usually found in multi-span bridges designed prior to 1970. Incorporating a hinge in a structure simplifies analysis. It also moves expansion joints (and drainage related damage) away from the bearings, abutments and piers (see Figure 10.7.3).

Modern design techniques and computer programs enable the engineer to design multi-span bridges without hinges. The problems associated with pin-and-hanger details far outweigh the advantages of placing expansion joints away from substructure units.

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

Figure 10.7.2 Single Pin with Riveted Pin Plate

Figure 10.7.3 Pin-and-Hanger Assembly Locations Relative to Piers

10.7.2 Design Characteristics

Primary and Secondary Members

There are many different components to a pin-and-hanger assembly as Figure 10.7.4 demonstrates.

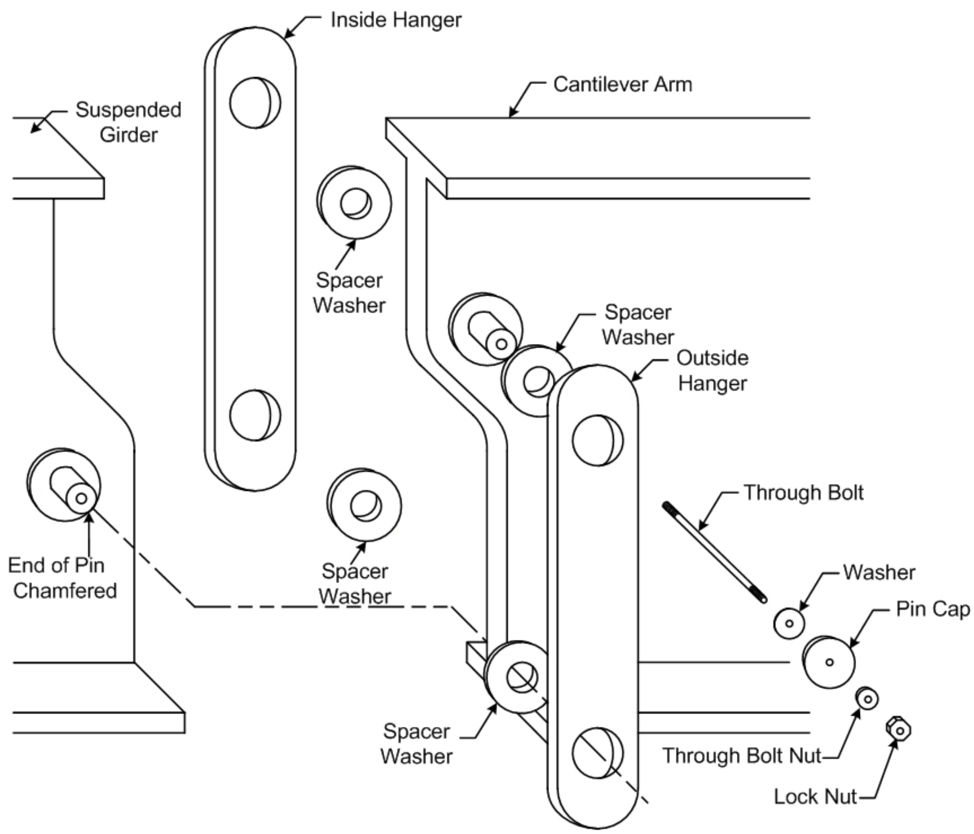

Figure 10.7.4 Pin-and-Hanger Assembly

The primary members of a pin-and-hanger assembly are the pin and the hanger link. The pin may be drilled to accept a through-bolt (see Figure 10.7.5) or threaded to accept a large nut (see Figure 10.7.6). Threaded pins are often stepped (or shouldered) to accept a small diameter nut. The hanger link may be a plain flat plate with two holes or an eyebar shaped plate (see Figure 10.7.7).

The secondary members of a pin-and-hanger assembly include through-bolts and the pin cap (see Figure 10.7.8), nuts (see Figure 10.7.9), cotter pins on small assemblies with pins less than 4 inches in diameter, spacer washers and doubler plates which reinforce the beam web around the pin hole (see Figure 10.7.10).

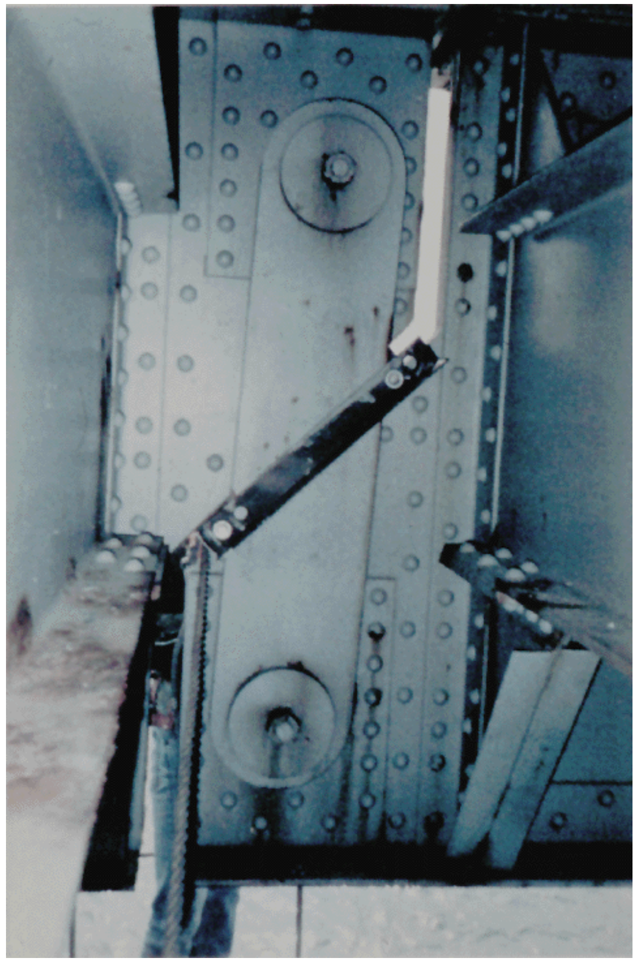

Figure 10.7.5 Pin Cap with Through Bolt

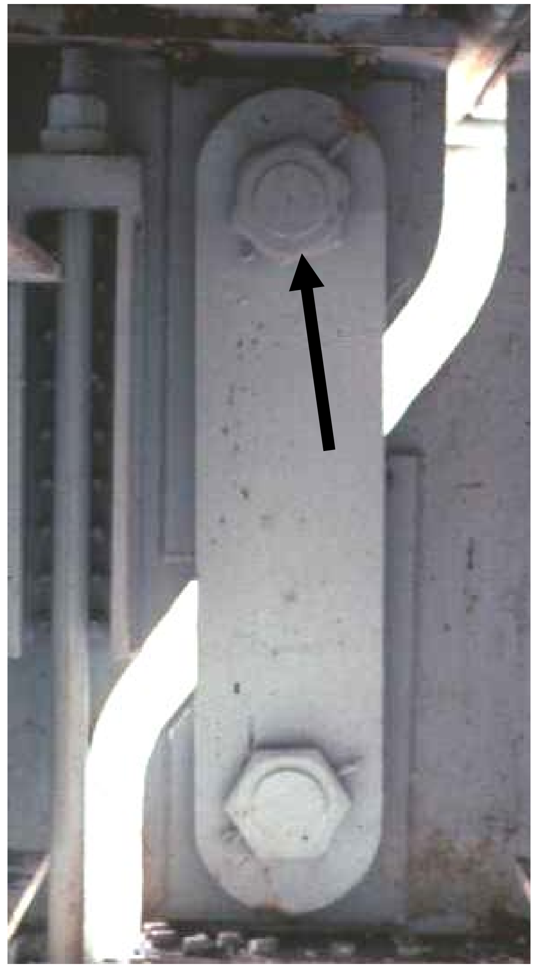

Figure 10.7.6 Threaded Pin with Retaining Nut

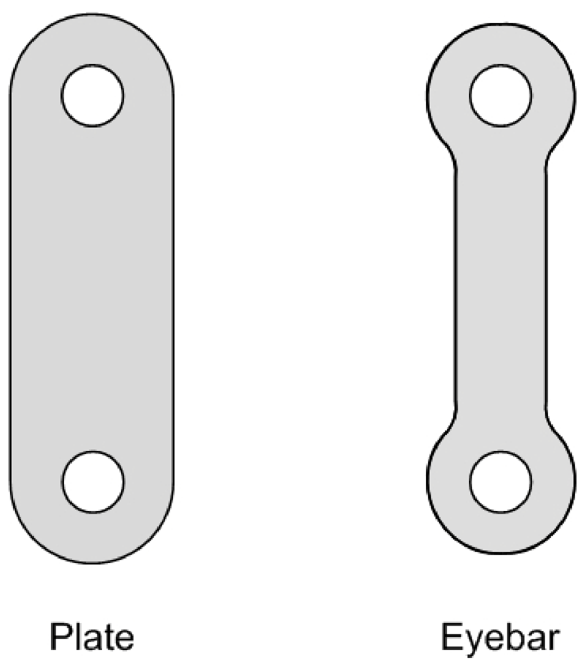

Figure 10.7.7 Plate Hanger and Eyebar Shape Hanger Link

Figure 10.7.8 Pin Cap, Through Bolt and Nut

Figure 10.7.9 Retaining Nut

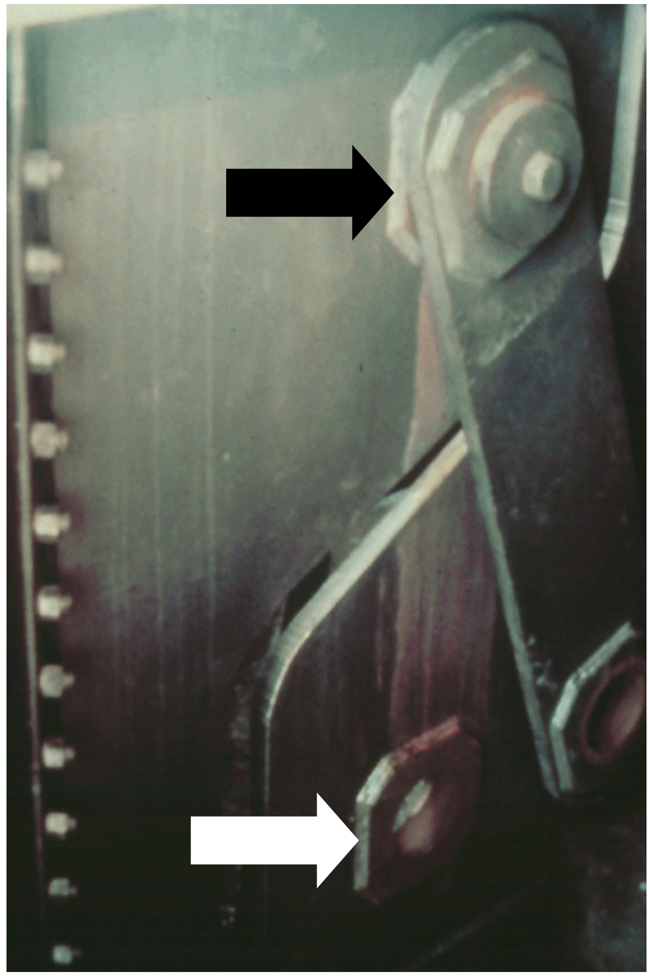

Figure 10.7.10 Web Doubler Plates

Forces in a Pin – Design vs. Actual

Some of the problems with the pin-and-hanger assembly can be attributed to deficiencies that cause forces that were not accounted for in the original design. The hanger or links are designed for pure tension forces only (see Figure 10.7.11). However, in actuality, hangers see both pure tension and bending. In-plane bending results from binding on the pins due to corrosion between the pin and the hanger (see Figure 10.7.12). Out-of-plane bending (perpendicular to the wide face) results from misalignment, pack rust, skewed geometry or improper erection.

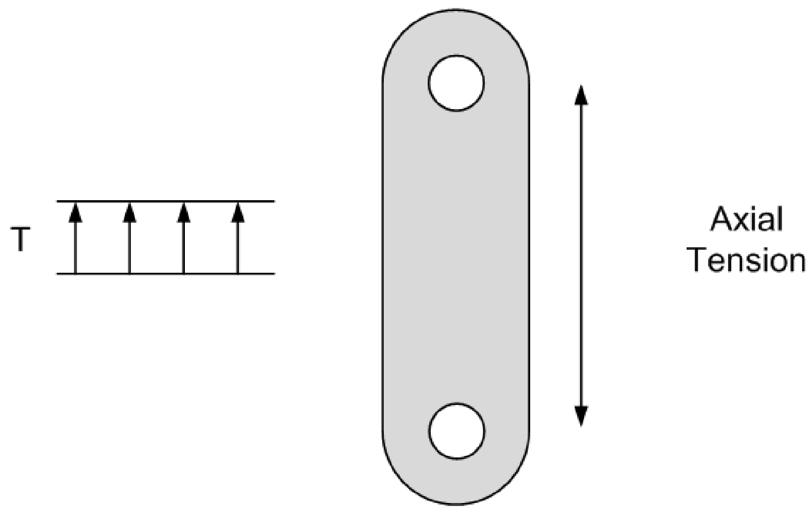

Figure 10.7.11 Design Stress in a Hanger Link(Tension Only)

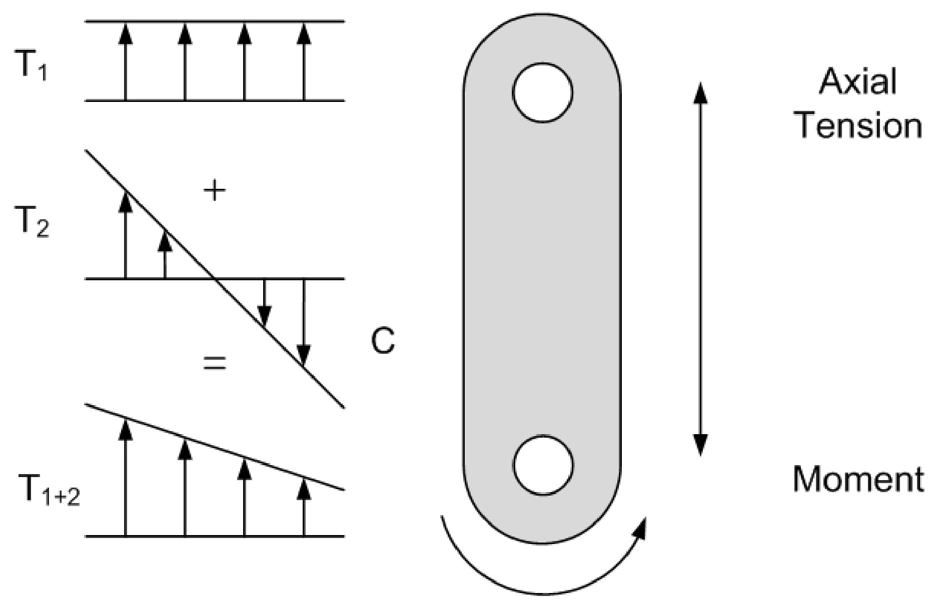

Figure 10.7.12 Actual Stress in a Hanger Link (Tension and Bending)

Pins are designed to resist shear and bearing on the full thickness of the hanger (see Figure 10.7.13). However, in addition to the designed forces, pins can see very high torsion (twisting) forces if they lose their ability to turn freely (see Figure 10.7.14). Section loss in the pin may cause a loss of bearing areas between the pin, the hangers and the web. This loss can cause unsysmetrial loading which results in possible out-of-plane bending in the web and hanger (see Figure 10.7.14).

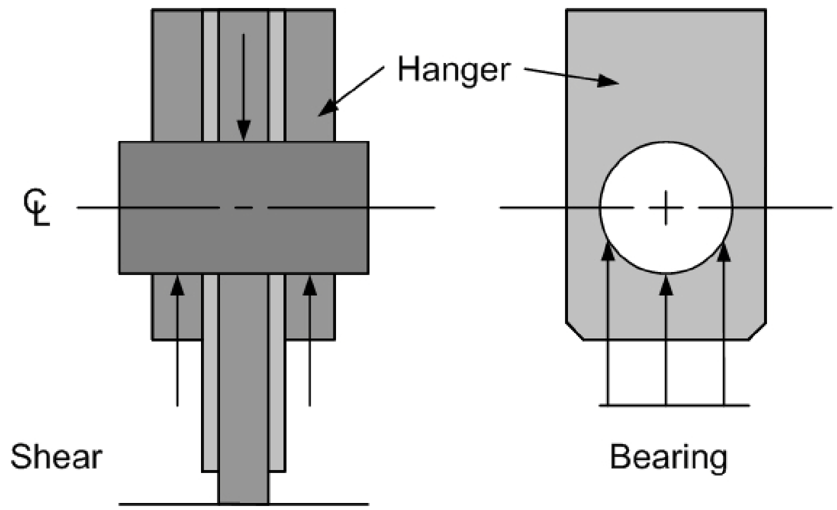

Figure 10.7.13 Design Stress in a Pin (Shear and Bearing)

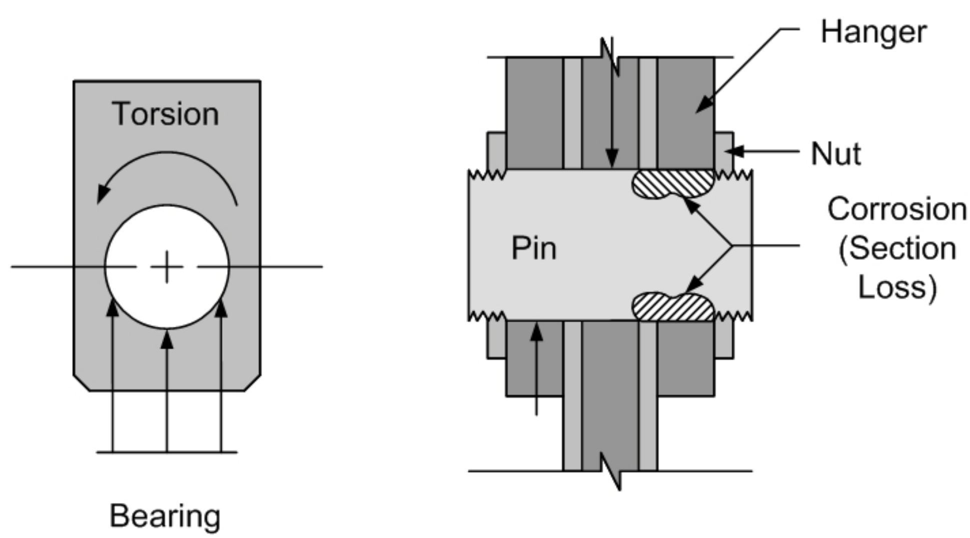

Figure 10.7.14 Actual Stress in a Pin (Shear, Bearing and Torsion)

Fracture Critical Pin-and-Hanger Assemblies

The *AASHTO Manual for Bridge Evaluation*, Section 4.8.3.11 states that when pin-and-hangers are present on trusses or two-girder systems, they are considered to be fracture critical. Therefore, it will be important to pay special attention during the inspection of pin-and-hanger connections when they are located in those types of superstructures. Failure of one pin or one hanger will cause collapse of the suspended span since there is no alternate load path. The collapse can be catastrophic as demonstrated by the Mianus River Bridge failure shown in Figure 10.7.15. The Mianus River Bridge failed due to the formation of rust between the hangers and the girder webs. As steel rusts, the rust can occupy up to 10 times the original steel volume causing unwanted expansion forces when in a confined space. When rust creates this type of expansion force, it is called “rust packing”. In the case of the Mianus River Bridge, the rust packing pushed the hangers to the ends of the deteriorated pins and the pins eventually failed in bearing. The failure may have been compounded by the heavily skewed geometry of the bridge that intensified the lateral force on the pin-and-hanger assembly.

Figure 10.7.15 Mianus River Bridge Failure

Pin-and-hanger assemblies in multi-girder structures are not technically fracture critical, since multiple load paths are available. However, they do have the potential for progressive collapse. If the pin-and-hanger assemblies at a joint location are frozen and consequently overstressed, the failure of one could cause an adjacent assembly to fail and so on (see Figure 10.7.16). Some bridge owners treat all pin-and-hanger assemblies as fracture critical, regardless of whether or not the girders they support are redundant.

Figure 10.7.16 Multi-girder Bridge with Pin-and-Hanger Assemblies

10.7.3 Overview of Common Deficiencies

Common deficiencies that occur on steel pin-and-hanger bridge assemblies are:

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

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

10.7.4 Inspection Methods and Locations

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

Methods

Visual

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

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

Physical

Removal of paint can be done using a wire brush, grinding, or sand blasting, depending on the size and location of the suspected deficiency. Exercise care when clearing suspected areas that are cracked. When clearing steel surfaces avoid any type of clearing process that would tend to close discontinuities such as blasting. The use of degreasing spray before and after removal of the paint may help in revealing the deficiency.

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, examine any other similar locations and details on the bridge.

Advanced Inspection Methods

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

- Acoustic emissions testing
- Corrosion sensors
- Smart coatings

- Dye penetrant
- Magnetic particle
- Radiographic testing
- Computer tomography
- Robotic inspection
- Ultrasonic testing
- Eddy current
- Electrochemical fatigue sensor (EFS)

Other methods, described in Topic 15.3.3, include:

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

Visual inspection of the pin may not be very effective. The majority of the pin is concealed inside the assembly and at best only the surface is available for inspection. Many internal flaws and defects can go undetected if an advanced inspection technique such as ultrasonic testing is not used.

Ultrasonic testing is currently the most common means available of checking pins in place (see Figure 10.7.17). For the results to be valid, careful planning and testing by trained, certified technicians is required. For a more detailed look at ultrasonic testing refer to Topic 15.3.

Figure 10.7.17 Ultrasonic Testing of a Pin

Another method for inspecting the pin is to disassemble the pin-and-hanger unit. Undertake the disassembly of a pin-and-hanger joint only after proper engineering design is performed and auxiliary support supplied. It is not a routine bridge inspection procedure (see Figure 10.7.18).

Figure 10.7.18 Alternate Hanger Link Retaining System

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

Locations

General

Observe and record the general condition of the pin-and-hanger assembly. Check for alignment of the adjacent beam webs and flanges with a straight edge. If present, inspect the wind lock for signs of excessive transverse movement. A wind lock consists of steel or neoprene members attached to both the suspended and cantilever bottom flanges. It restricts differential latitudinal movement between the cantilevered and suspended girders. Note if deck drainage is entering the assembly.

Measure the actual dimensions between the pins and also the distance from each pin to the end of the hanger assembly and compare these values to the as-built dimensions (see Figure 10.7.19).

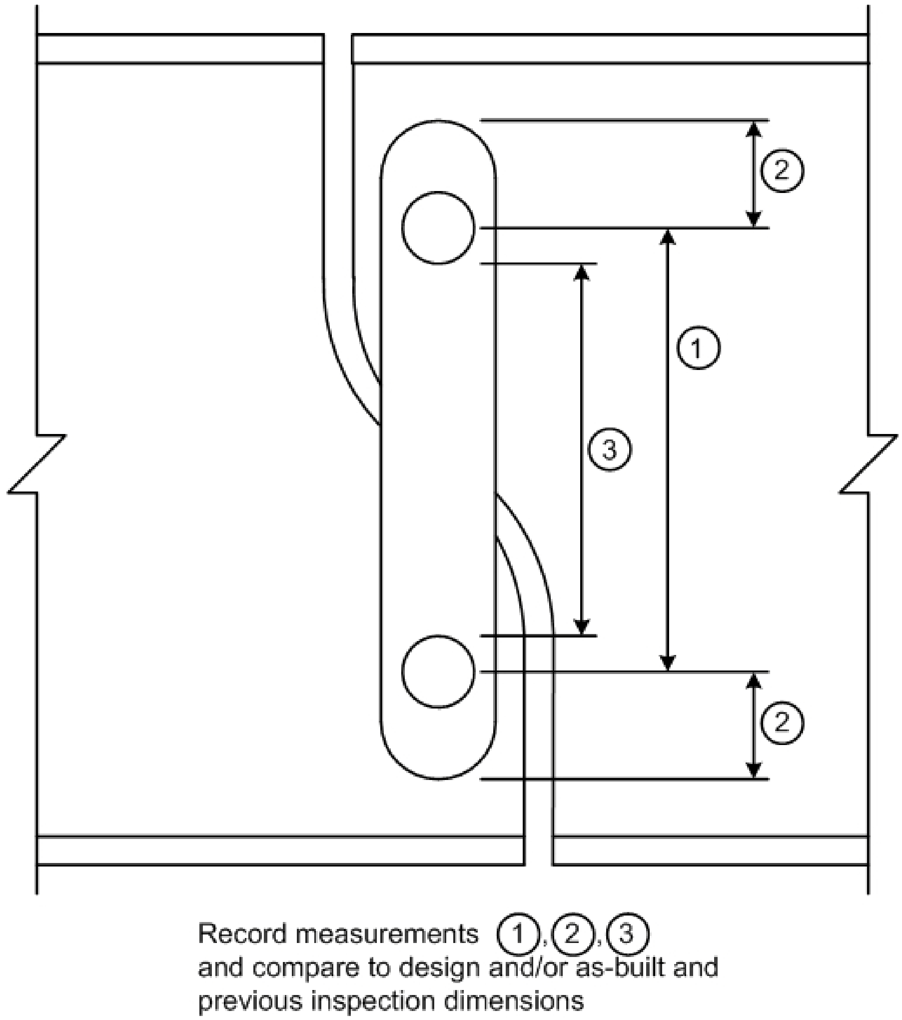

Figure 10.7.19 Pin Measurement Locations

Try to determine if movement is taking place. Corrosion can cause fixity at pin-and-hanger connections. This changes the structural behavior of the connection and is a source of cracking. Powdery red or black rust where surfaces rub indicates movement (see Figure 10.7.20). It may or may not indicate appreciable section loss. Where there is relative movement expected, an unbroken paint film across a surface indicates the pin is frozen.

Figure 10.7.20 Rust Stains from Pin Corrosion

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

Study the railing, expansion dam, beam ends, and any other structural components in the hinge area to see if any unusual displacements have taken place.

Hangers

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

Hangers are easier to inspect than pins since they are exposed and readily accessible. Try to determine whether the hanger-pin connection is frozen, as this can induce large bending moments in the hanger plates.

Examine accessible surface and edges closely for cracks (see Figure 10.7.21). The most critical areas are the ends beyond the pin centerlines and the juncture between the heads and shanks of eyebars. Note surface condition and section loss.

Figure 10.7.21 Corroded Hanger Plate

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

Examine both sides of the plate for cracks due to bending of the plate from a frozen pin connection. Observe the amount of corrosion buildup between the webs of the girders and the back faces of the plates. Inspect the hanger plate for bowing or out-of-plane distortion from the webs of the girders (see Figure 10.7.22). Investigate any welds for cracks. If the plate is bowed, check carefully at the point of maximum bow for cracks that might be indicated by a broken paint film and corrosion.

Measure the distance between the back of the hanger and the face of the web at several locations. Compare these measurements from location to location and hanger to hanger. Variations greater than 1/8 inch could indicate twisting of the hanger bars or lateral movement due to rust packing. Carefully describe and record these measurements in permanent notes for comparison with as-built drawings and/or measurements taken at the next inspection.

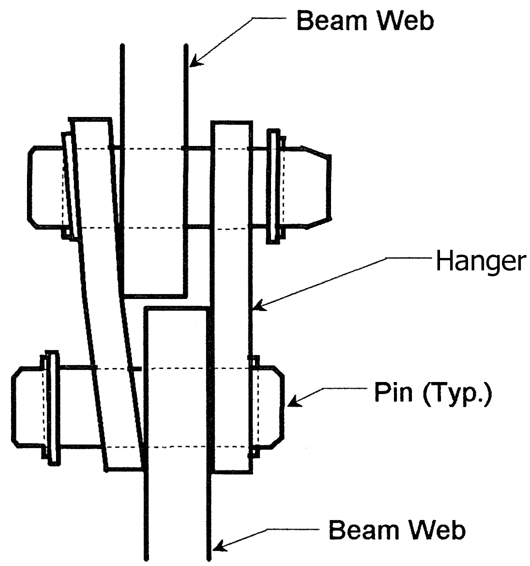

Figure 10.7.22 Bowing Due to Out of Plane Distortion of Hanger

Problematic Areas

The critical areas most likely to develop cracks are outlined below and shown in Figure 10.7.23:

- At welds used to connect hanger plates
- At welds used to connect web doubler plates
- In the base metal at the ends of hangers adjacent to pin holes
- In the base metal at the juncture between heads and shanks of eyebars

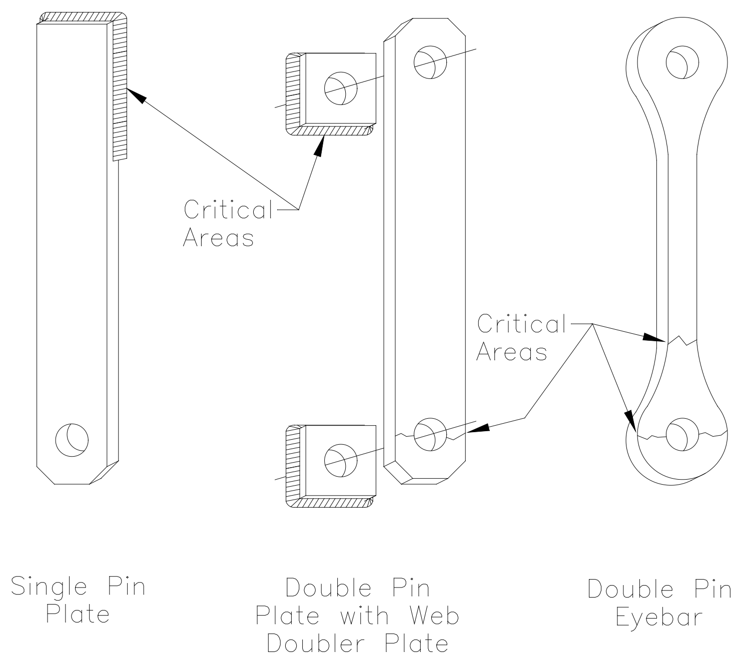

Figure 10.7.23 Fatigue Cracks in Pin-and-Hanger Assemblies

Pins

Rarely is the pin directly exposed in a pin-and-hanger assembly. As a result, its inspection is difficult but not impossible. By carefully taking certain measurements, the apparent wear can be determined. If more than 1/8 inch net section loss of the diameter has occurred, bring it to the attention of the bridge engineer at once (see Figure 10.7.24). Wear to the pins and hangers will generally occur in two locations: at the top of the pin and top of the hanger on the cantilevered span and at the bottom of the pin and the bottom of the hanger on the suspended span. Sometimes wear, loss of section, or lateral slippage may be indicated by misalignment of the deck expansion joints or surface over the hanger connection. When inspecting a pin-and-hanger assembly, locate the center of the pin, measure the distance between the center of the pin and the end of the hanger, and compare to the plan dimensions, if available. Remember to allow for any tolerances since the pin was not machined to fit the hole exactly. Generally, this tolerance will be 1/32 inch. If plans are not available, compare to previous measurements. The reduction in this length will be the apparent wear on the pin.

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

Check the pin cap, if part of the assembly, with a straight edge for flatness.

Figure 10.7.24 Corroded Pin-and-Hanger Assembly

Retrofits

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

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

The first two (rod & saddle and underslung catcher), are added to the structure and only carry load if the pin or hanger fails (see Figure 10.7.25). The gap between the "catcher" and the girder must be kept as small as possible to limit impact loading. If it is too tight, however, joint movement may be restrained. A neoprene bearing may be included in the assembly to lessen impact. Find out the relative design positions of the components and measure the critical points in the field for comparison.

The seated beam connection completely replaces the pin-and-hanger assemblies. Vacant pin holes may be left under some schemes. Inspection of these details will be the same as inspection at intersecting stiffeners and bearings.

Sometimes a pin-and-hanger assembly is retrofitted by using a bolted field splice. This is done only after a structural engineer analyzes the bridge to determine if the members can support continuous spans instead of cantilevered spans. Remember to inspect both the positive and negative moment regions of the superstructure. Additional deflections may be introduced into piers and more movements may take place at expansion bearings when continuity is introduced. Pay extra attention to these areas.

Figure 10.7.25 Underslung Catcher Retrofit

Replacing the pin-and-hanger assembly in kind with a structural grade of stainless steel eliminates potential failures due to corrosion related problems (see Figure 10.7.26). Placing a non-metallic insert and washer prevents corrosion between the pin and hanger and allows for normal rotation.

Figure 10.7.26 Stainless Steel Pin-and-Hanger Assembly

10.7.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

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

Element Level Condition State Assessment

In an element level condition state assessment of a steel girder bridge with a pin-and-hanger assembly, possible AASHTO National Bridge Element (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Superstructure	
161	Steel Pin, Pin-and-Hanger Assembly, or both

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

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

The following Defect Flags are applicable in the evaluation of pin-and-hanger assemblies:

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

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

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Inspection and Evaluation of Steel Superstructures

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Topic 10.8 Gusset Plates

10.8.1 Introduction

Gusset plates are used to connect multiple superstructure members together. They may connect primary load-carrying members, namely truss and arch members (see Figures 10.8.1 and 10.8.2), or secondary (bracing) members. Gusset plates are constructed from steel plates, which may be arranged in pairs or as a single plate, and are fastened to the members through riveting, bolting, welding or a combination of these methods (see Figures 10.8.3 through 10.8.5). Gusset plates are considered fracture critical members themselves when they connect one or more fracture critical members.

Although typically used to connect steel truss or arch superstructure members, gusset plates may also be used to connect timber truss or arch superstructure members, which includes using gusset plates for timber superstructure repairs and retrofits (see Figure 10.8.6).

Figure 10.8.1 Steel Truss Superstructure with Gusset Plates

Figure 10.8.2 Steel Deck Arch Superstructure with Stiffening Truss and Gusset Plates

Figure 10.8.3 Steel Gusset Plate with Riveted Connections

Figure 10.8.4 Steel Gusset Plate with Welded Connections

Figure 10.8.5 Steel Gusset Plate with Riveted, Bolted and Welded Connections

Figure 10.8.6 Steel Gusset Plates Connecting Timber Primary Truss Members

10.8.2 Design Characteristics

Gusset plates may connect primary load-carrying members or secondary (bracing) members.

Gusset Plates Connecting Primary Members

Gusset plates are often the principal means of connecting primary load-carrying members together at panel points for truss and arch superstructures. Gusset plates used for these applications may connect two to more members together (see Figure 10.8.7); though connections between three to five members are most common (see Figure 10.8.8). Types of primary load-carrying members connected with gusset plates include the following:

- Truss top chords
- Truss bottom chords
- Truss web members (vertical and diagonal members)
- Arch members (main arch members and tie members)
- Arch struts or vertical members

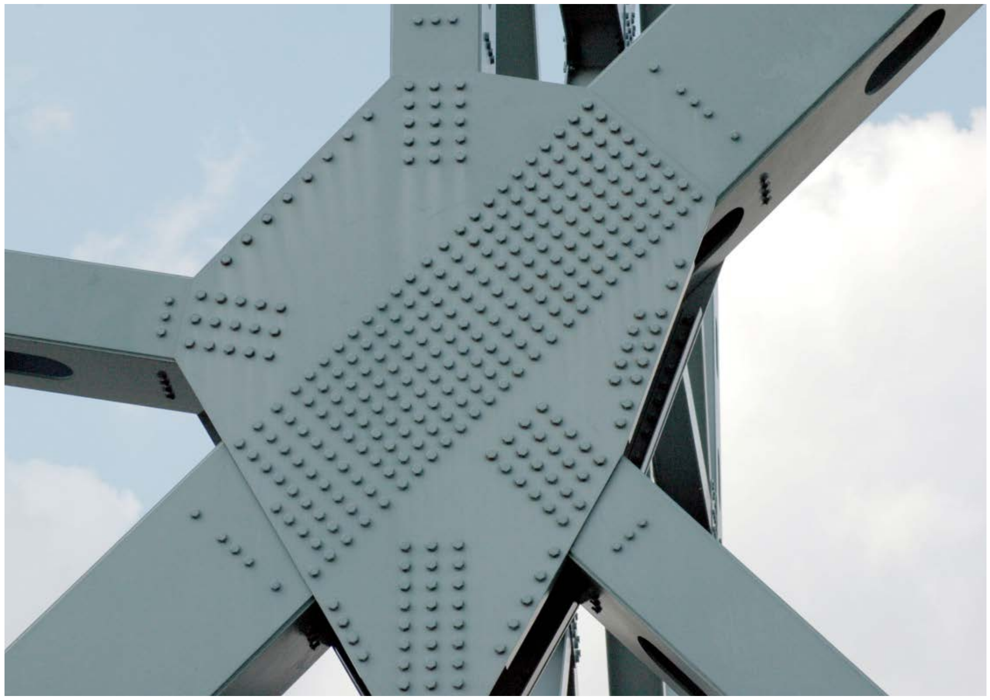

Figure 10.8.7 Odd-Shaped Gusset Plate Connecting Primary Load-Carrying Truss Members

Figure 10.8.8 Gusset Plate Connecting Primary Load-Carrying Truss Members

Gusset plates connecting primary load-carrying members are responsible for resisting various combinations of forces and stresses. Since gusset plates provide a connection between two or more members, the internal forces developed within the gusset plates may be extremely complex. For this reason, the inspection of gusset plates is a very careful and detail-oriented procedure. In an effort to better understand the potential failure modes of gusset plates, it is important that bridge inspectors familiarize themselves with the gusset plate design criteria.

Gusset plate design is broken down into two different categories: resistance of fasteners and resistance of gusset plates.

Resistance of Fasteners

Gusset plate fasteners are generally either bolts or rivets, though a combination of the two may be observed in the field. The bolts and rivets in gusset connections are evaluated to prevent shearing and plate bearing failures. The strength of bolts is typically found on the design plans. Because gusset plates incorporating riveted connections were often constructed before construction records were maintained, the information regarding the strength of the rivets used in construction may not be available. Many Bridge Owners specify guidance for the strength of rivets according to the year of construction.

Resistance of Gusset Plates

Gusset plates resist tension, shear, compression, and combined flexural and axial loads.

Gusset Plates in Tension

Gusset plates subjected to axial tension are investigated for three conditions:

- Yield on the gross section - The resistance of the gusset plate is calculated using the gross cross-sectional area of the member and yield strength of the gusset plate.
- Fracture on the net section - The resistance of the gusset plate is calculated using the net cross-sectional area (accounting for bolt holes and the effective width of the gusset plate) and the tensile strength of the plate.
- Block shear rupture - The resistance of the gusset plate is calculated from the combined resistance of parallel and perpendicular planes; one in axial tension and the others in shear. Examples of potential block shear rupture planes for gusset planes in tension are illustrated in Figure 10.8.9.

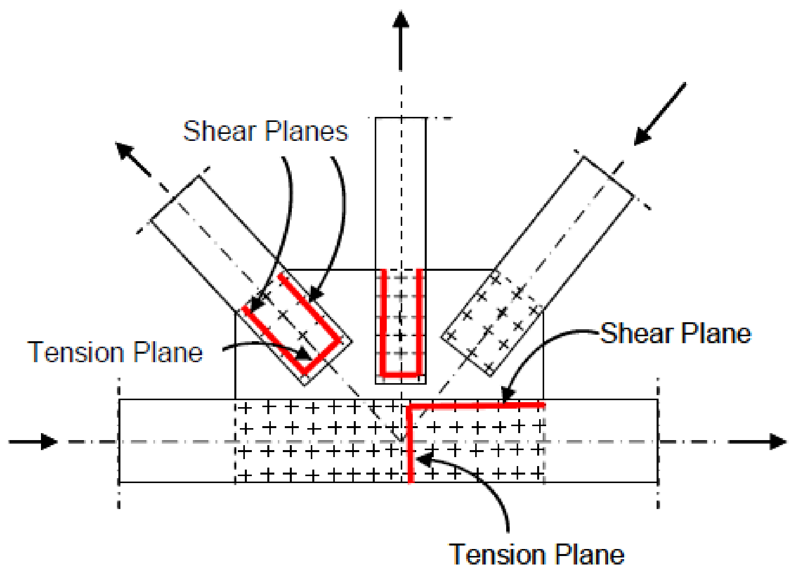

Figure 10.8.9 Potential Block Shear Rupture Planes for Gusset Plates in Tension

Gusset Plates in Shear

Gusset plates subject to shear are investigated for two conditions:

- Gross section shear yielding - The shear resistance of the gusset plate is calculated using the gross cross-sectional area of the member, yield strength of the gusset plate, and the shear resistance. Examples of gross section shear yielding planes are illustrated in Figure 10.8.10.
- Net section shear fracture - The shear resistance of the gusset plate is calculated using the net cross-sectional area, tensile strength of the plate, and the shear resistance. Examples of net section shear yielding planes are illustrated in Figure 10.8.11.

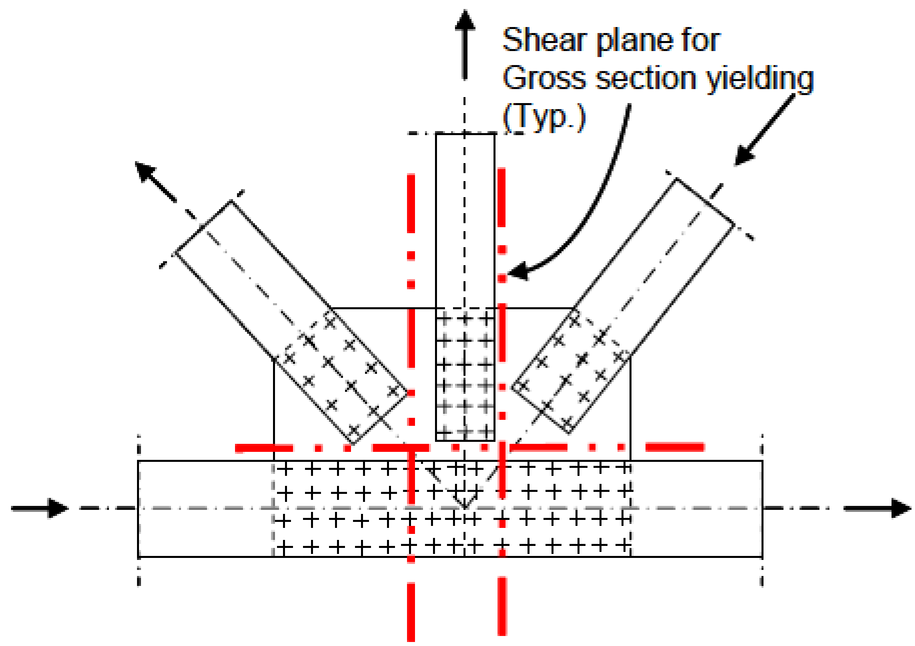

Figure 10.8.10 Examples of Gross Section Shear Yielding Planes

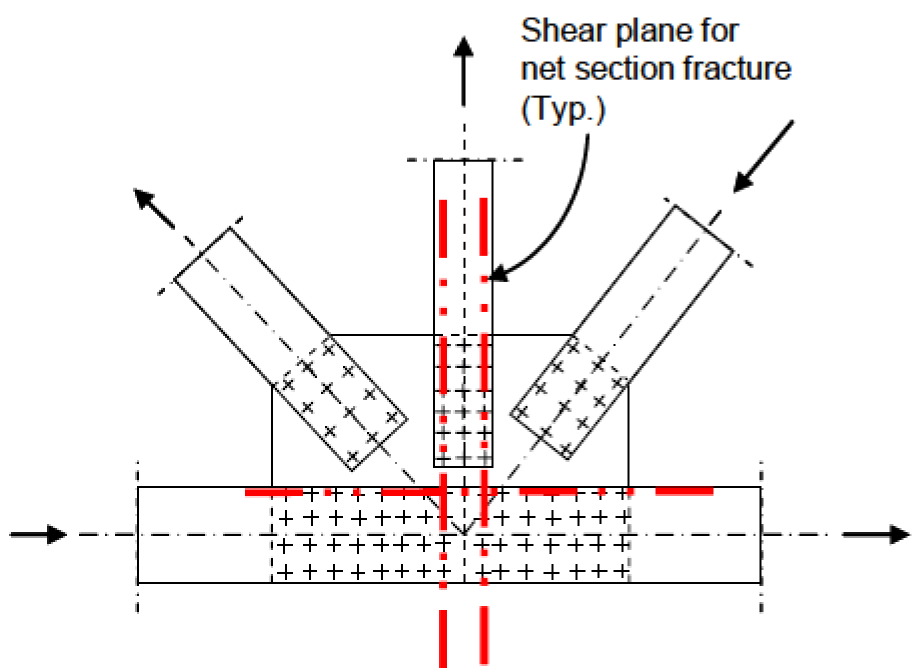

Figure 10.8.11 Examples of Net Section Shear Fracture Planes

Gusset Plates in Compression

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

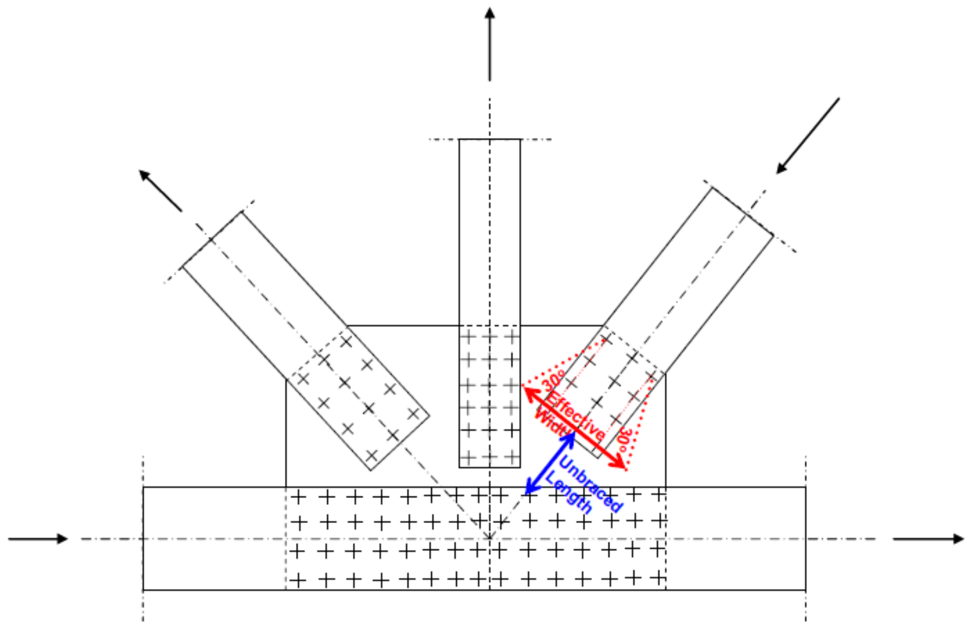

Figure 10.8.12 Example Showing the Unbraced Length and Effective Width for a Gusset Plate in Compression

Gusset Plates under Combined Flexural and Axial Loads

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

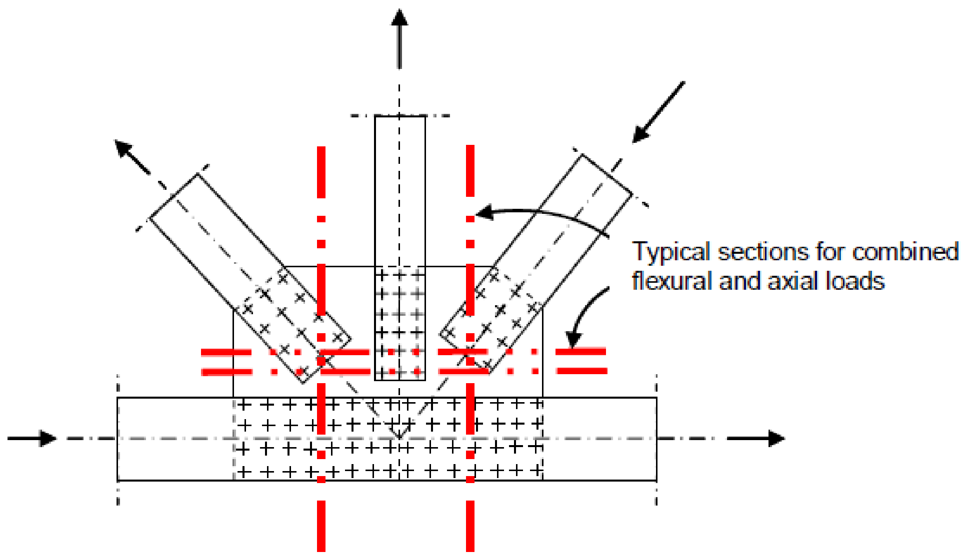

Figure 10.8.13 Examples of Combined Flexural and Axial Load Planes

Gusset Plates Connecting Secondary Members

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

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

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

10.8.3 Overview of Common Deficiencies

Common deficiencies that occur on steel gusset plates include:

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

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

10.8.4 Inspection Methods and Locations

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

Methods

Visual

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

Physical

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

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss and the original drawings.

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

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, examine any other similar locations and details.

Advanced Inspection Methods

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

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

Other methods, described in Topic 15.3.3, include:

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

Locations

General

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

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

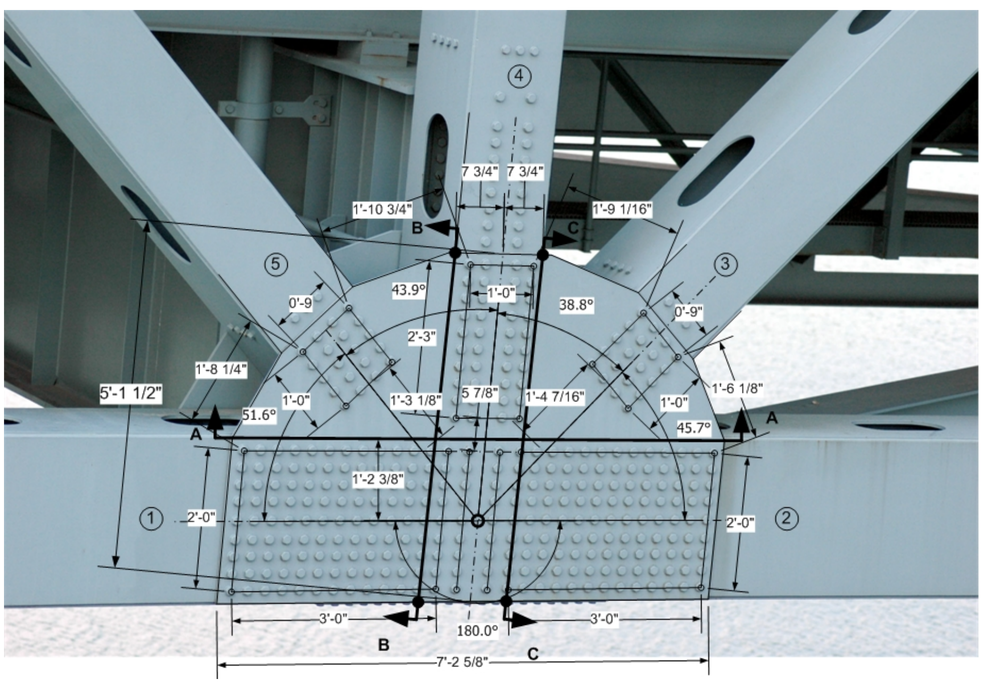

Figure 10.8.16 Gusset Plate Field Measurements

Areas with Corrosion

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

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

Figure 10.8.17 General Corrosion of Gusset Plates

Areas with Section Loss

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

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

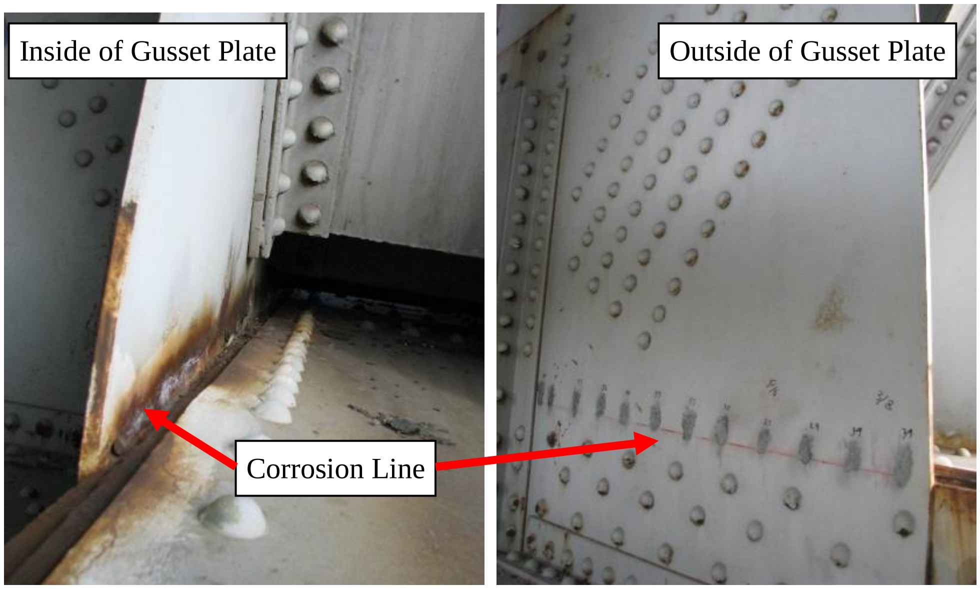

Figure 10.8.18 Corrosion Line Viewed from Inside and Outside of Gusset Plate

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

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

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

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

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

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

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

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

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

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

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

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

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

Figure 10.8.23 Inspector Using a Portable Ultrasonic Testing Inspection System

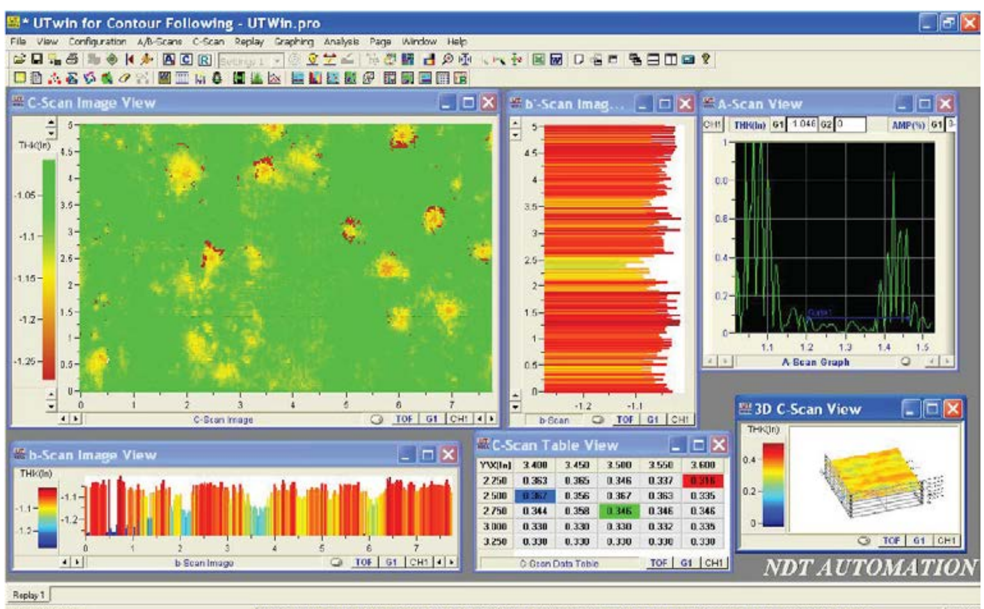

Figure 10.8.24 Ultrasonic Testing Inspection Acquisition Software

Areas Susceptible to Fatigue Cracking

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

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

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

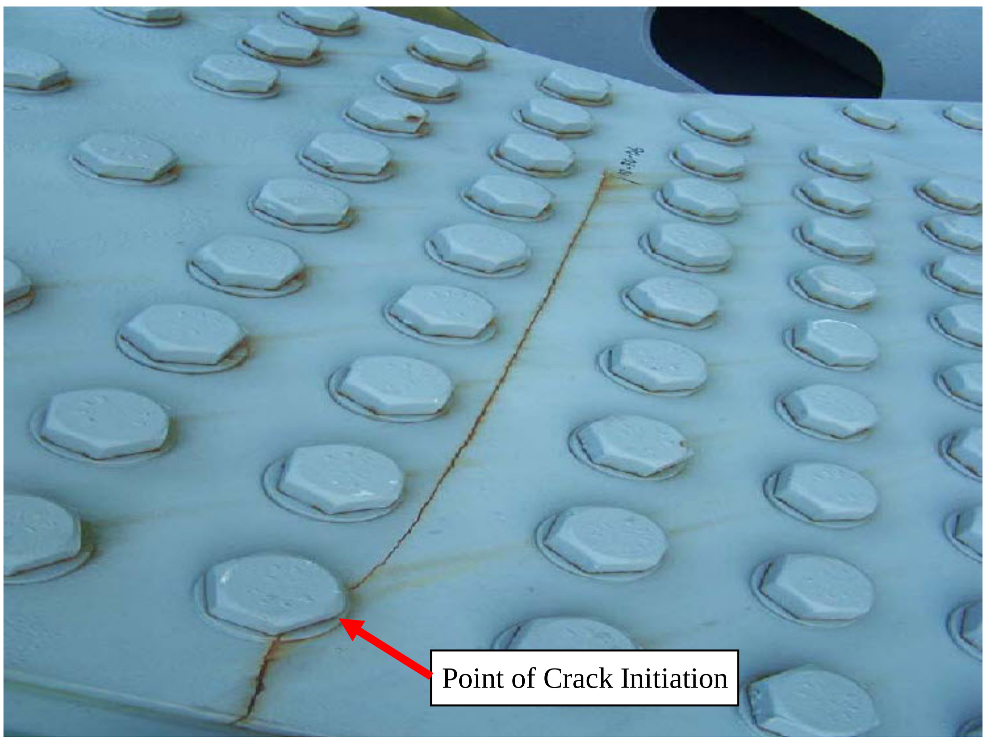

Figure 10.8.25 Cracked Gusset Plate and Point of Crack Initiation

Areas with Tack Welds

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

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

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

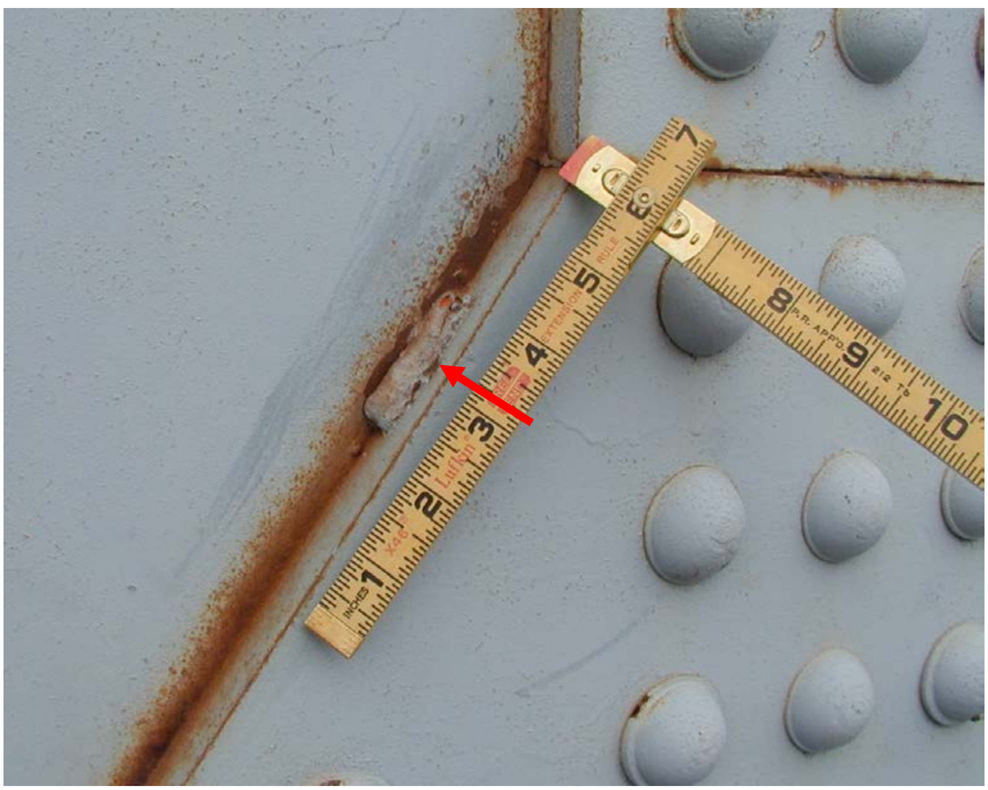

Figure 10.8.26 Partial Length Cracked Tack Weld

Areas Subject to Overstress

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

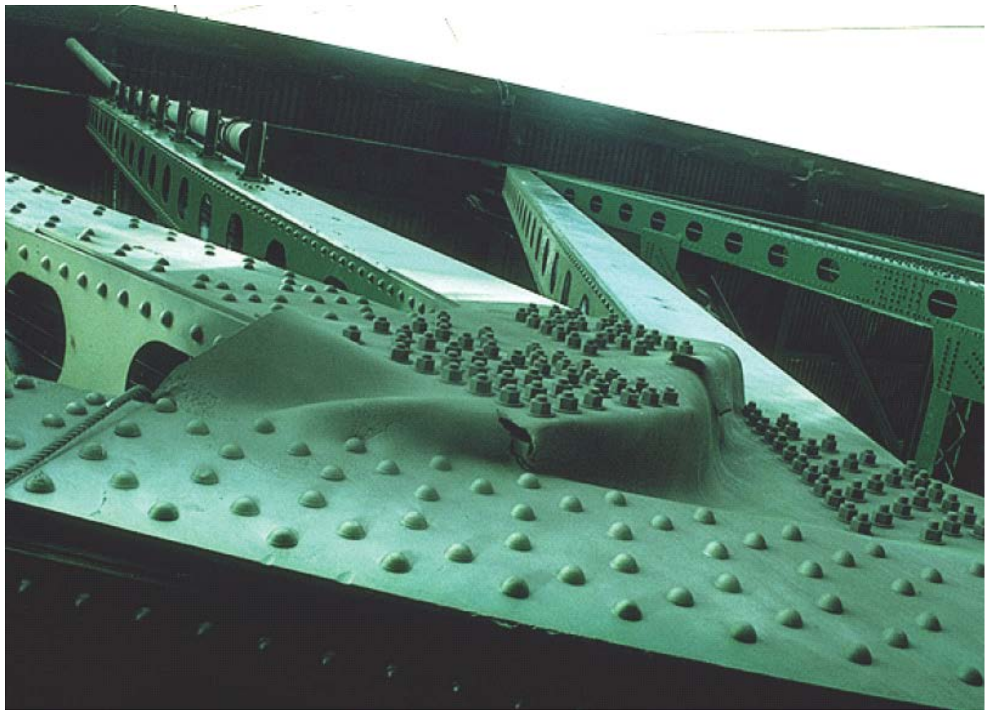

Figure 10.8.27 Gusset Plate Buckling (Compression) Failure due to Major Gusset Plate Section Loss

Areas with Paint Failure

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

Protective systems for gusset plates include:

- Protective coating
- Galvanizing
- Weathering steel

Figure 10.8.28 Gusset Plate with Paint Failure

Loose, Missing or Deteriorated Fasteners

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

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

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

Figure 10.8.29 Missing Bolts on Gusset Plate

Areas with Repairs and Retrofits

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

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

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

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

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

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

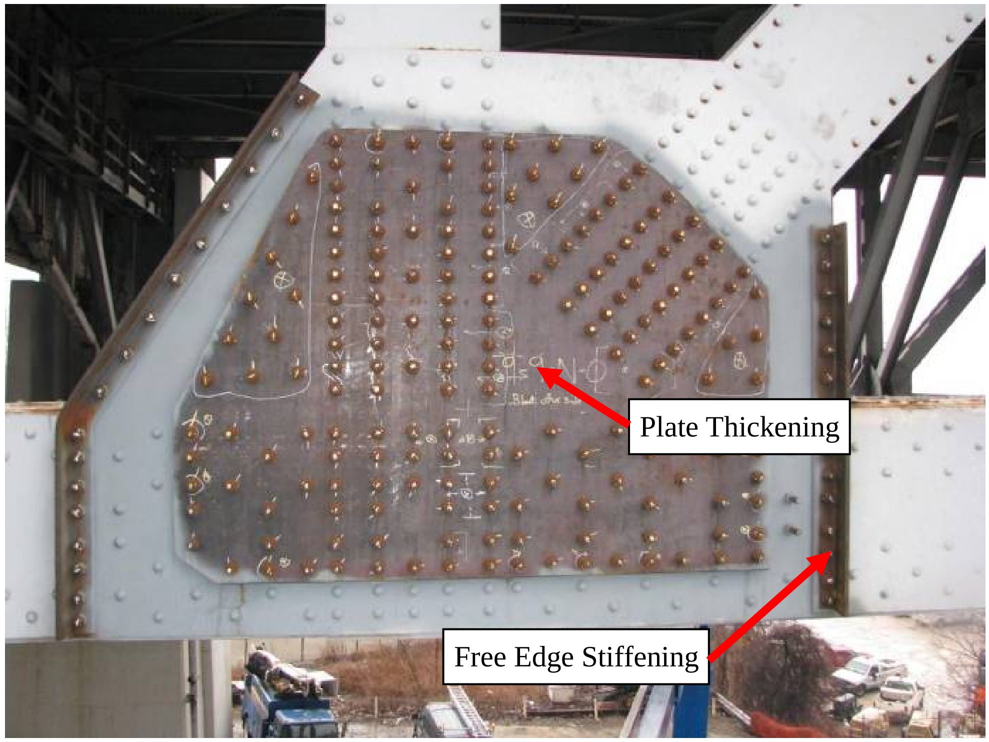

Figure 10.8.30 Plate Thickening and Free Edge Stiffening on Gusset Plate

Figure 10.8.31 Poorly Designed Welded Retrofit

Areas with Out-of-Plane Distortion

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

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

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

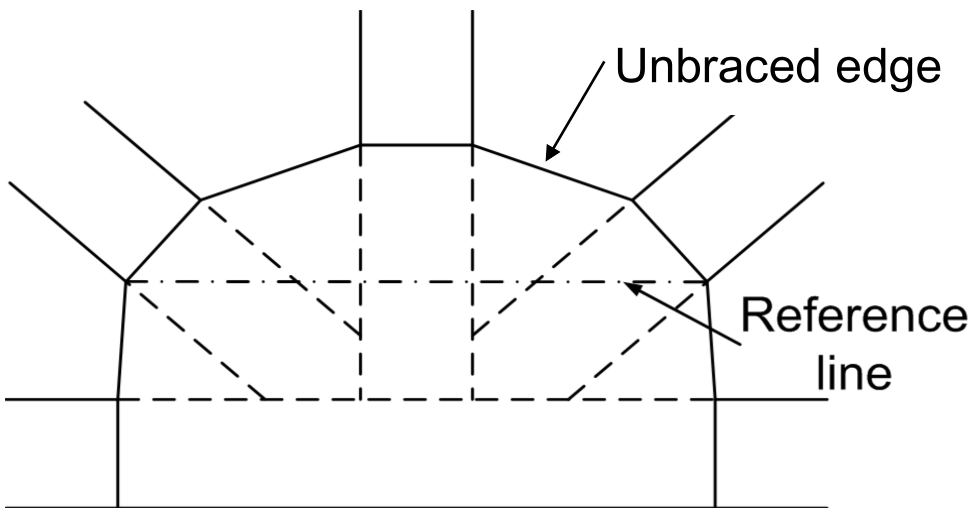

Figure 10.8.32 Unbraced Gusset Plate Edges and Reference Line

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

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

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

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

10.8.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

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

Element Level Condition State Assessment

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

NBE No.	Description
Superstructure	
162	Gusset Plate

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

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

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

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

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

10.8.6 Reasons to Inspect

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

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

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

Figure 10.8.34 Collapsed I-35W Mississippi River Bridge

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Inspection and Evaluation of Steel Superstructures

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Topic 10.9 Steel Eyebars

10.9.1 Introduction

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

Figure 10.9.1 Typical Eyebar Tension Member on an Arch

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

Figure 10.9.3 Eyebar Chain Suspension Bridge

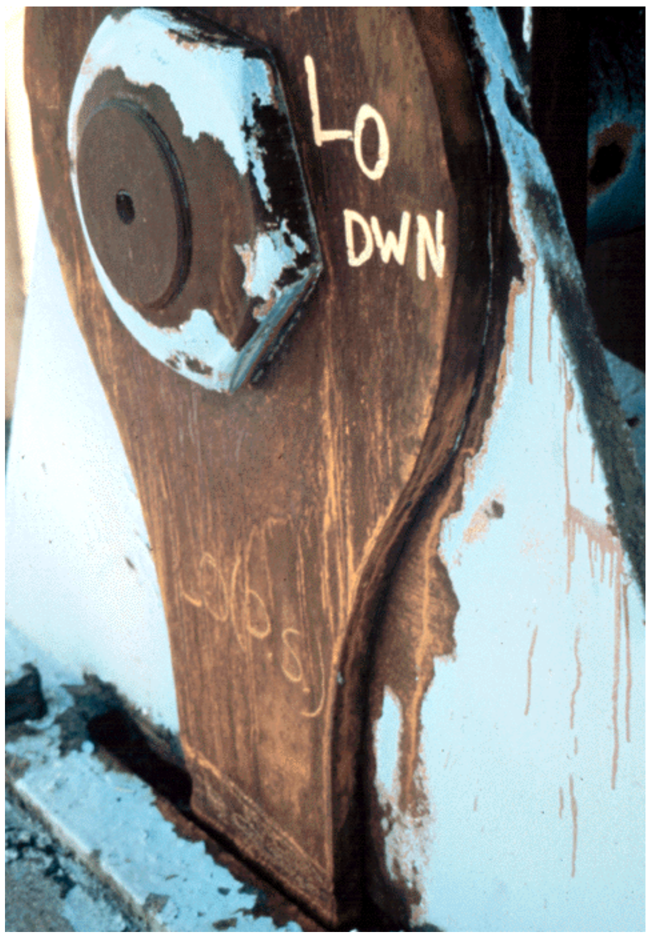

Figure 10.9.4 Anchorage Eyebar

Heat treated steel eyebars have been used in bridges around the world. One of these eyebars failed on December 15, 1967, sending the Point Pleasant Bridge (Silver Bridge), built in 1928, into the Ohio River between Point Pleasant, West Virginia and Kanauga, Ohio (see Figure 10.9.5). Forty-six people died and nine were injured due to the fracture of an eyebar in the north suspension chain on the Ohio side.

Figure 10.9.5 Collapsed Silver Bridge

Since the collapse of the Silver Bridge, there has been considerable public and professional concern over the safety of existing bridges, especially those containing eyebars. Many of these structures have been inspected and analyzed (see Figure 10.9.6). As a result, costly structural modifications and retrofits were made to many of these bridges (see Figure 10.9.7), while some others have been demolished. Eyebars are rarely used in new bridge designs but are present on many existing bridges.

Figure 10.9.6 Inspection of Eyebars

Figure 10.9.7 Retrofit of Eyebars to Add Redundancy

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

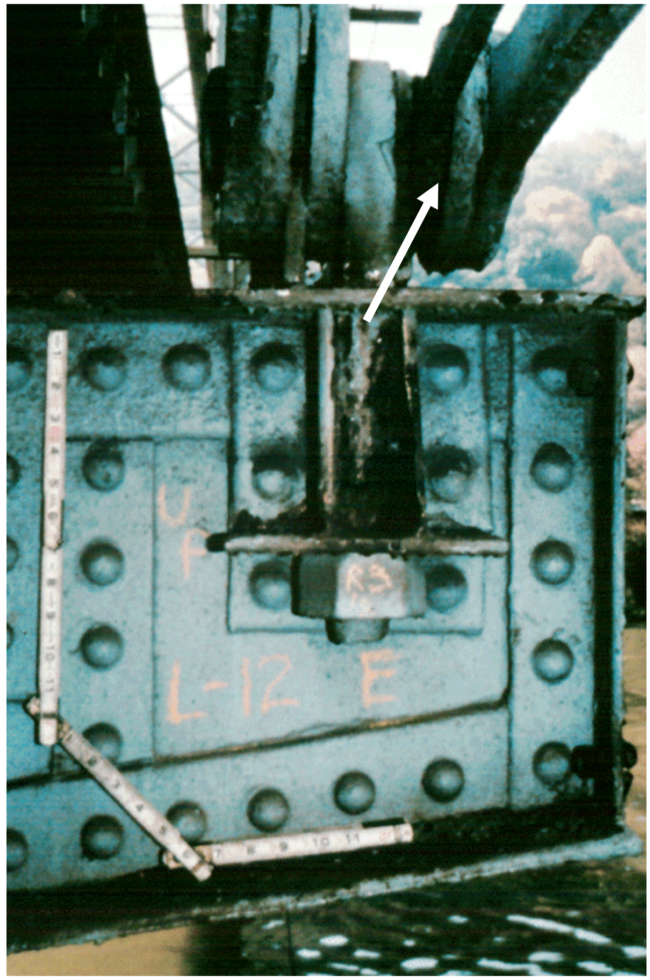

Figure 10.9.8 Eyebar Connection with Corrosion

10.9.2 Design Characteristics

Development of Steel Eyebars

In the late 1800's and early 1900's bridge spans began to increase in length, providing a need for higher strength steel. Prior to this time eyebars were made of wrought iron. The Eads Bridge in St. Louis, completed in 1874, was the first major steel bridge in America and the first in the world to use alloy steel (see Figure 10.9.9).

Figure 10.9.9 Eads Bridge, St. Louis

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

Around 1915, mild grade heat-treated steel eyebars were developed with a yield point of 50,000 psi and an ultimate strength of 80,000 psi. This steel was basically "1035" steel, or plain carbon steel with 35 percent carbon content. Eyebars manufactured from this steel were only 1 cent more per pound than common carbon steel.

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

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

Forging

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

Hammering

Hammering was the first method employed in shaping metals. An early form of the eyebar, shaped in this manner, is known as a loop rod (see Figures 10.9.10 and 10.9.11). Loop rods were first made of wrought iron (and later from steel) by forging a heated bar around a pin and pounding the bar until a closed loop was formed.

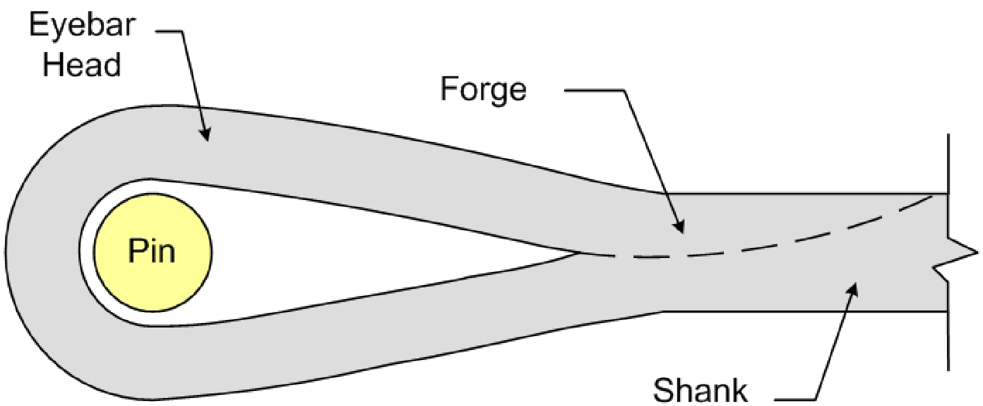

Figure 10.9.10 Forged Loop Rod

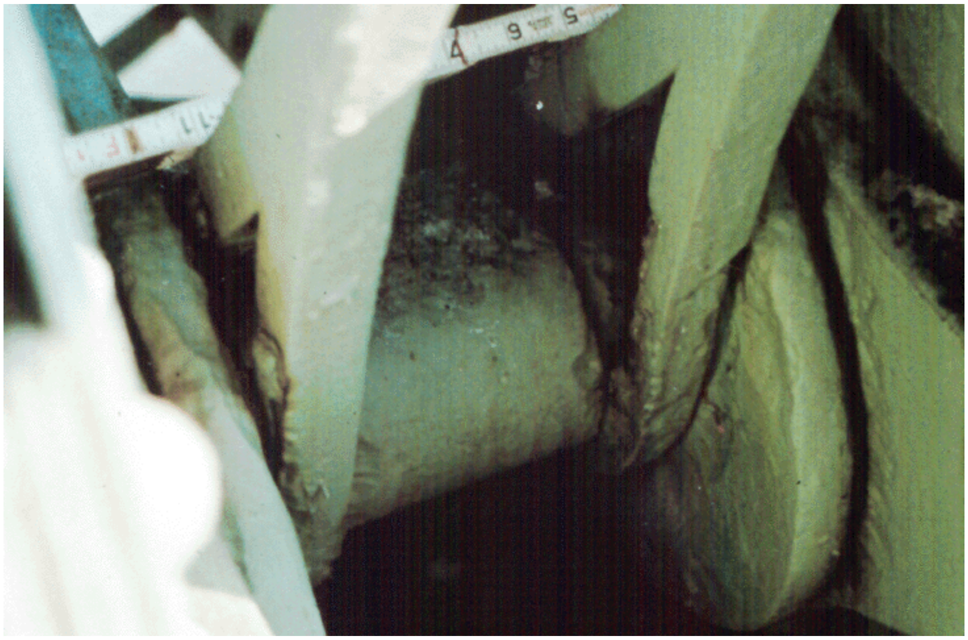

Figure 10.9.11 Close-up of the End of a Loop Rod

Pressing

Steel eyebars were also formed with a special type of mechanical forge press called an upsetting machine. The eyebar consists of the two heads (formed by casting) joined to the ends of the shaft (see Figure 10.9.12). The upsetting machine clamps the eyebar pieces between two dies with vertical faces. The eyebar is then forged and shaped by the horizontal action of a ram operated by a crankshaft. Most other forging presses operate with vertical rams.

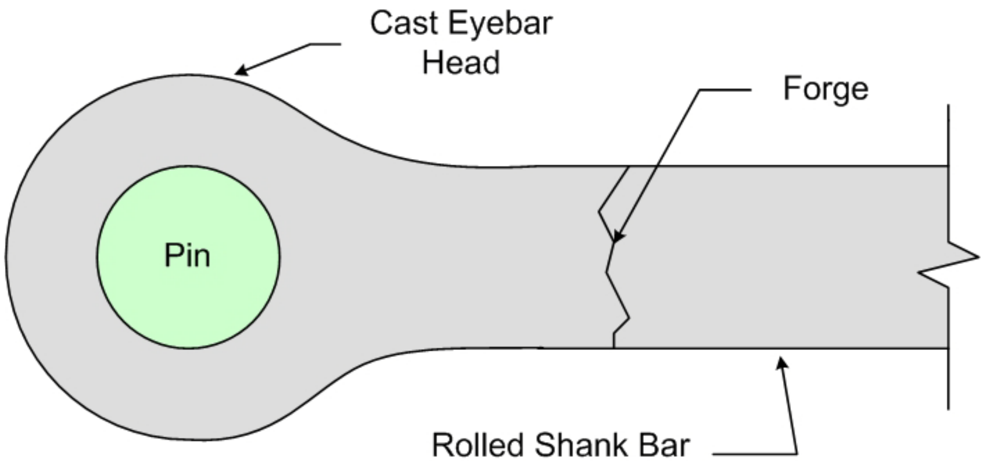

Figure 10.9.12 Forged Eyebar by Mechanical Forge Press

Pin Hole

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

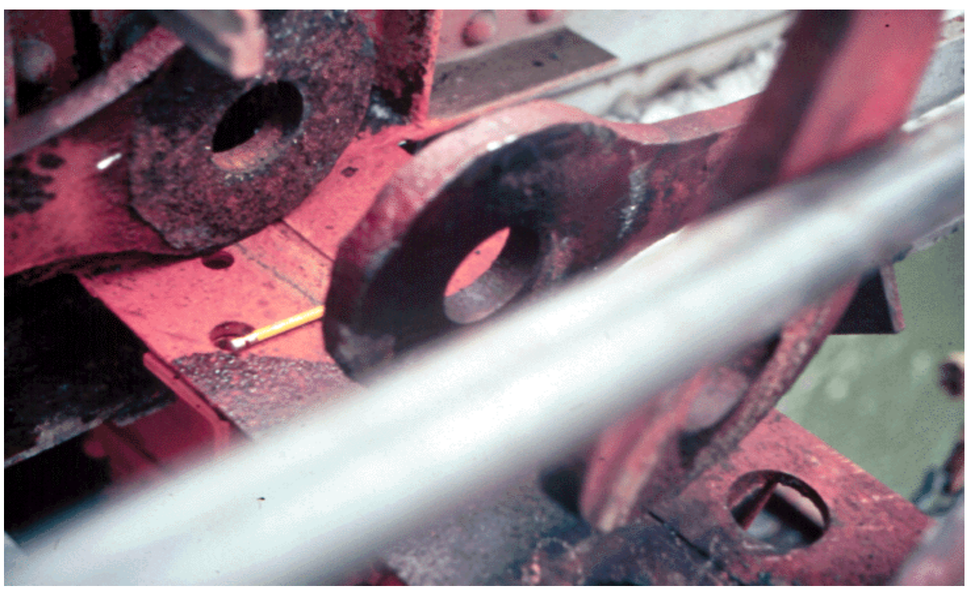

Figure 10.9.13 Eyebar Pin Hole (Disassembled Connection)

Heat Treating and Annealing

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

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

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

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

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

Dimensions and Nomenclature

The dimensions of a typical eyebar are as follows:

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

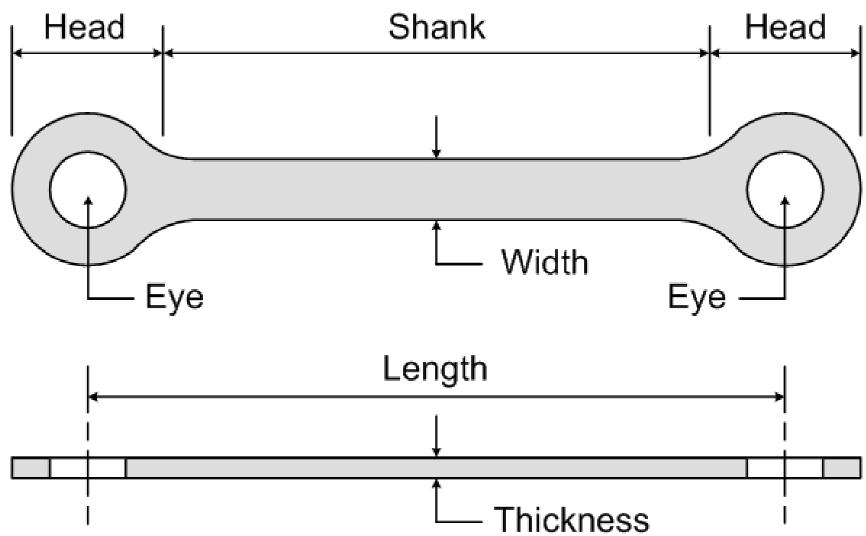

Figure 10.9.14 Eyebar Dimensions

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

Packing

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

Figure 10.9.15 Loosely Packed Eyebar Connection

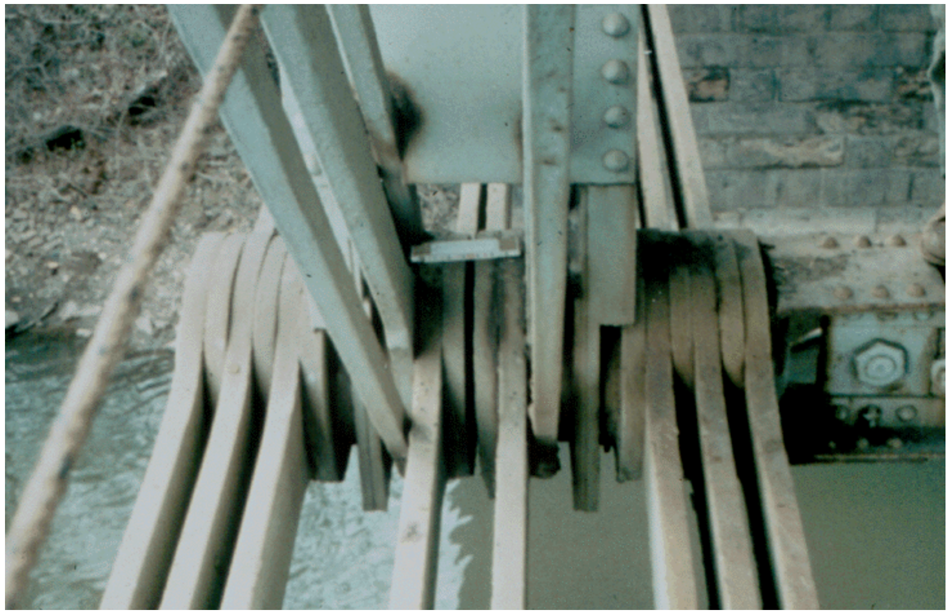

Figure 10.9.16 Tightly Packed Eyebar Connection

Spacers

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

Figure 10.9.17 Steel Pin Spacer or Filling Ring

Redundancy

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

The collapse of the Silver Bridge is attributed to the failure of an eyebar within a nonredundant eyebar member. When the first eyebar failed, the second eyebar was unable to carry the load due to lack of internal redundancy. The Silver Bridge was also not load path redundant which contributed to the complete collapse of the structure. Load path and internal (member) redundancy are discussed in detail in Topics 5.1 and 6.4.

Figure 10.9.18 Non-redundant Eyebar Member

10.9.3 Overview of Common Deficiencies

Common deficiencies that occur on eyebars and eyebar connections include:

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

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

10.9.4 Inspection Methods and Locations

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

Methods

Visual

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

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

Physical

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

When section loss occurs, use a wire brush, grinder or hammer to remove loose or flaked steel. After the flaked steel is removed, measure the remaining section and compare it to a similar section with no section loss.

The usual and most reliable sign of fatigue cracks is the oxide or rust stains that develop after the paint film has cracked. Experience has shown that cracks have generally propagated to a depth between one-fourth and one-half the plate thickness before the paint film is broken, permitting the oxide to form. This occurs because the paint is more flexible than the underlying steel.

Smaller cracks are not likely to be detected visually unless the paint, mill scale, and dirt are removed by carefully cleaning the suspect area. If the confirmation of a possible crack is to be conducted by another person, it is advisable not to disturb the suspected crack area so that re-examination of the actual conditions can be made.

Once the presence of a crack has been verified, examine other similar details and similar locations on the bridge.

Advanced Inspection Methods

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

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

Other methods, described in Topic 15.3.3, include:

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

Locations

Forge Zone

Inspect carefully the forged area around the eyebar head and the shank for cracks. Check the loop rods for cracks where the loop is formed (see Figure 10.9.19 and 10.9.20). Most eyebar failures are likely to occur in the forge zone.

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

Figure 10.9.20 Forge Loop is Completely Apart

Tension Zone

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

Alignment and Load Distribution

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

Figure 10.9.21 Bowed Eyebar Member

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

Figure 10.9.22 Buckled Eyebar due to Abutment Movement

Areas That Trap Water and Debris

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

Spacers

Examine the spacers on the pins to be sure they are holding the eyebars in their proper position (see Figure 10.9.23).

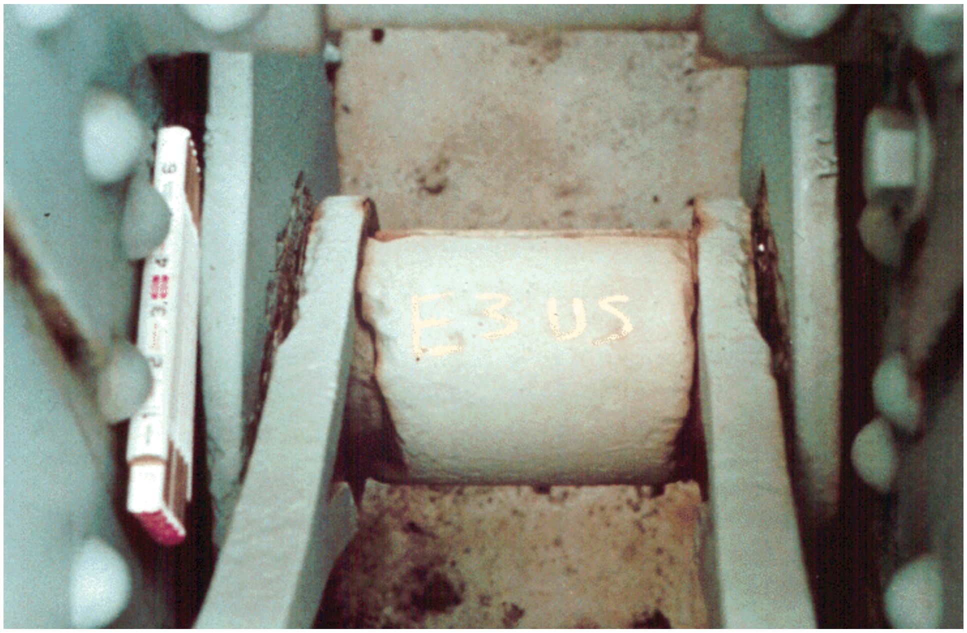

Figure 10.9.23 Corroded Spacer

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

Verify the eyebars are symmetrical about the central plane of the spacer (see Figure 10.9.24).

Figure 10.9.24 Asymmetry at an Eyebar Connection

Load Distribution

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

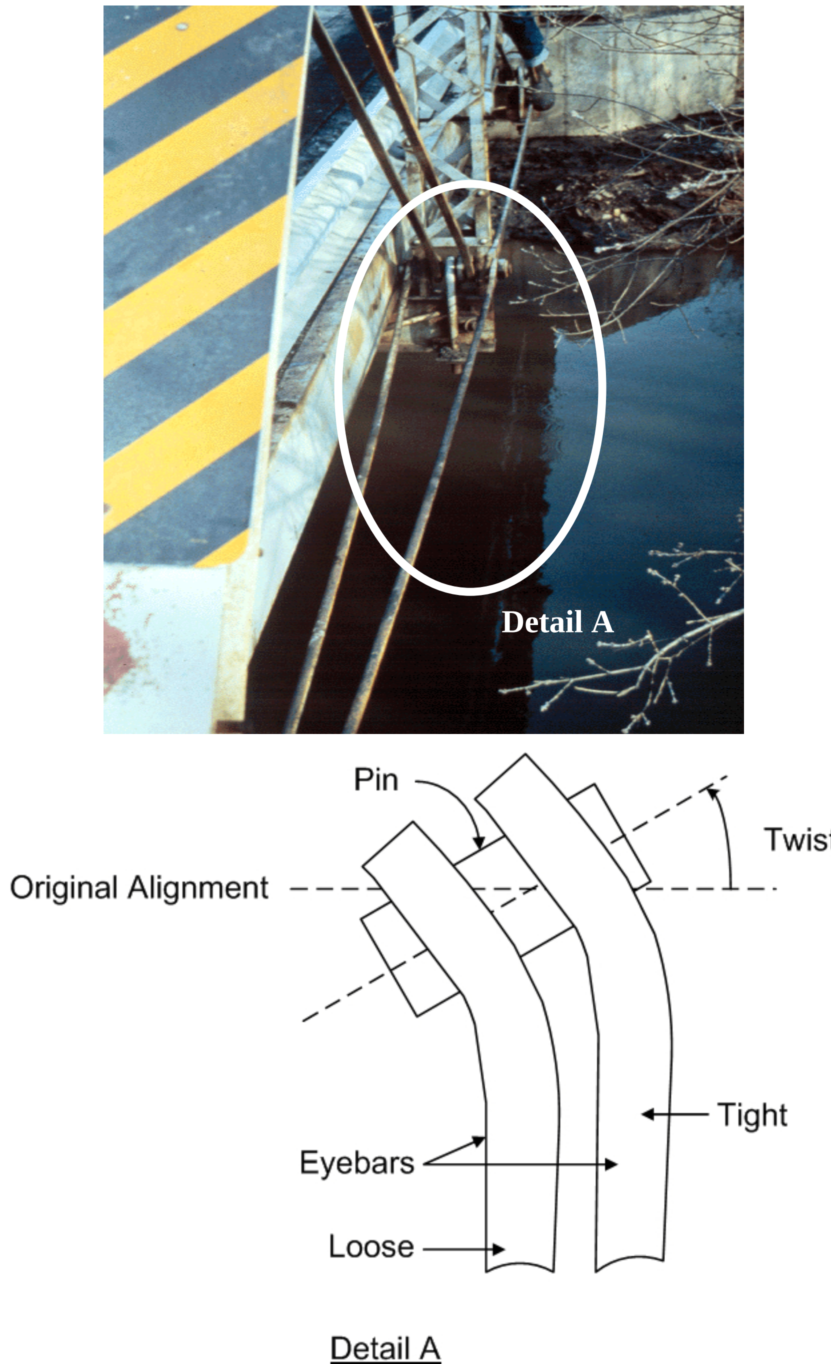

Figure 10.9.25 Eyebar Member with Unequal Load Distribution

Weldments

Evaluate the integrity of any welded repairs to the eyebar (see Figure 10.9.26). Check for any welds used in repairing or strengthening the eyebar, as well as field welds for utility supports (see Figure 10.9.26). Include weld locations in the inspection report so that the engineer can analyze the severity of their effect on the member (see Figure 10.9.27). Most of these bridges are old and constructed of steel which is considered "unweldable" by today's standards. It is difficult to obtain a high quality "field" weld.

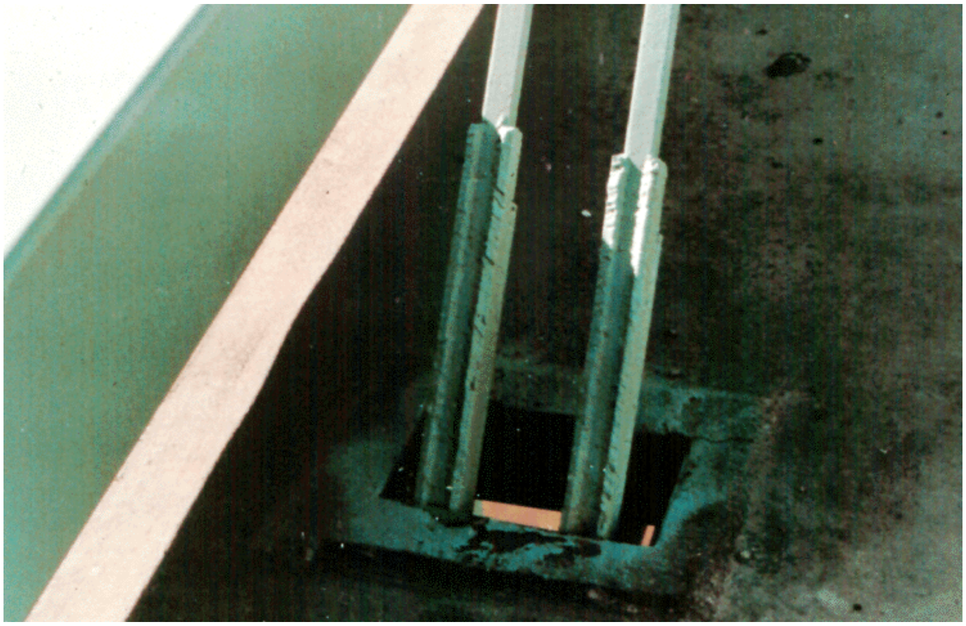

Figure 10.9.26 Welds on Loop Rods

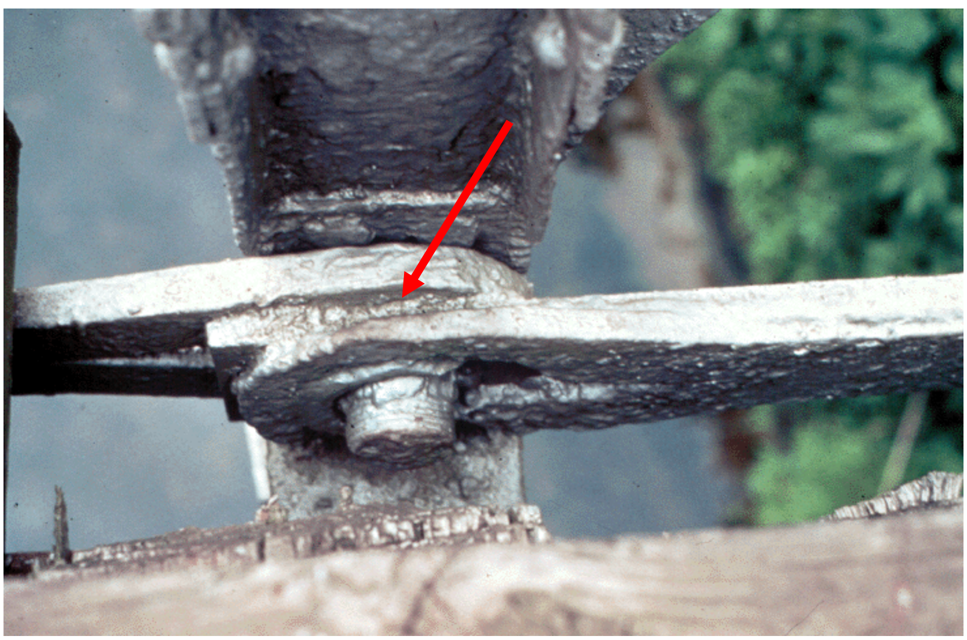

Figure 10.9.27 Welded Repair to Loop Rods

Turnbuckles

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

Figure 10.9.28 Turnbuckle on a Truss Diagonal

Figure 10.9.29 Welded Repair to Turnbuckles

Areas Exposed to Traffic

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

Pins

Inspect pins for signs of wear and corrosion. Nondestructive methods such as ultrasonic inspection are recommended since visual inspection cannot reveal internal material flaws that may exist (see Figure 10.9.30).

Figure 10.9.30 Ultrasonic Inspection of Eyebar Pin

Fracture Critical Members

Eyebars are normally used on truss or suspension bridges. Since these bridge types normally only have two load paths between substructure supports, the bridges are considered non-load path redundant. If a steel eyebar member failure would cause total or partial collapse of the bridge, then that eyebar is considered a fracture critical member. Truss members that have one or two eyebars between panel points are not considered internally redundant (see Figure 10.9.31). Truss members that have three or more eyebars between panel points may be considered internally redundant (see Figure 10.9.32). See Topic 6.4 for a detailed discussion on fracture critical members and types of redundancy.

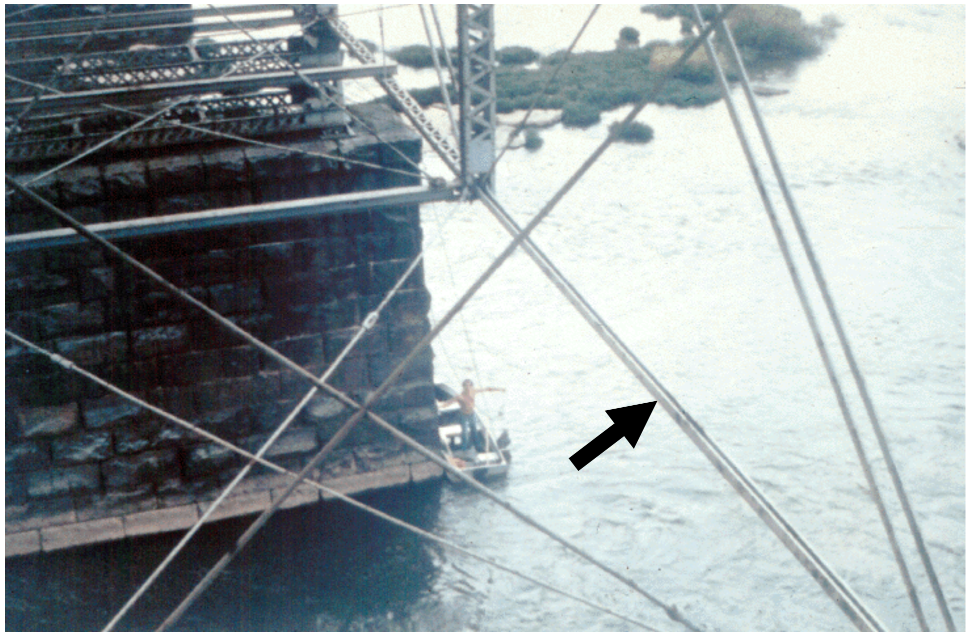

Figure 10.9.31 Fracture Critical Bottom Chord Truss Member: Internally Non-redundant Eyebar

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

10.9.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

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

Element Level Condition State Assessment

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

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

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

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

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

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

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

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Chapter 11

Inspection and Evaluation of Bridge Bearings

Chapter 11
Inspection and Evaluation of Bridge Bearings

Topic 11.1 Bridge Bearings

11.1.1 Introduction

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

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

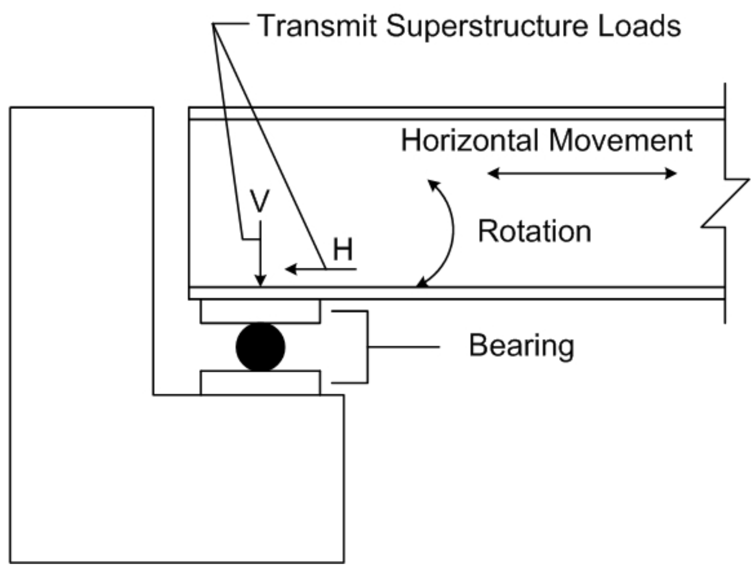

Figure 11.1.1 Three Functions of a Bearing

Fixed and Moveable Bearings

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

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

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

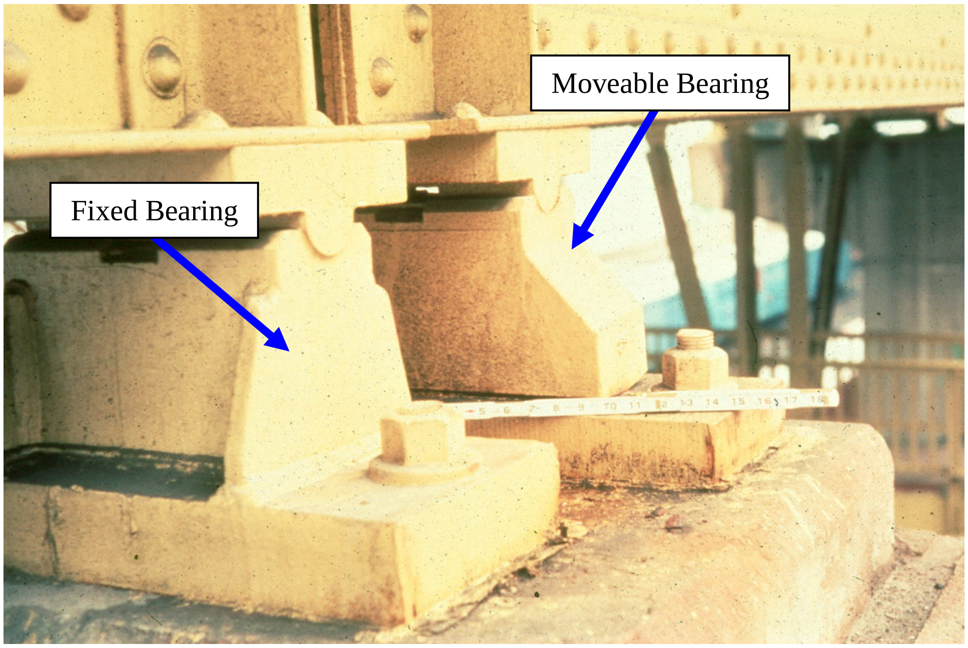

Figure 11.1.2 Fixed and Moveable Bearings

11.1.2 Four Basic Elements of a Bearing

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

Sole Plate

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

Bearing or Bearing Device

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

Masonry Plate

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

Anchor Bolts

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

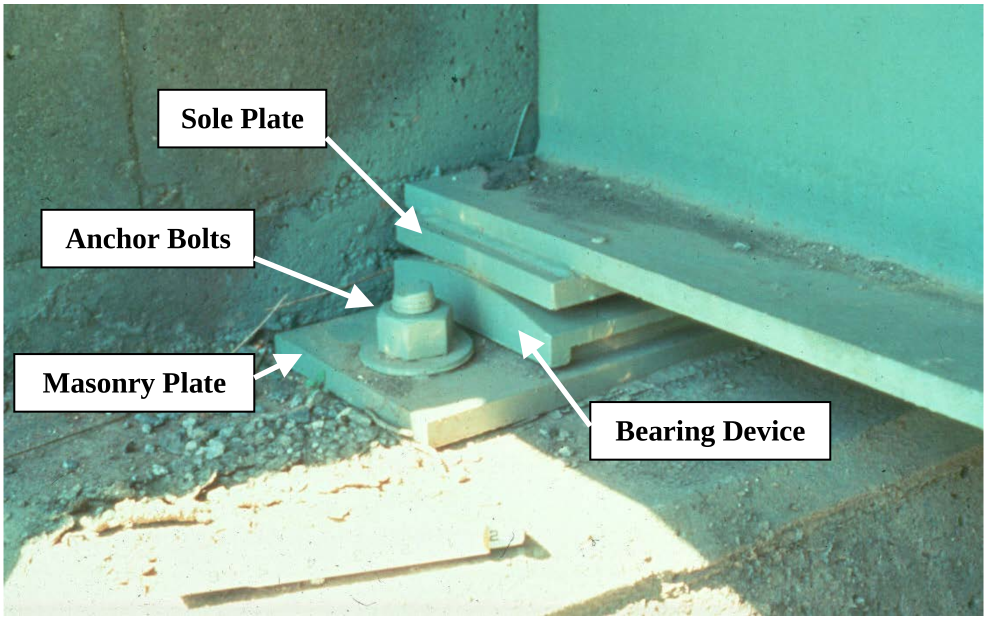

Figure 11.1.3 Elements of a Typical Bridge Bearing

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

11.1.3 Bearing Types and Functionality

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

These six types of bearings are:

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

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

Moveable Bearings

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

- Sliding plate bearings
- Roller bearings
- Rocker bearings

Sliding Plate Bearings

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

Lubricated Steel Plates

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

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

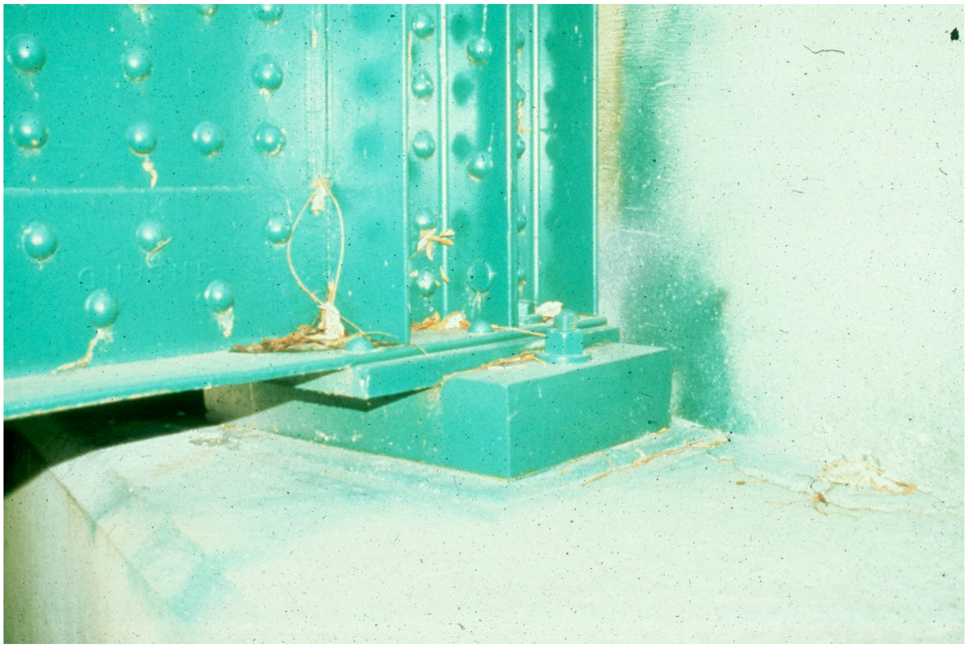

Figure 11.1.4 Lubricated Steel Plate Bearing

Lead Sheets Between Steel Plates

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

Bronze Bearing Plates

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

Figure 11.1.5 Bronze Sliding Plate Bearing

Asbestos Sheet Packing Between Metal Plates

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

Self-Lubricating Bronze Bearings

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

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

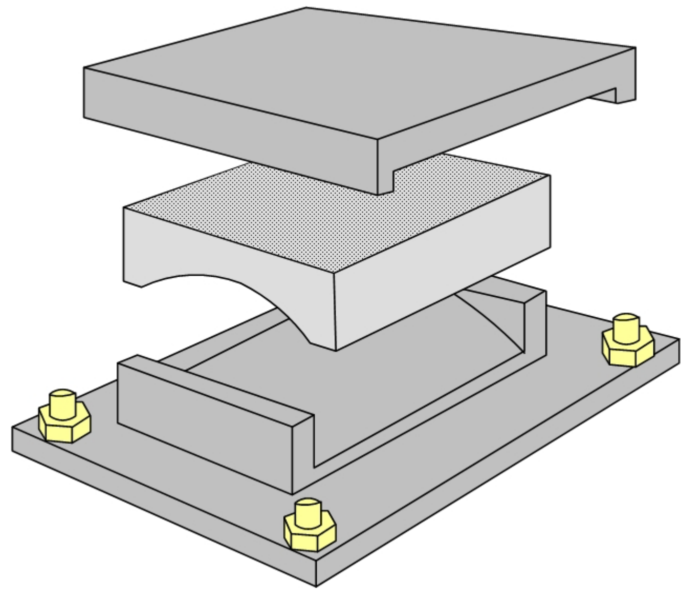

Figure 11.1.6 Self-lubricating Bronze Sliding Plate Bearing

Roofing Felt or Tar Paper

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

PTFE on Stainless Steel Plates

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

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

Roller Bearings

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

Single Roller Bearings

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

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

Figure 11.1.7 Single Roller Bearing

Roller Nest Bearings

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

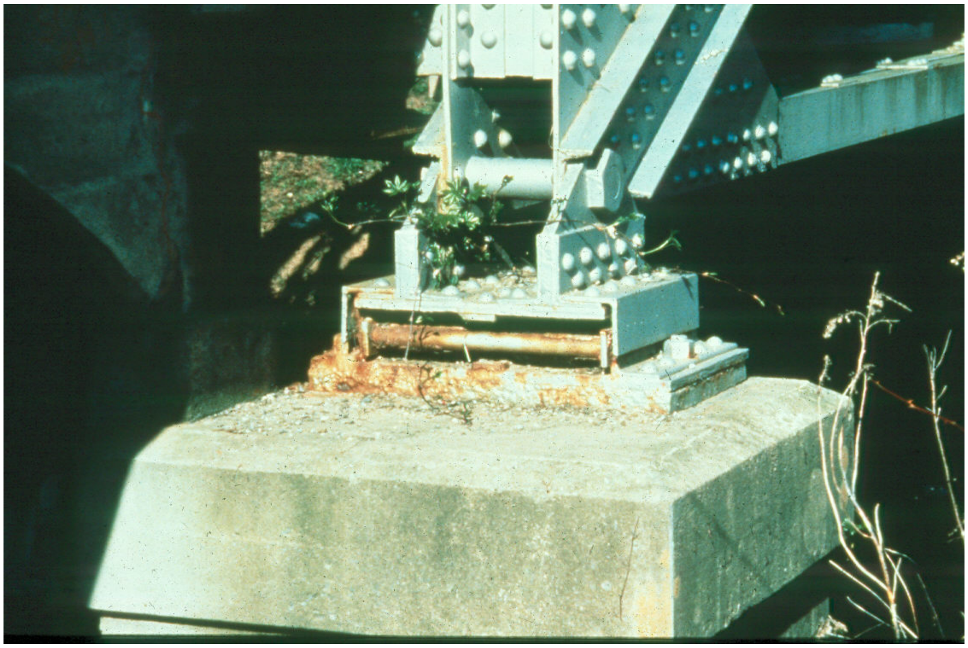

Figure 11.1.8 Roller Nest Bearing

Rocker Bearings

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

Figure 11.1.9 Rocker Bearing

Segmental Rocker Bearings

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

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

Figure 11.1.10 Segmental Rocker Bearing

Rocker Nest Bearings

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

Figure 11.1.11 Segmental Rocker Nest Bearing

Pinned Rocker Bearings

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

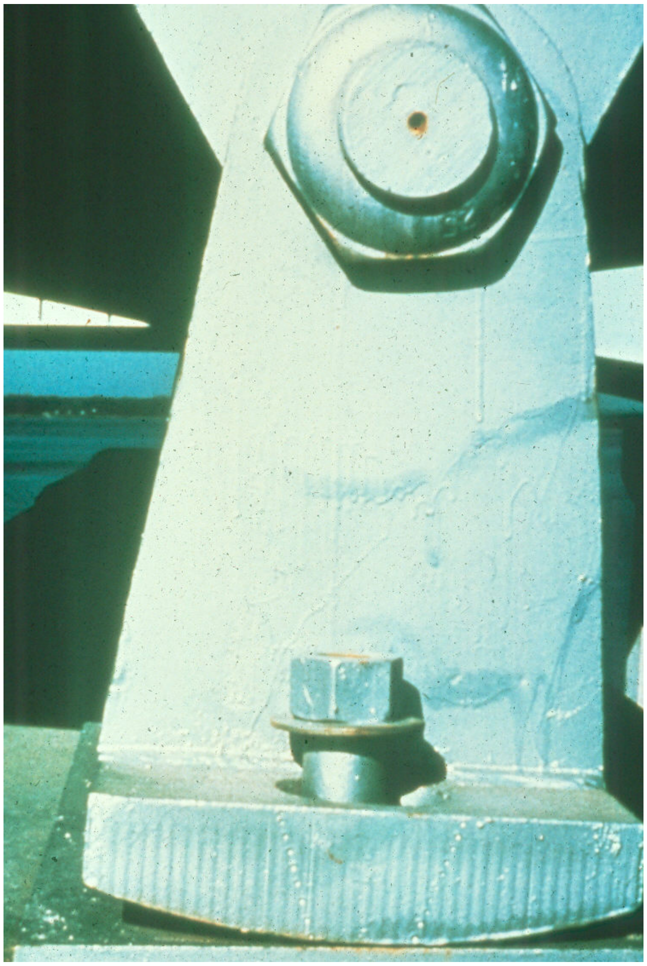

Figure 11.1.12 Pinned Rocker Bearing

Elastomeric Bearings

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

Plain Neoprene Pads

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

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

Figure 11.1.13 Plain Neoprene Bearing Pad

Laminated Neoprene Pads

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

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

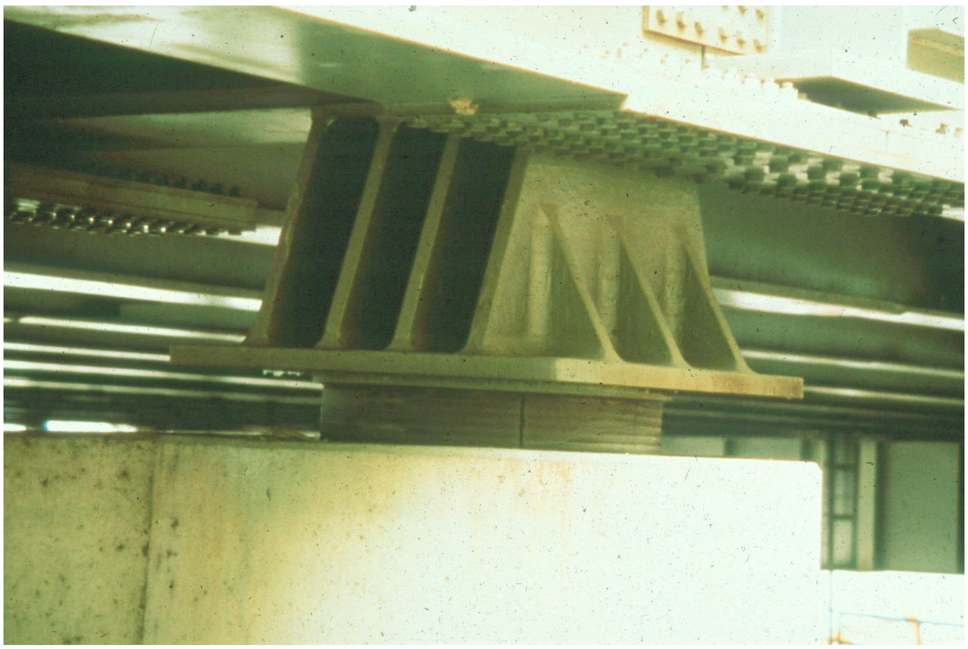

Figure 11.1.14 Laminated Neoprene Bearing Pad

Pot Bearings

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

Neoprene Pot Bearings

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

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

Figure 11.1.15 Neoprene Pot Bearing with Guide Bars

Disk Bearings

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

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

Figure 11.1.16 Disk Bearing

Fixed Bearings

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

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

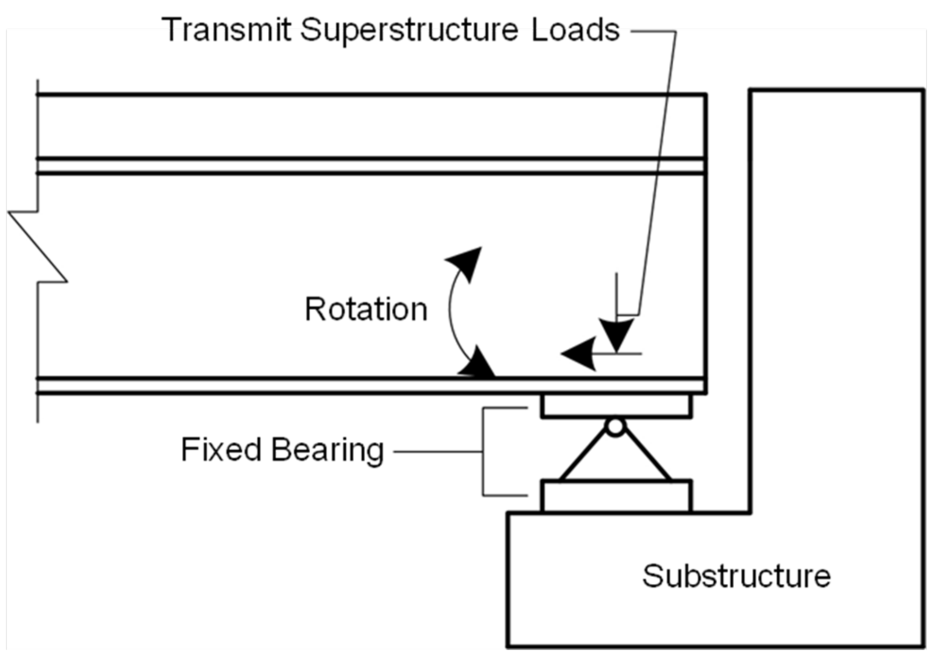

Figure 11.1.17 Fixed Bearing

Enclosed or Concealed Bearings

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

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

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

Figure 11.1.18 Enclosed or Concealed Bearing

Uncommon Bearings

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

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

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

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

Figure 11.1.19 Pin and Link Bearing

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

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

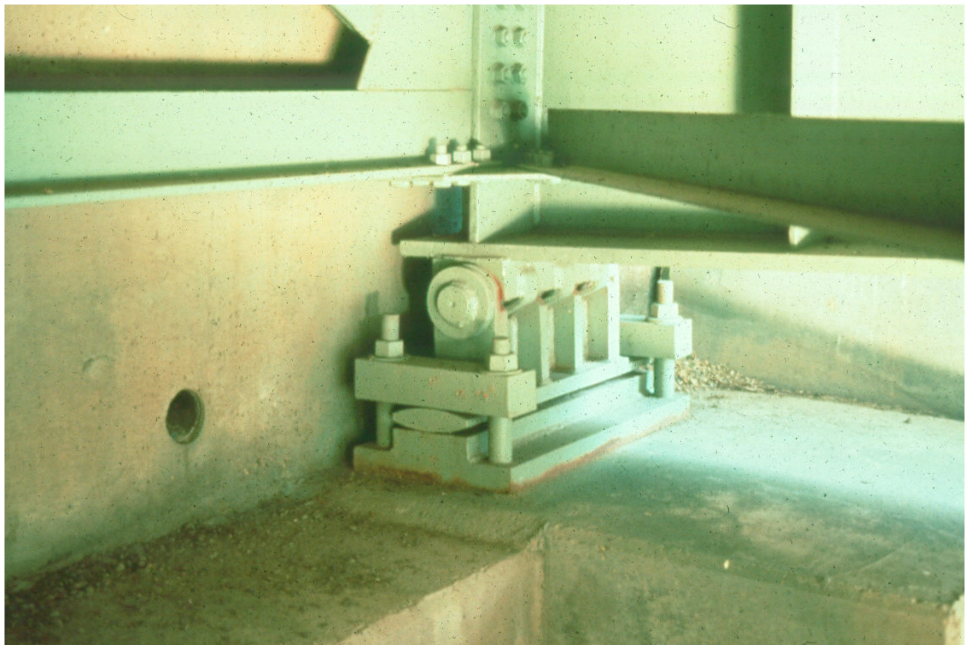

Figure 11.1.20 Restraining Bearing

Isolation Bearings (Moveable, Elastomeric or Pot Bearings)

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

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

Lead Core Bearings (Elastomeric Bearings)

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

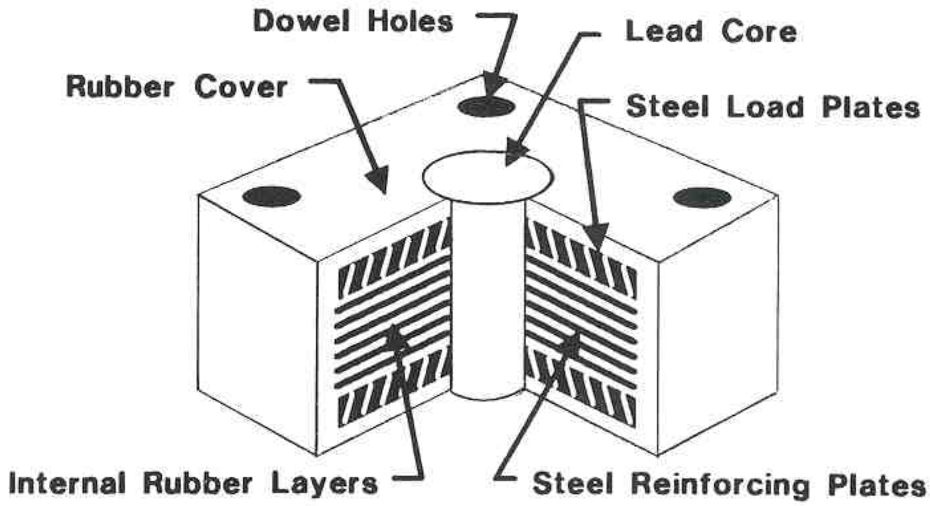

Figure 11.1.21 Sketch of a Lead Core Isolation Bearing

Figure 11.1.22 Lead Core Isolation Bearing

Friction Pendulum Bearings (Moveable or Pot Bearings)

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

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

Figure 11.1.23 Friction Pendulum Bearing

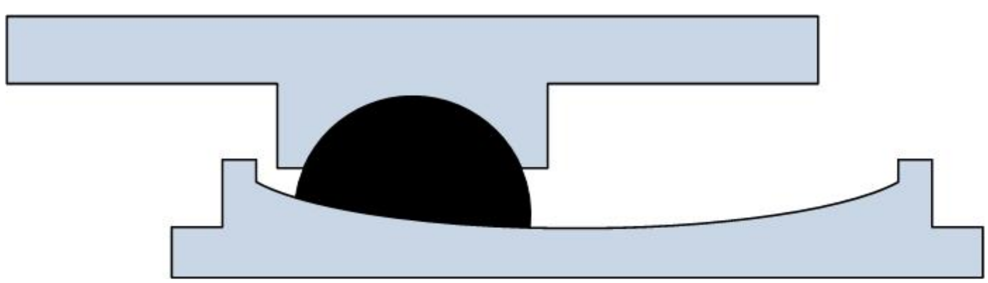

Figure 11.1.24 Sketch of a Friction Pendulum Bearing

High-Damping Rubber Bearings (Elastomeric Bearings)

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

Spherical Pot Bearings (Pot Bearings)

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

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

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

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

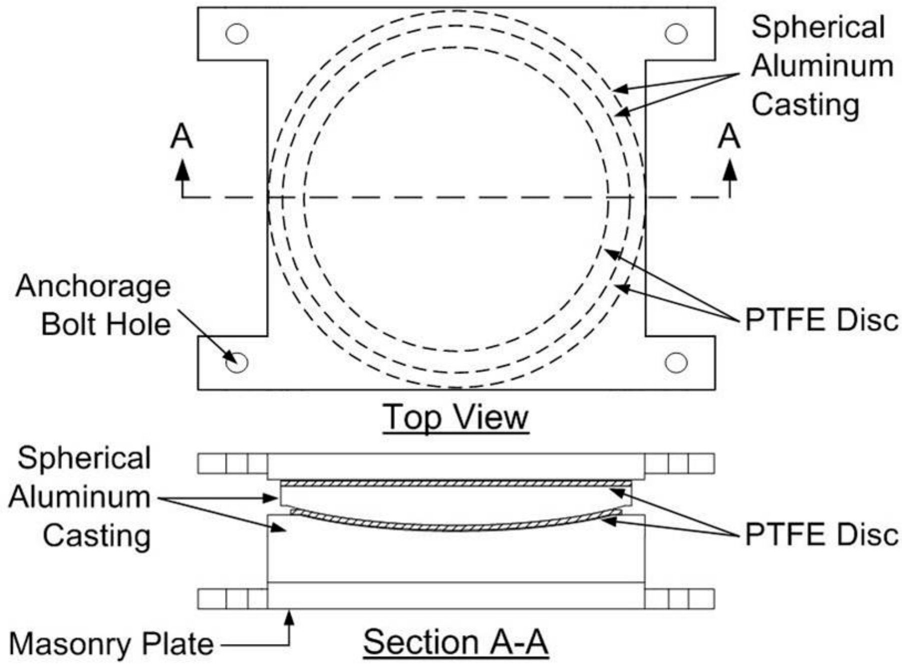

Figure 11.1.25 Spherical Pot Bearing

11.1.4 Inspection Methods and Locations

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

- General
- Steel bearings
- Elastomeric bearings

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

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

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

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

General

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

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

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

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

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

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

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

Inspection of Steel Bearings

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

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

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

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

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

Figure 11.1.27 Ultrasonic Testing Inspection of a Pin in a Bearing

Corrosive Forces

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

Figure 11.1.28 Heavy Corrosion on a Steel Rocker Bearing

Looseness

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

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

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

Figure 11.1.29 Rocker Bearing with Excessive Horizontal Movement

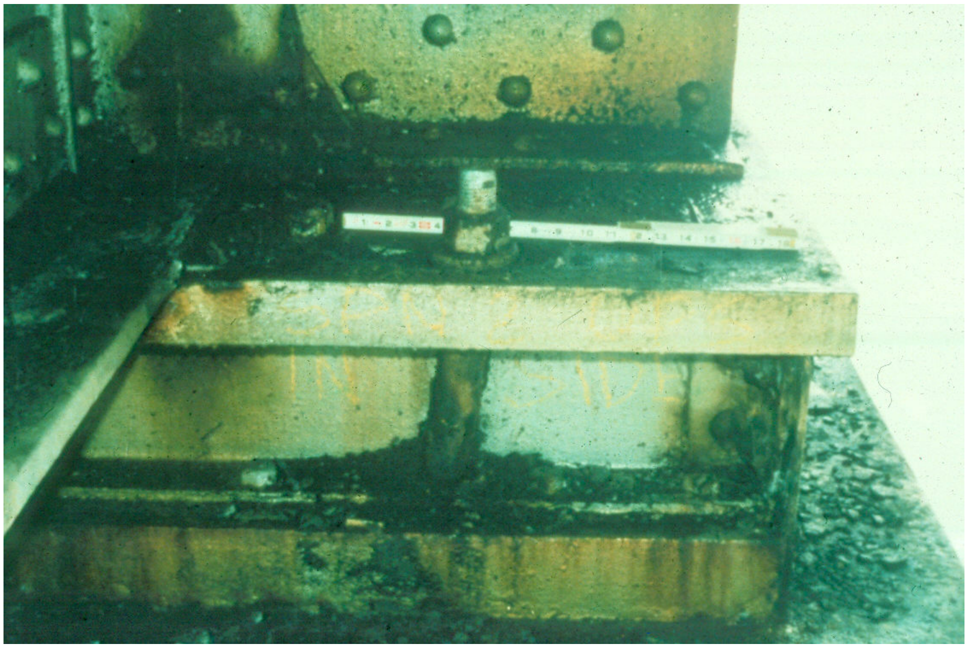

Figure 11.1.30 Bent Anchor Bolt due to Excessive Horizontal Movement

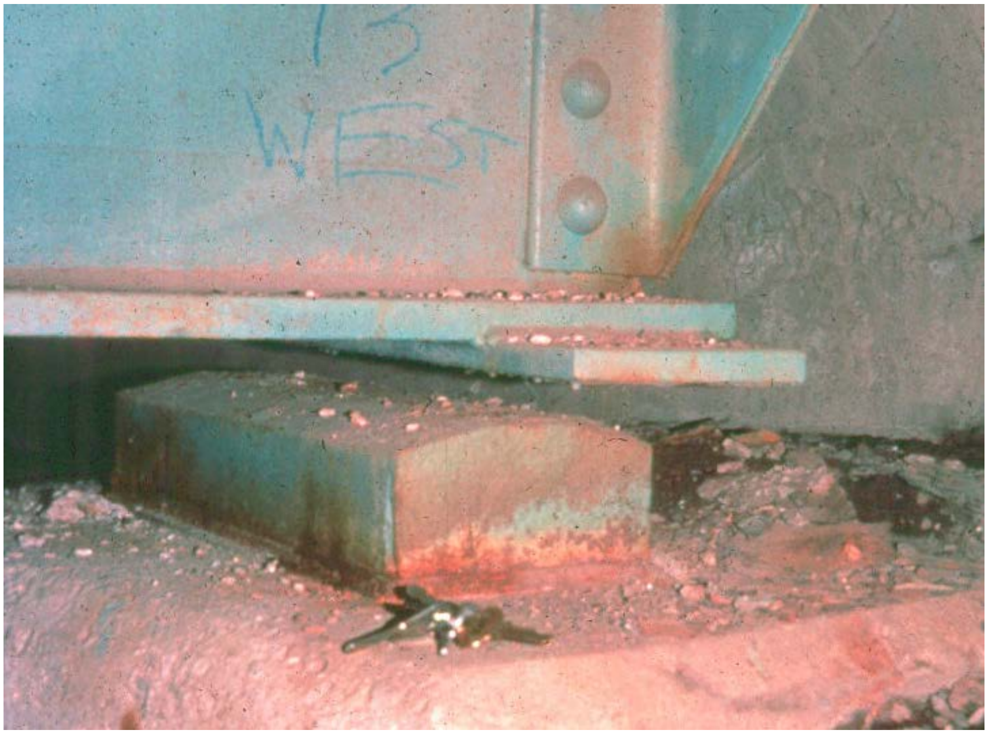

Figure 11.1.31 Uplift at Bridge Bearing

Sliding Plate Bearings

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

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

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

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

Figure 11.1.32 Longitudinal Misalignment in Bronze Sliding Plate Bearing

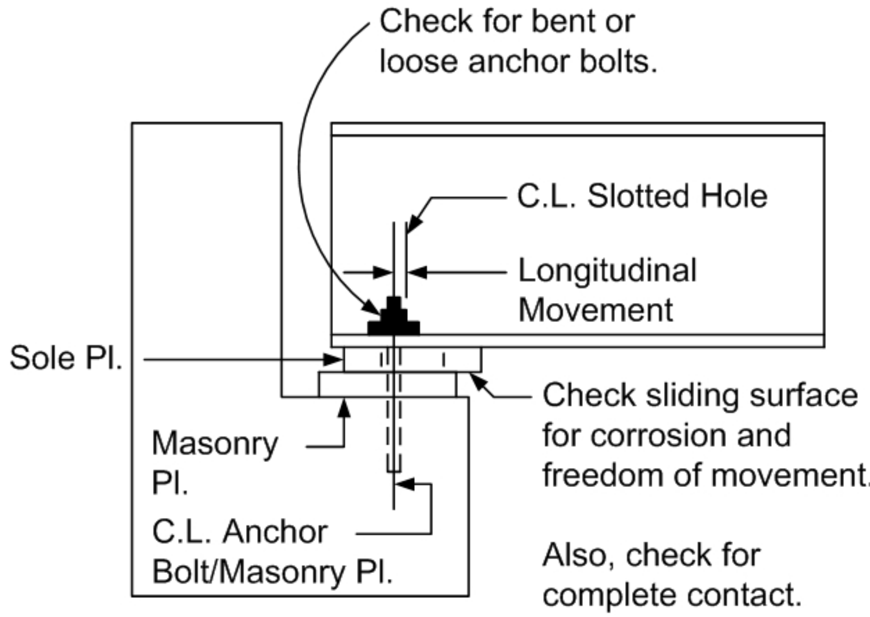

Figure 11.1.33 Sliding Plate Bearing Inspection Checklist Items

Roller Bearings

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

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

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

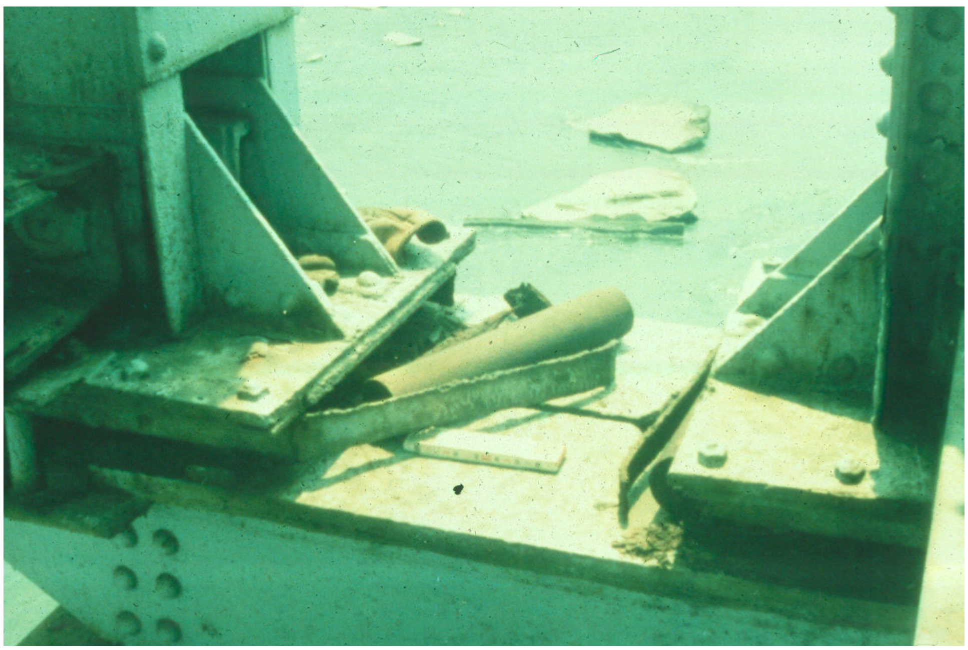

Figure 11.1.34 Damaged Roller Nest Bearing

Rocker Bearings

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

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

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

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

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

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

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

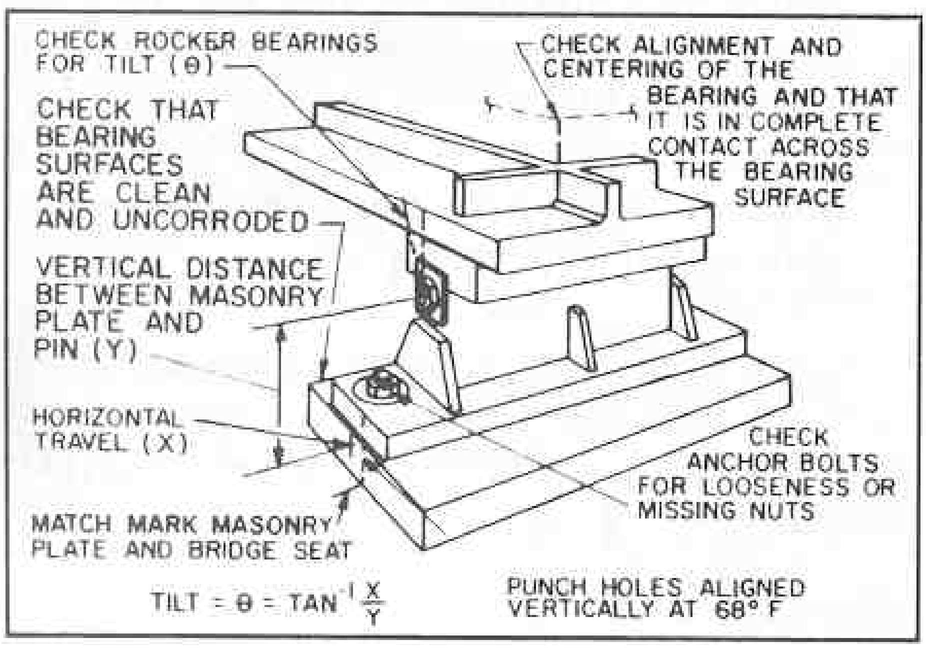

Figure 11.1.35 Rocker Bearing Inspection Checklist Items

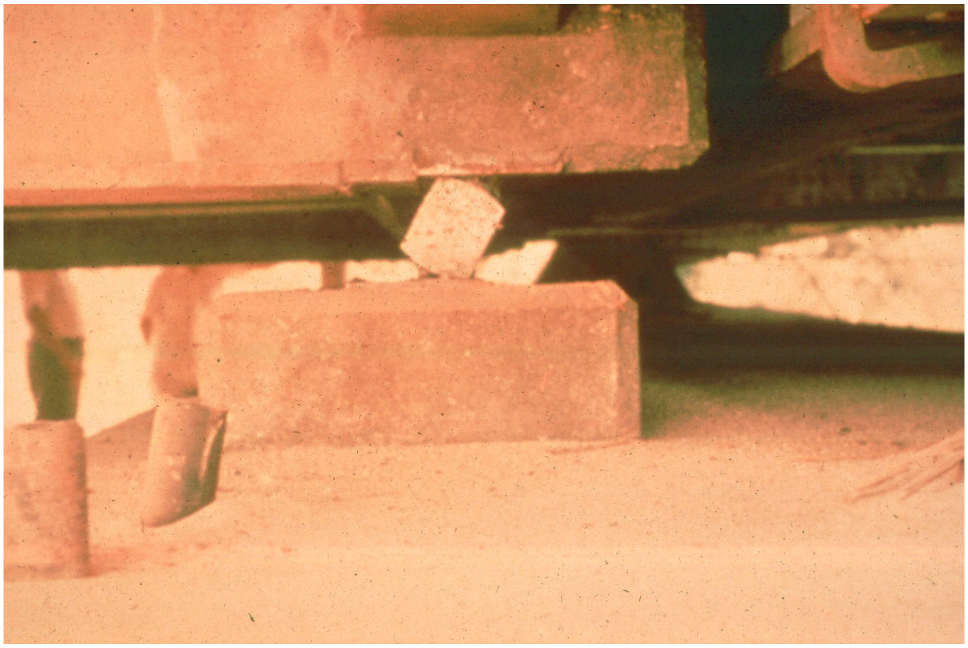

Figure 11.1.36 Excessive Tilt in a Segmental Rocker

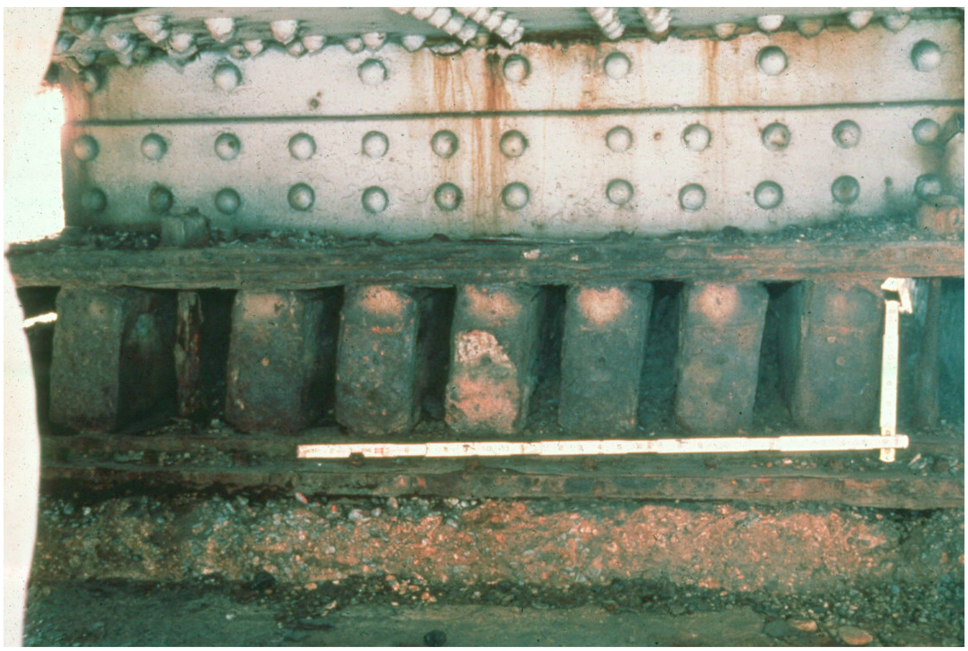

Figure 11.1.37 Frozen Rocker Nest

Pot Bearings

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

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

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

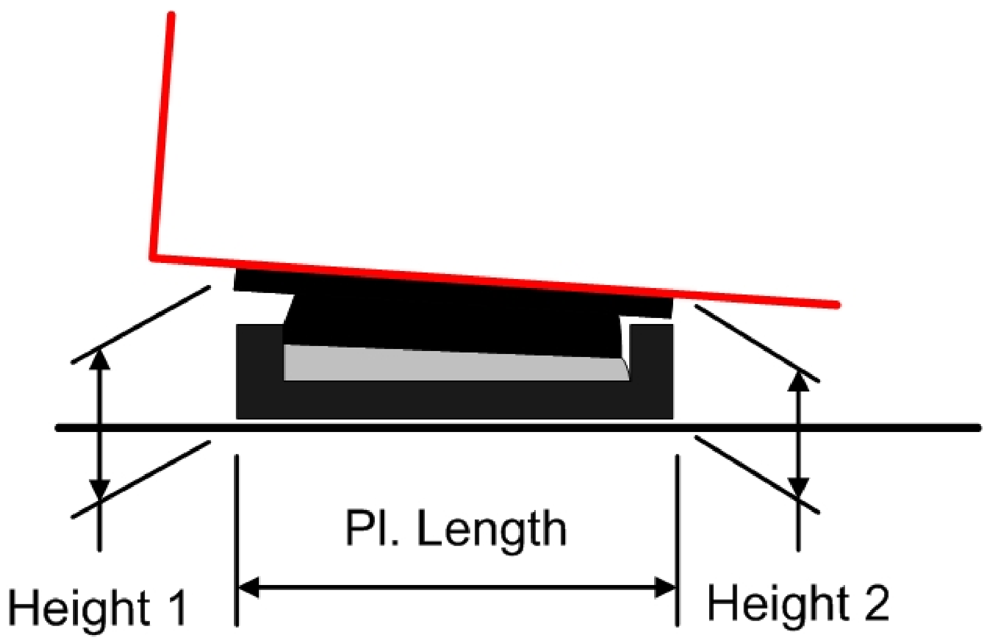

Figure 11.1.38 Frozen Rocker Nest

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

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

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

Pin and Link Bearings (Uncommon Bearing)

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

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

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

Restraining Bearings (Uncommon Bearing)

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

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

Inspection of Elastomeric Bearings

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

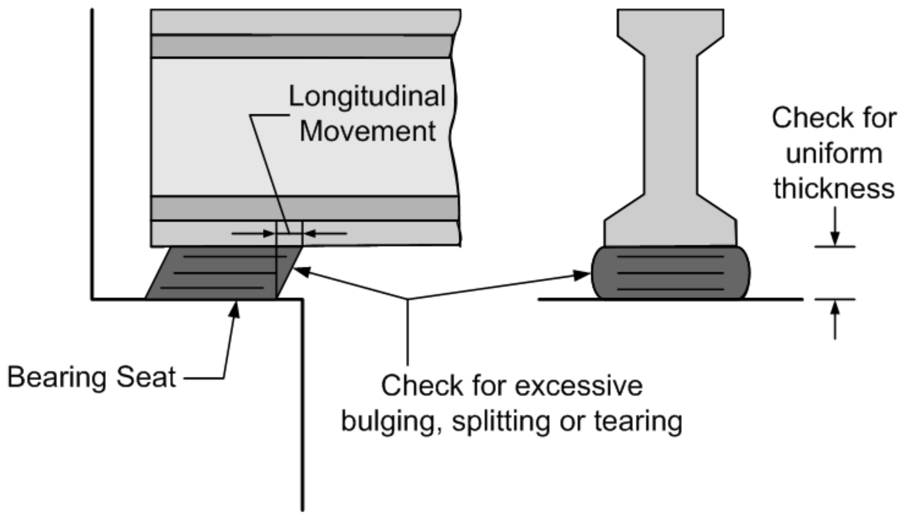

Figure 11.1.39 Elastomeric Bearing Inspection Checklist Items

Neoprene Bearings

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

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

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

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

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

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

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

Figure 11.1.40 Neoprene Bearing Pad Excessive Bulging

Isolation Bearings (Uncommon Bearing)

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

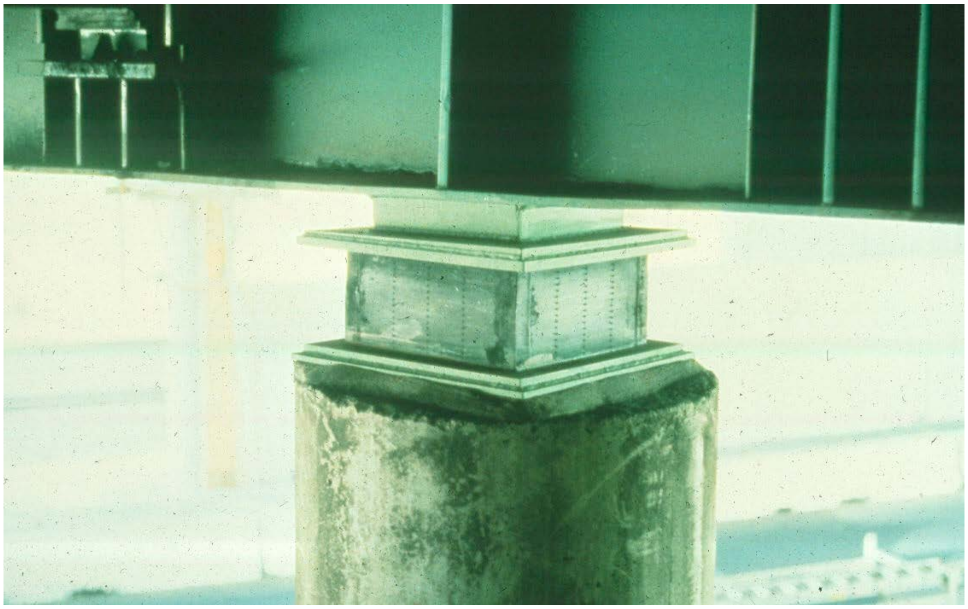

Figure 11.1.41 Lead Core Isolation Bearing

11.1.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

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

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

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

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

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

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

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

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

Serious Bearing Conditions

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

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

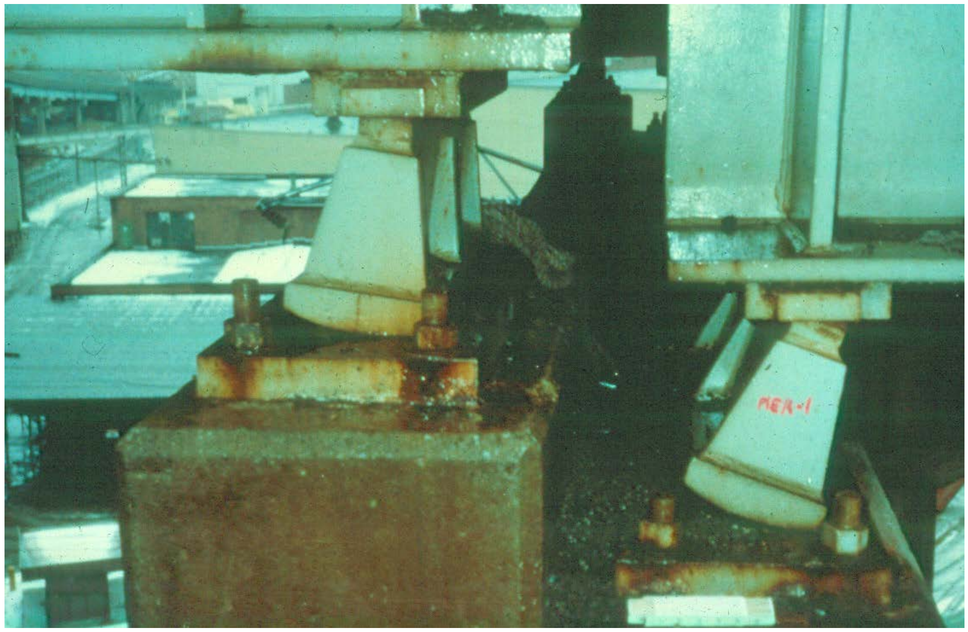

Figure 11.1.42 Serious Bearing Condition

Figure 11.1.43 Broken Pintle on a Bearing

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Inspection and Evaluation of Substructures

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Chapter 12
Inspection and Evaluation of Substructures

Topic 12.1 Abutments and Wingwalls

12.1.1 Introduction

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

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

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

12.1.2

Design Characteristics of Abutments

Common Abutment Types

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

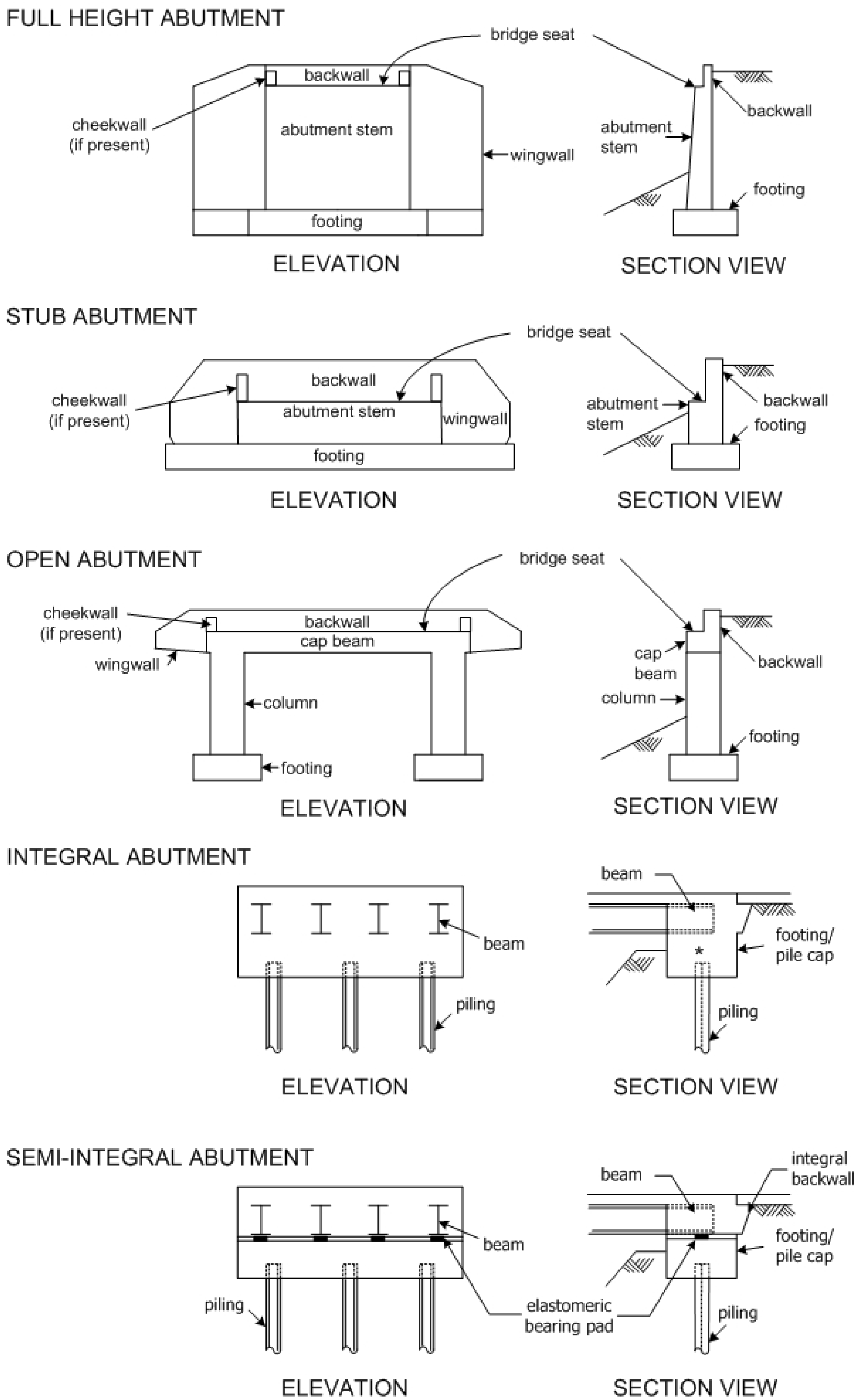
- Full height or closed type
- Stub, semi-stub, or shelf type
- Open or spill-through type
- Integral
- Semi-integral

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

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

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

Detailed descriptions of abutment elements are provided on page 12.1.15.



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

Figure 12.1.1 Schematic of Common Abutment Types

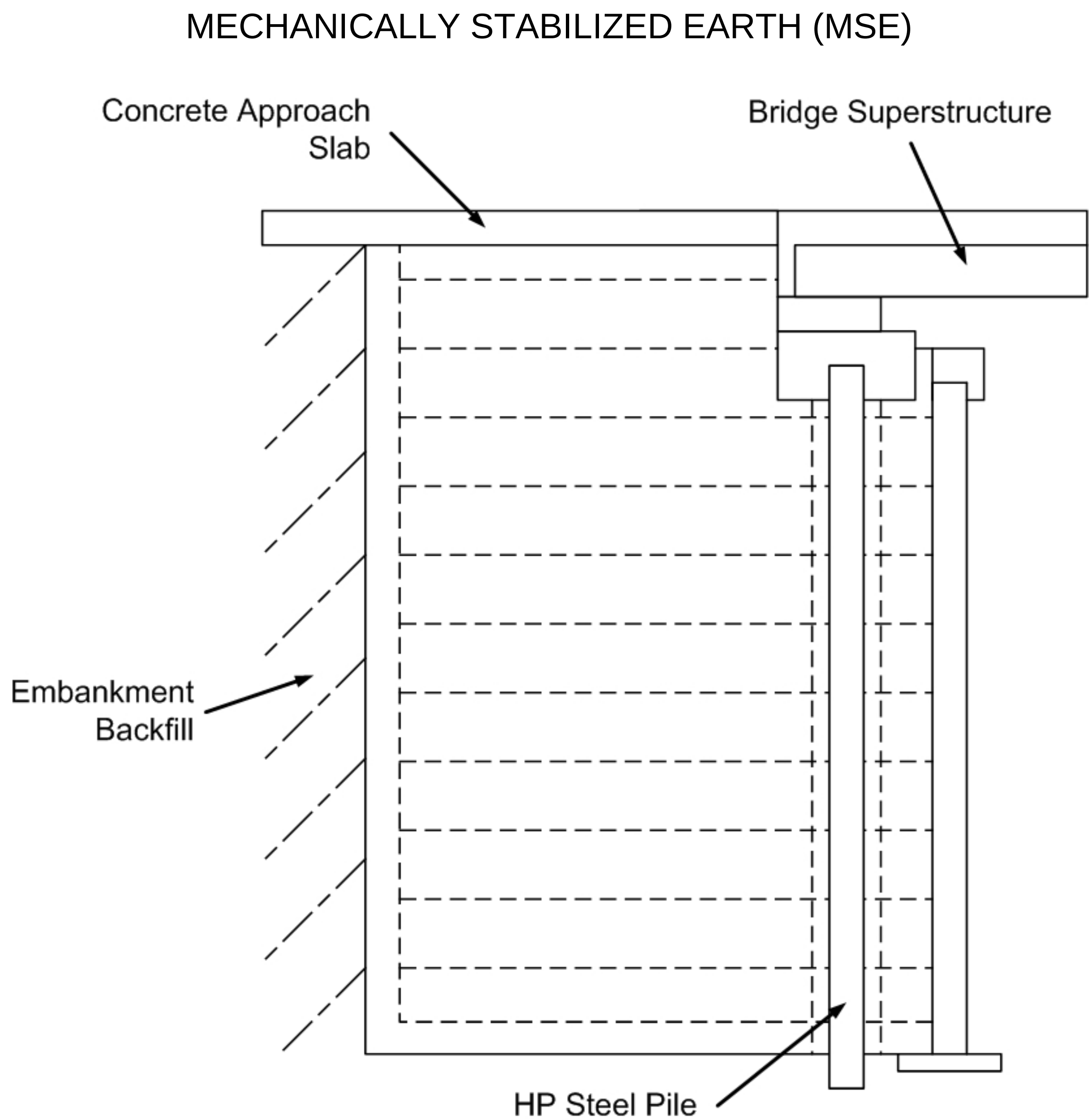


Figure 12.1.2   Section View of Less Common Abutment Types (Mechanically Stabilized Earth)

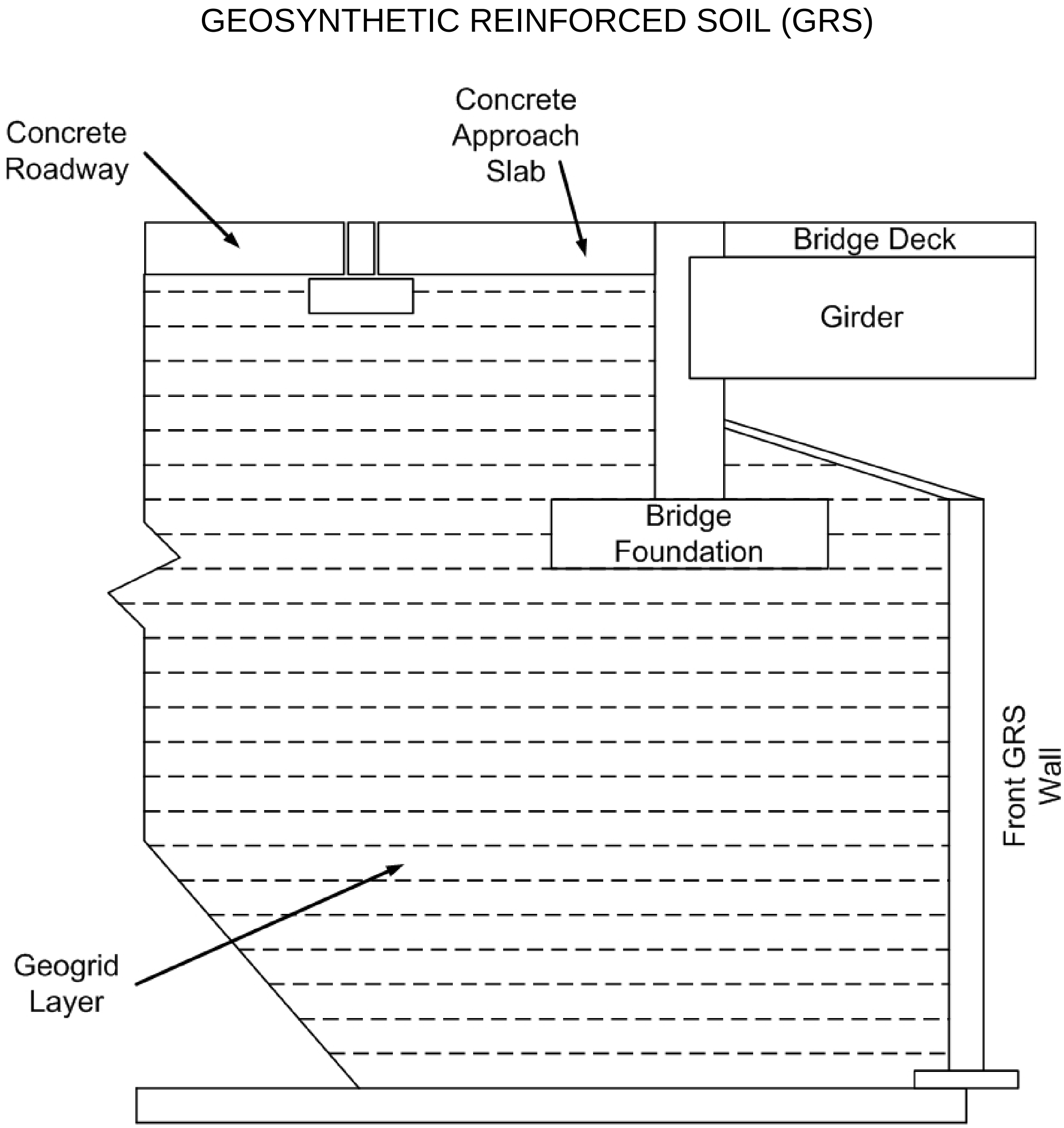


Figure 12.1.3 Section View of Less Common Abutment Types (Geosynthetic Reinforced Soil)

Full Height Abutments and Stub Abutments

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

Figure 12.1.4 Full Height Abutment

Figure 12.1.5 Stub Abutment

Open Abutments

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

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

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

Figure 12.1.6 Open Abutment

Integral Abutments and Semi-Integral Abutments

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

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

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

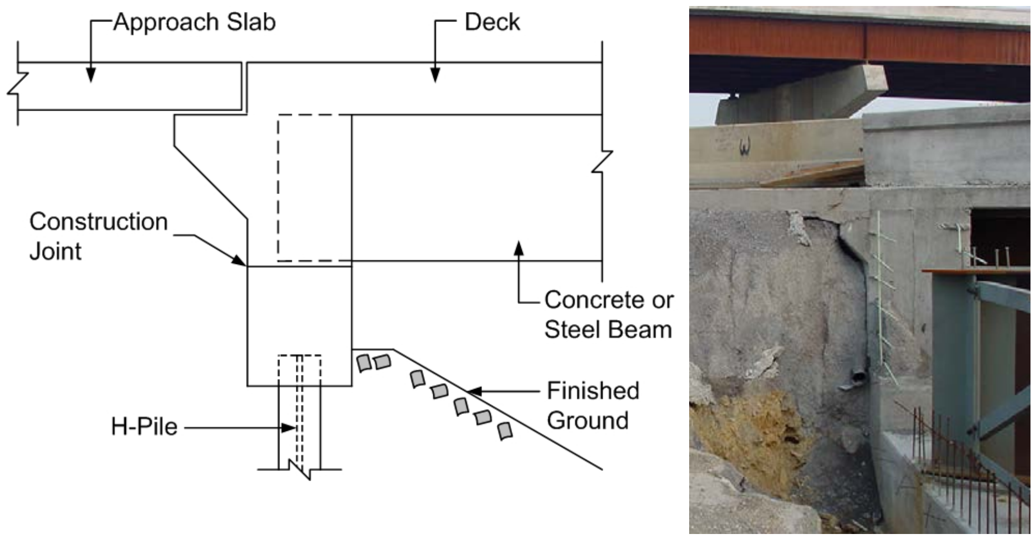


Figure 12.1.7 Integral Abutment

Figure 12.1.8 Integral Abutment

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

Less Common Abutment Types

Some Agencies utilize additional, less common abutment types.

Mechanically Stabilized Earth Abutments

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

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

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

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

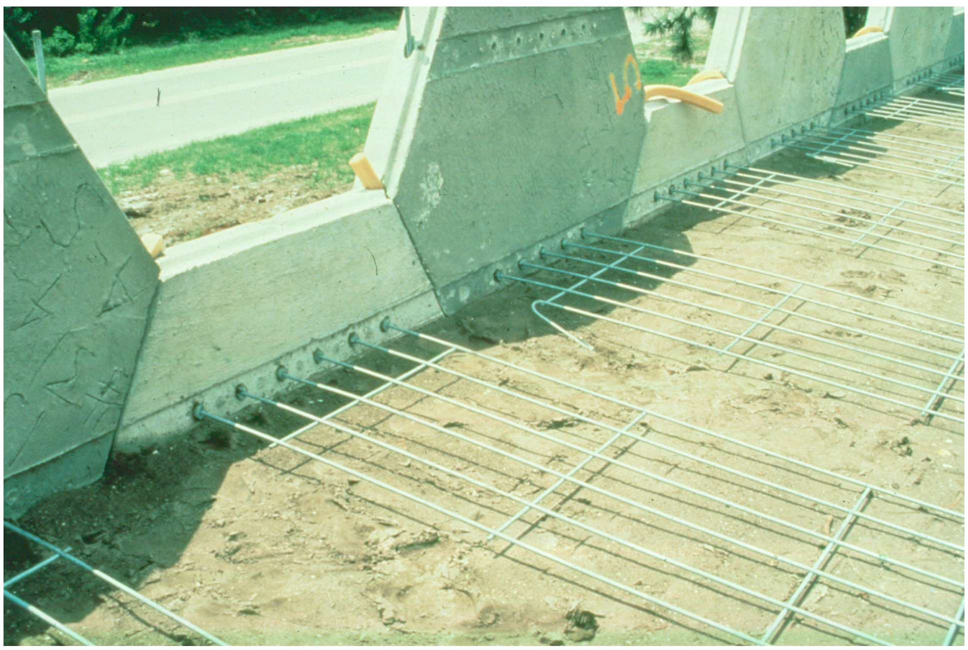


Figure 12.1.10 Mechanically Stabilized Earth Wall Under Construction

Geosynthetic Reinforced Soil Abutments

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

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

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

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

Primary Materials

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

Figure 12.1.13 Plain Unreinforced Concrete Gravity Abutment

Figure 12.1.14 Reinforced Concrete Cantilever Abutment

Figure 12.1.15 Stone Masonry Gravity Abutment

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

Figure 12.1.17 Steel Abutment

Primary and Secondary Reinforcement

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

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

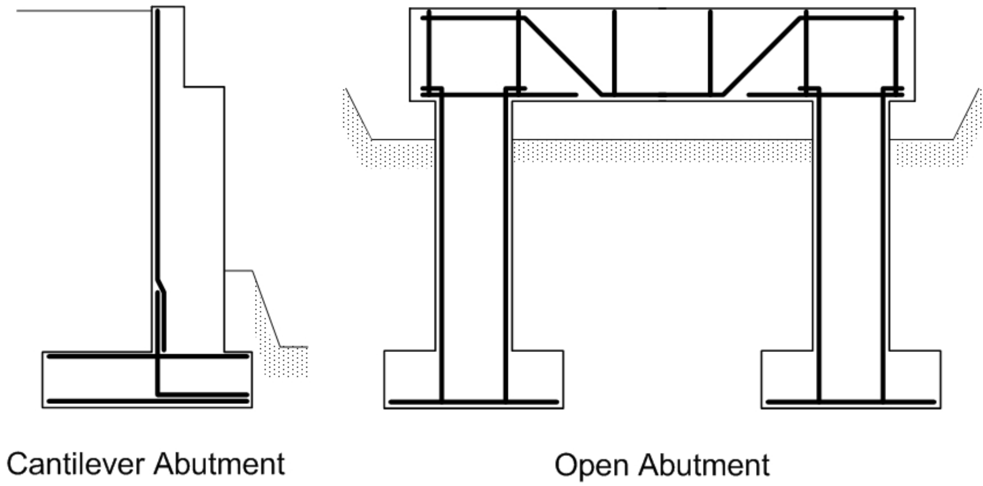


Figure 12.1.18 Primary Reinforcement in Concrete Abutments

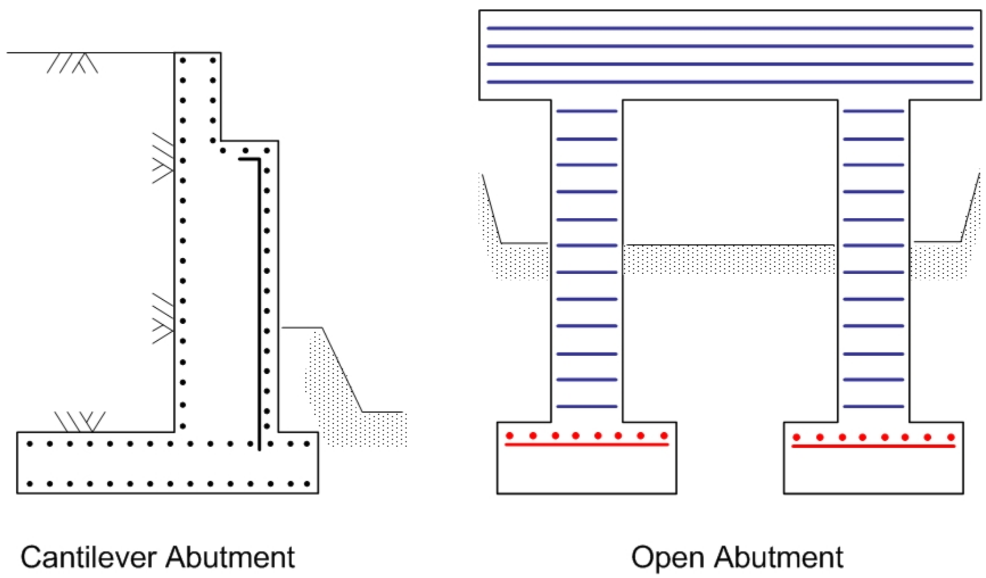


Figure 12.1.19 Secondary Reinforcement in Concrete Abutments

Abutment Members

Common abutment members include:

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

- Precast panels
- Spread footings
- Deep foundations
- Geotextiles

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

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

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

Figure 12.1.20 Cheek Wall

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

Foundation Types

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

- Spread footings
- Deep foundations

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

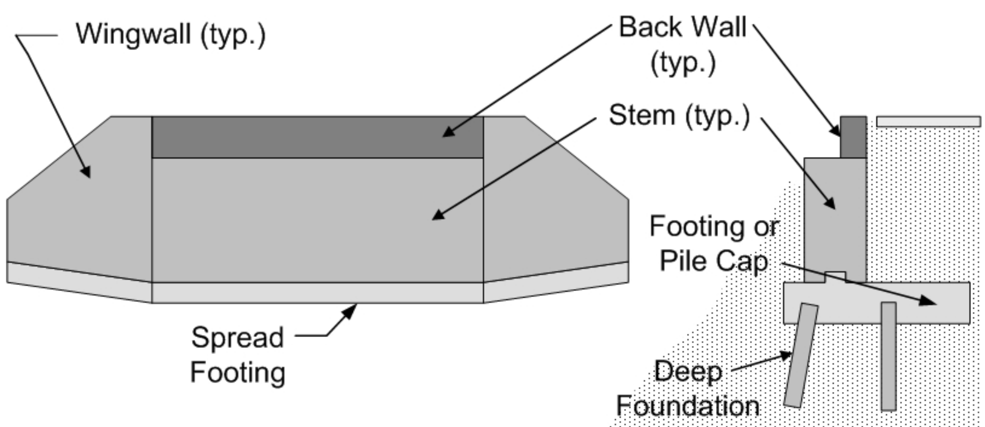


Figure 12.1.21 Spread Footing and Deep Foundations

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

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

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

Figure 12.1.22 Stub Abutment on Piles with Piles Exposed

12.1.3 Design Characteristics of Wingwalls

General

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

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

Figure 12.1.23 Typical Wingwall

Geometrical Classifications

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

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

Figure 12.1.24 Straight Wingwall

Figure 12.1.25 Flared Wingwall

Figure 12.1.26 U-wingwall

Construction Classifications

There are several construction classifications of wingwalls:

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

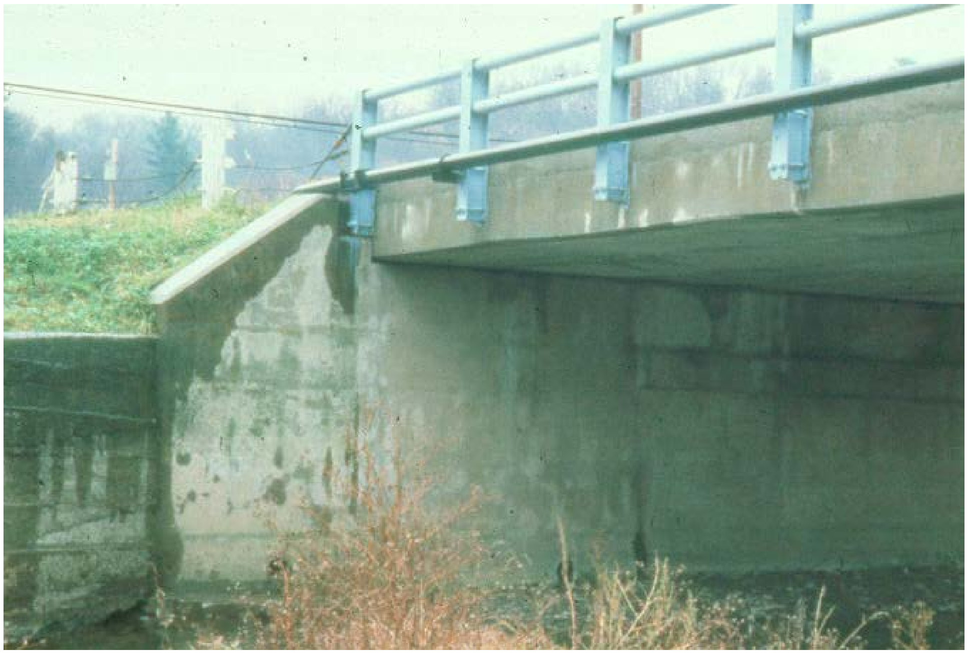


Figure 12.1.27 Integral Wingwall

Figure 12.1.28 Independent MSE Wingwall

Primary Materials

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

Figure 12.1.29 Masonry Wingwall

Primary and Secondary Reinforcement

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

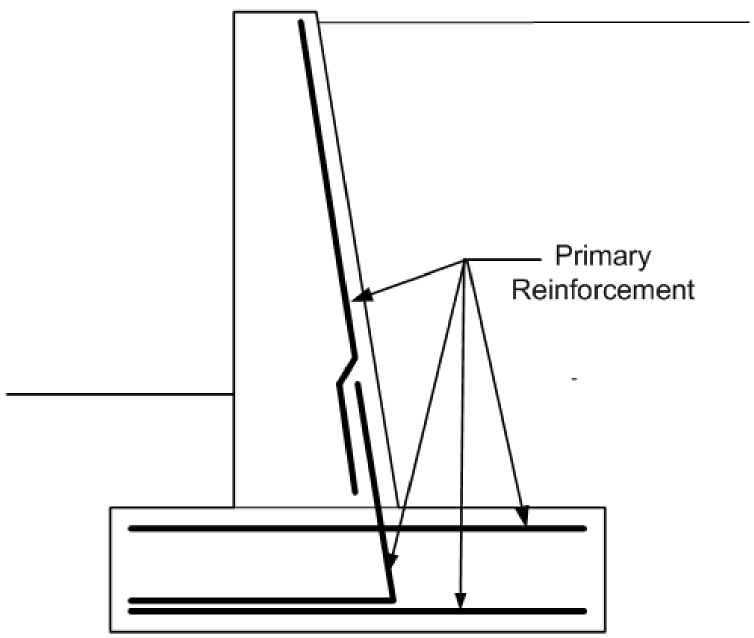


Figure 12.1.30 Primary Reinforcement in Concrete Cantilever Wingwall

12.1.4 Inspection Methods and Locations

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

Methods

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

Visual

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

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

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

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

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

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

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

Figure 12.1.31 Cracking in Bearing Seat of Concrete and Stone Abutment

Figure 12.1.32 Spalled Concrete Wingwall

Figure 12.1.33 Cracking and Efflorescence in Abutment Backwall

Figure 12.1.34 Stone Masonry Abutment with Deteriorated Joints

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

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

Figure 12.1.35 Steel Abutment

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

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

Figure 12.1.36 Decay caused by insects in Timber Abutment

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

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

Physical

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

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

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

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

Advanced Inspection Methods

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

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

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

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

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

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

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

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

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

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

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

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

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

Locations

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

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

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

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

Areas Subjected to Movement

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

- Vertical movement
- Lateral movement
- Rotational movement

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

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

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

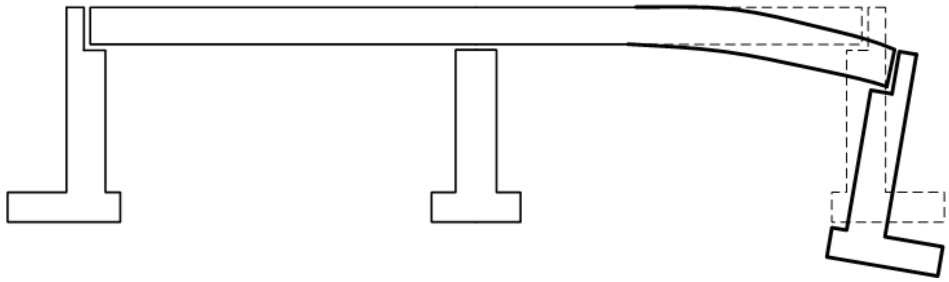


Figure 12.1.39 Differential Settlement Between Different Substructure Units

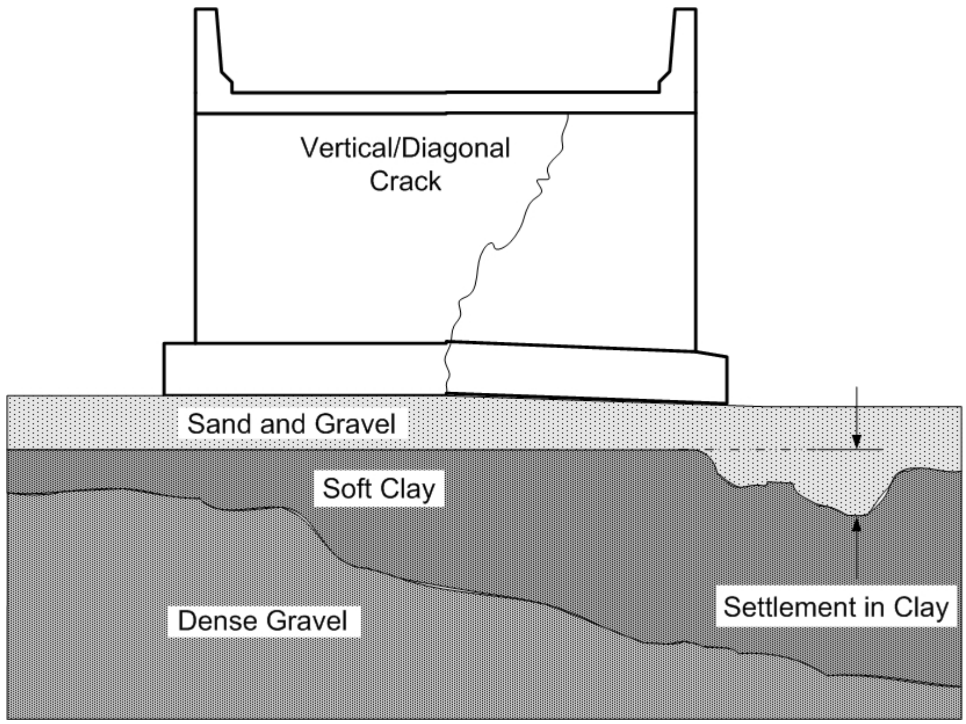


Figure 12.1.40 Differential Settlement Under an Abutment

Inspection for vertical movement, or settlement, includes:

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

Figure 12.1.41 Crack in Abutment due to Settlement

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

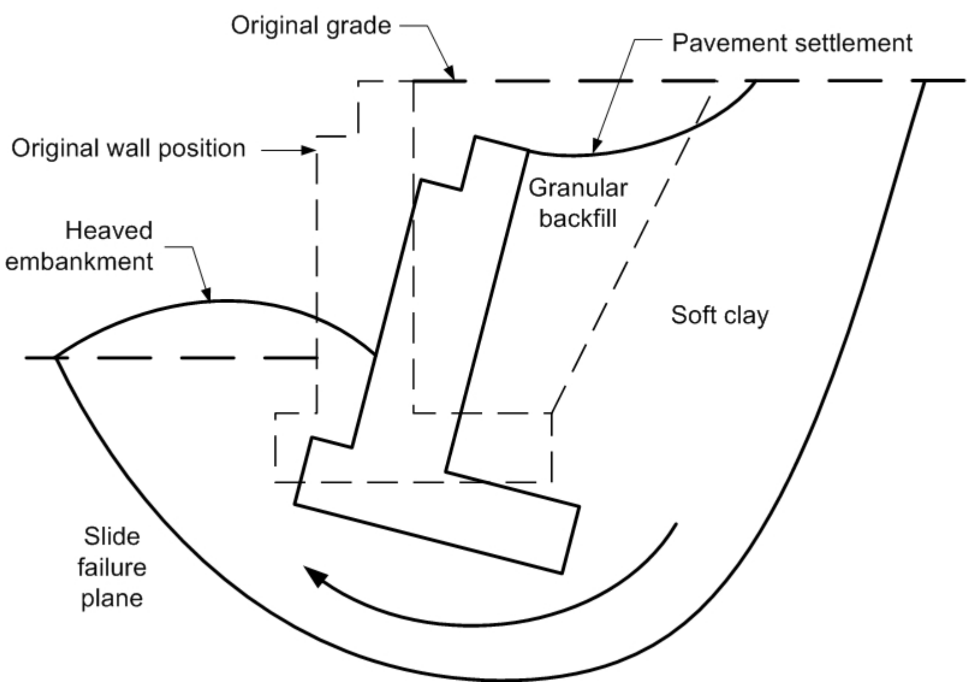


Figure 12.1.42 Lateral Movement of an Abutment due to Slope Failure

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

Inspection for lateral movement, or sliding, includes:

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

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

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

Figure 12.1.45 Erosion at Abutment Exposing Footing

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

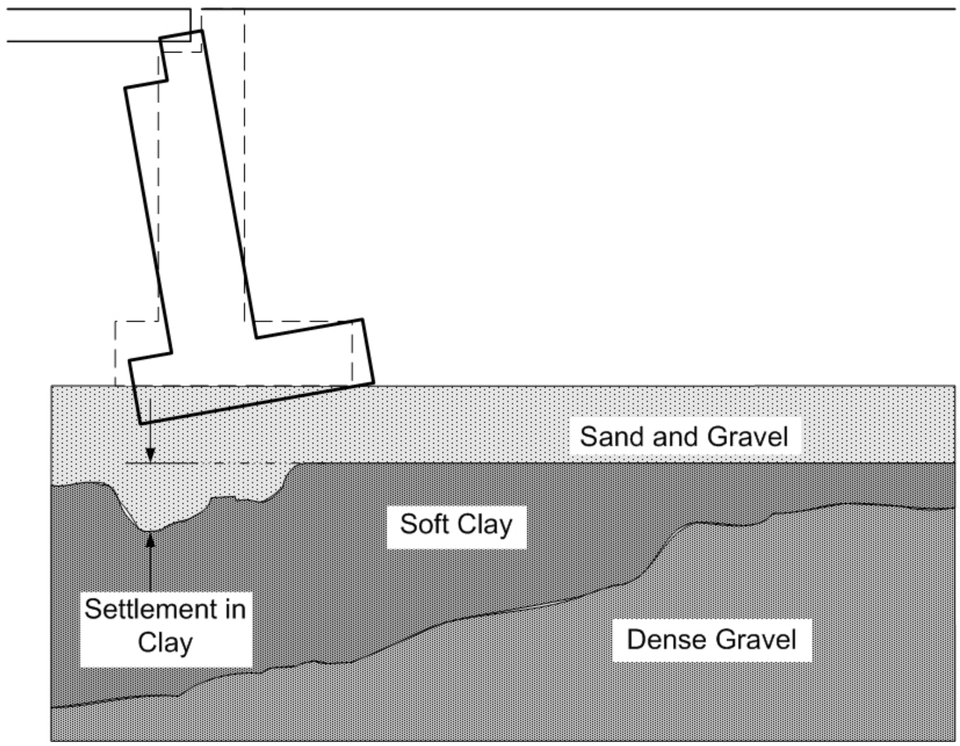


Figure 12.1.46 Rotational Movement of an Abutment

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

Inspection for rotational movement, or tipping, includes:

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

Figure 12.1.47 Rotational Movement at Abutment

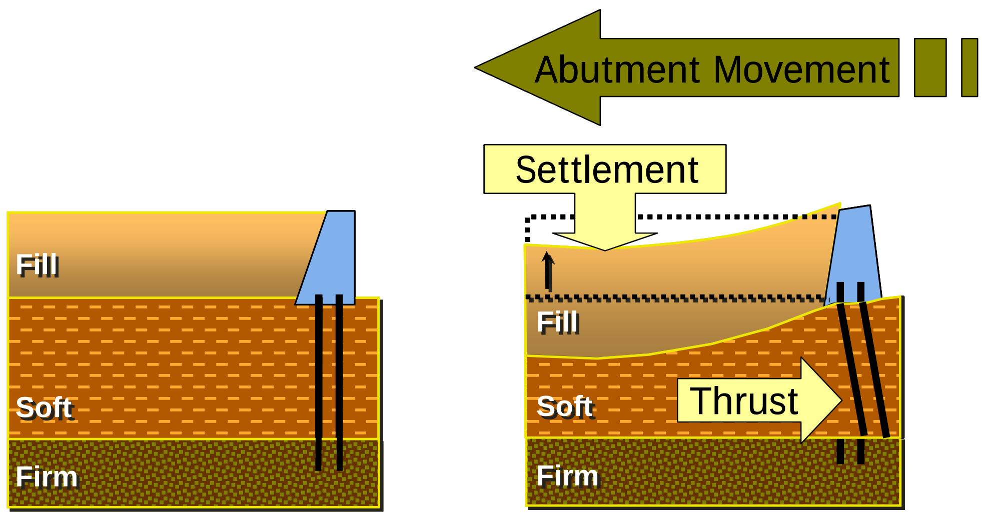


Figure 12.1.48 Rotational Movement due to “Lateral Squeeze” of Embankment Material

Figure 12.1.49 Rotational Movement at Concrete Wingwall

Bearing Areas

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

Shear Zones

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

Flexural Zones

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

Areas Exposed to Drainage

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

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

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

Areas Exposed to Traffic

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

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

Areas Previously Repaired

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

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

Scour and Undermining

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

Figure 12.1.50 Abutment with Undermining due to Scour

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

Figure 12.1.51 Inspector Checking for Scour

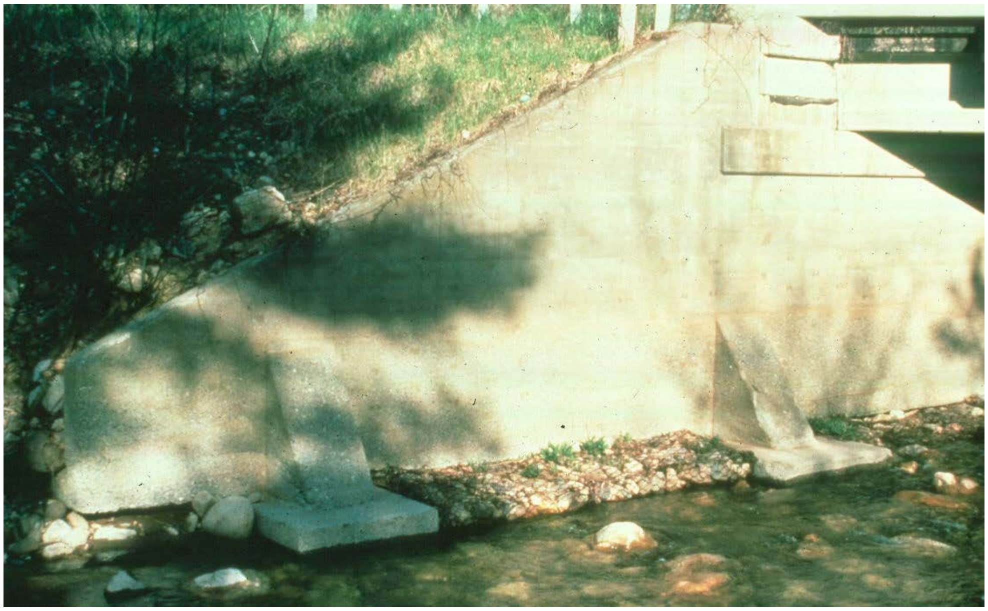


Figure 12.1.52 Scour and Possible Undermining of Concrete Wingwall

Problematic Details and Fracture Critical Members

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

12.1.5 Evaluation

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

NBI Component Condition Rating Guidelines

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

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

Element Level Condition State Assessment

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

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

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

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

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

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

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

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Chapter 12
Inspection and Evaluation of Substructures

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Topic 12.2 Piers and Bents

12.2.1 Introduction

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

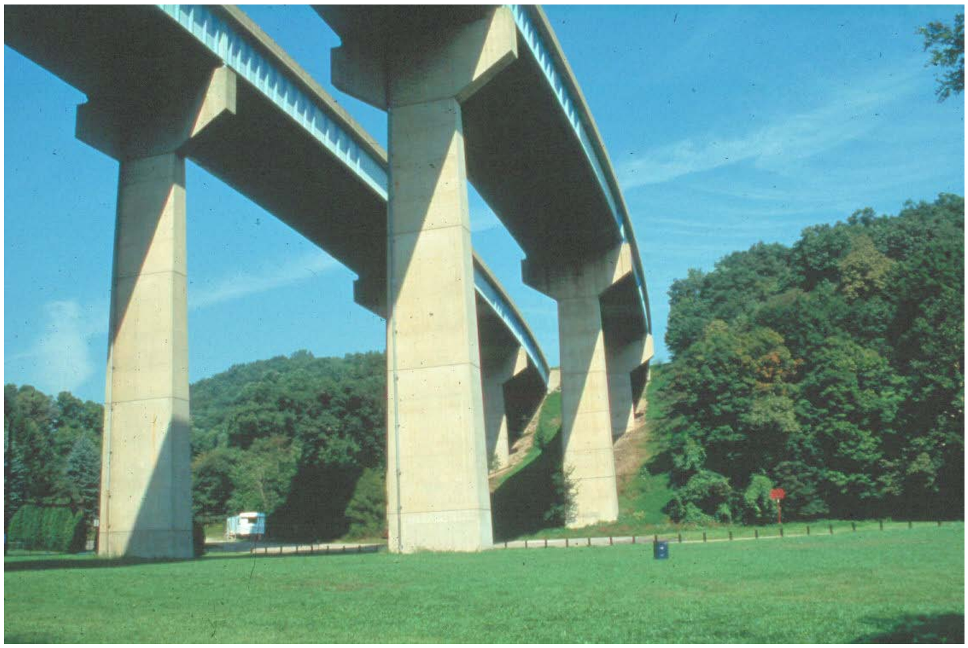

Figure 12.2.1 Example of Piers as Intermediate Supports for a Bridge

12.2.2 Design Characteristics

Pier and Bent Types

The most common pier and bent types are:

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

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

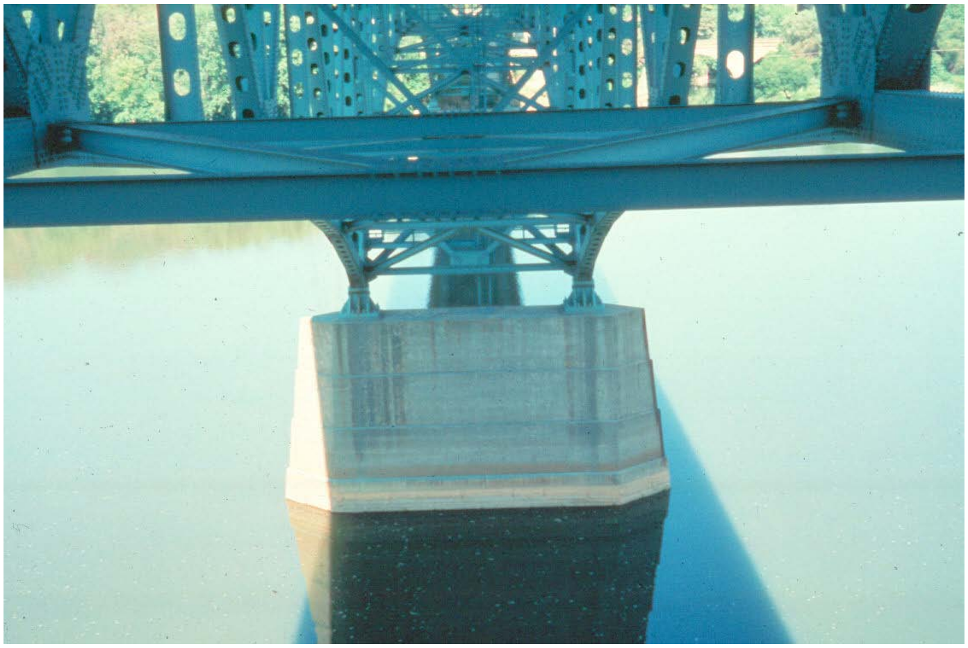

Figure 12.2.2 Solid Shaft Pier

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

Figure 12.2.3 Column Pier

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

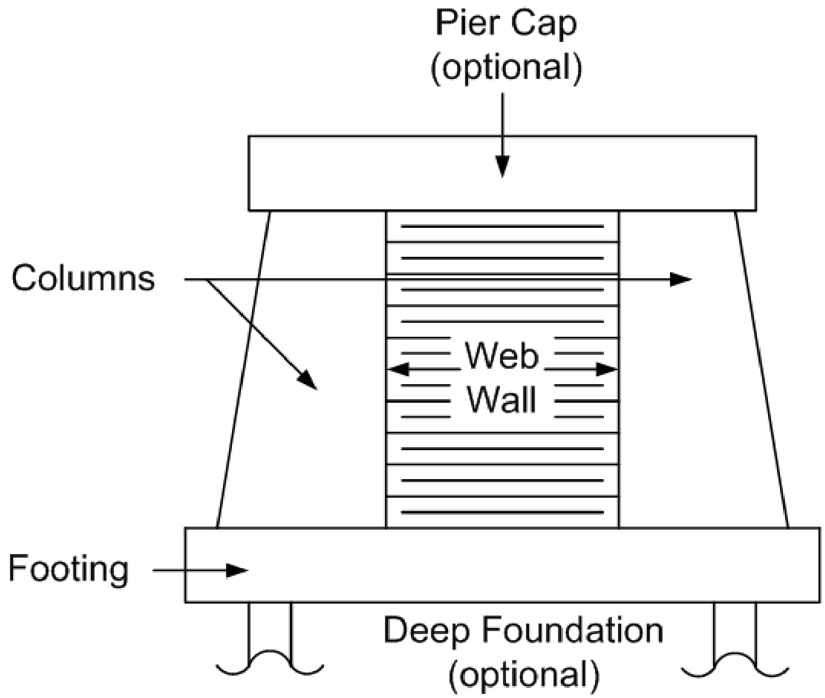

Figure 12.2.4 Column Pier with Web Wall

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

Figure 12.2.5 Column Pier with Web Wall

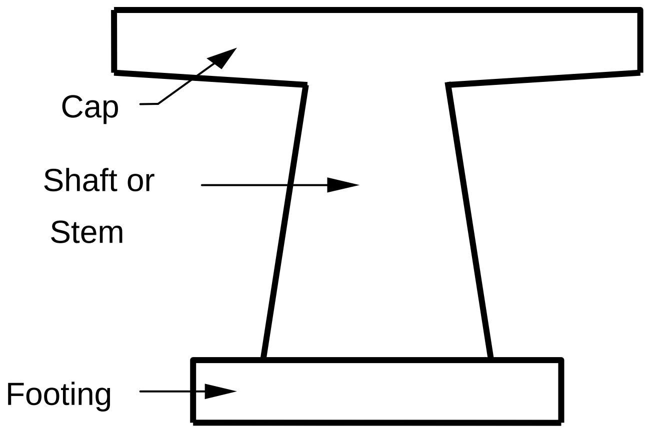

Figure 12.2.6 Single Stem Pier (Cantilever or Hammerhead)

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

Figure 12.2.7 Cantilever Pier

Figure 12.2.8 Column Bent or Open Bent

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

Figure 12.2.9 Concrete Pile Bent

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

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

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

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

Figure 12.2.10 Concrete Pier with Integral Steel Pier Cap

Figure 12.2.11 Integral Concrete Pier and Pier Cap

Figure 12.2.12 Integral Concrete Pier and Pier Cap

Primary Materials

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

Figure 12.2.13 Reinforced Concrete Piers under Construction

Figure 12.2.14 Stone Masonry Pier

Figure 12.2.15 Steel Bent

Figure 12.2.16 Timber Pile Bent

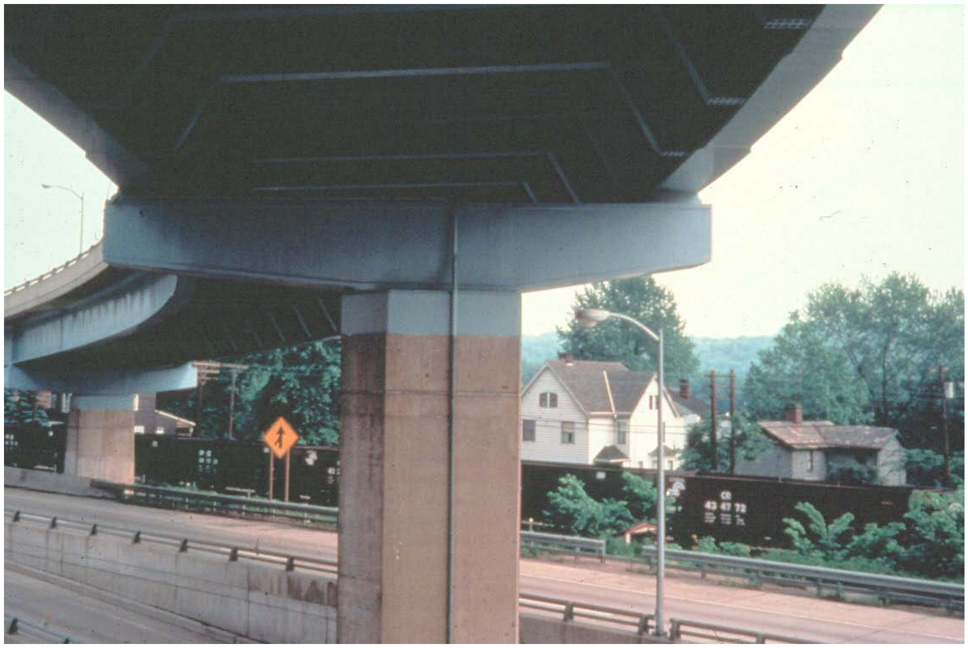

Figure 12.2.17 Combination: Reinforced Concrete Column with Steel Pier Cap

Primary and Secondary Reinforcement

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

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

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

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

Figures 12.2.18 thru 12.2.21 illustrate typical reinforcement patterns.

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

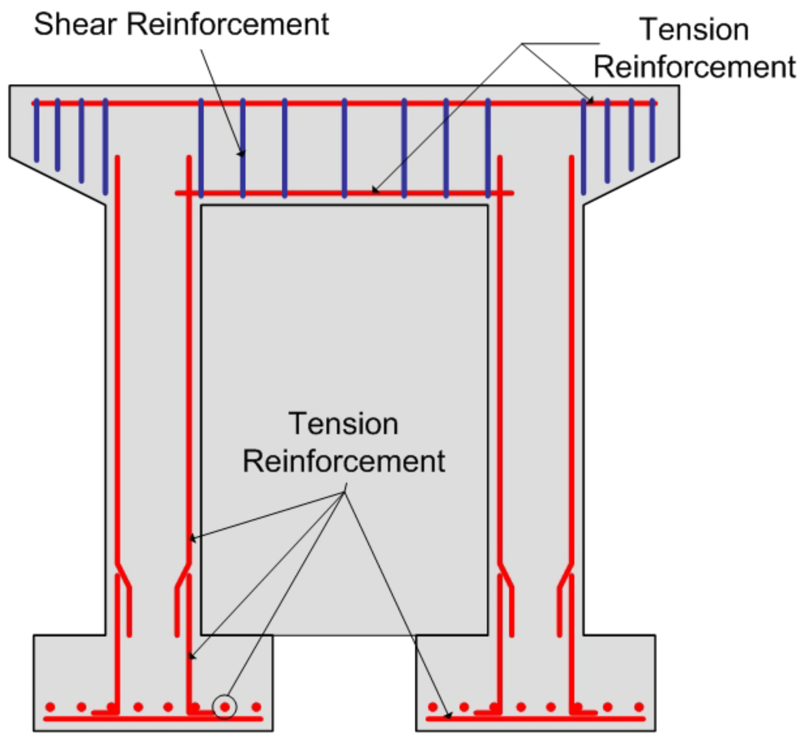

Figure 12.2.18 Primary Reinforcement in Column Bent with Web Wall

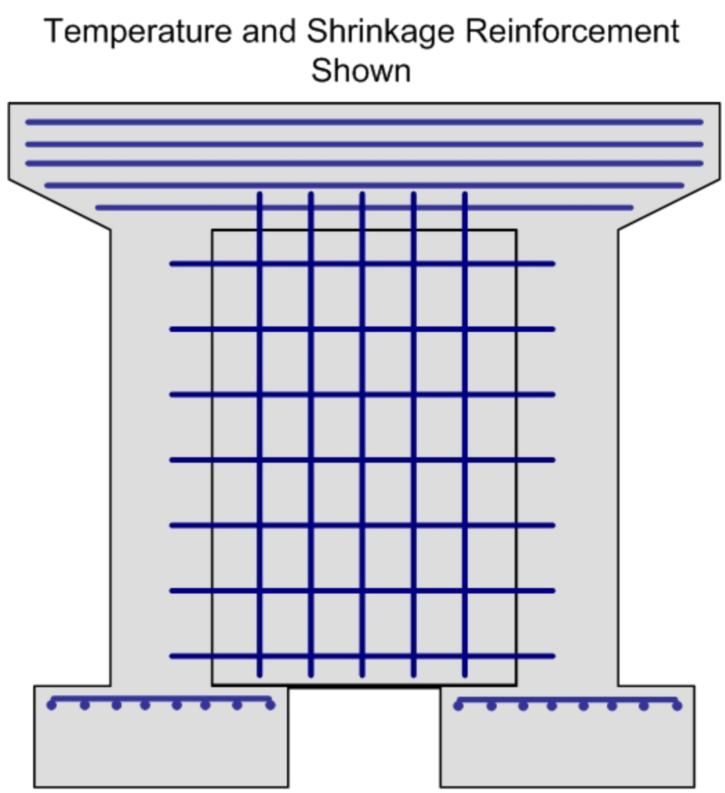

Figure 12.2.19 Secondary Reinforcement in Column Bent with Web Wall

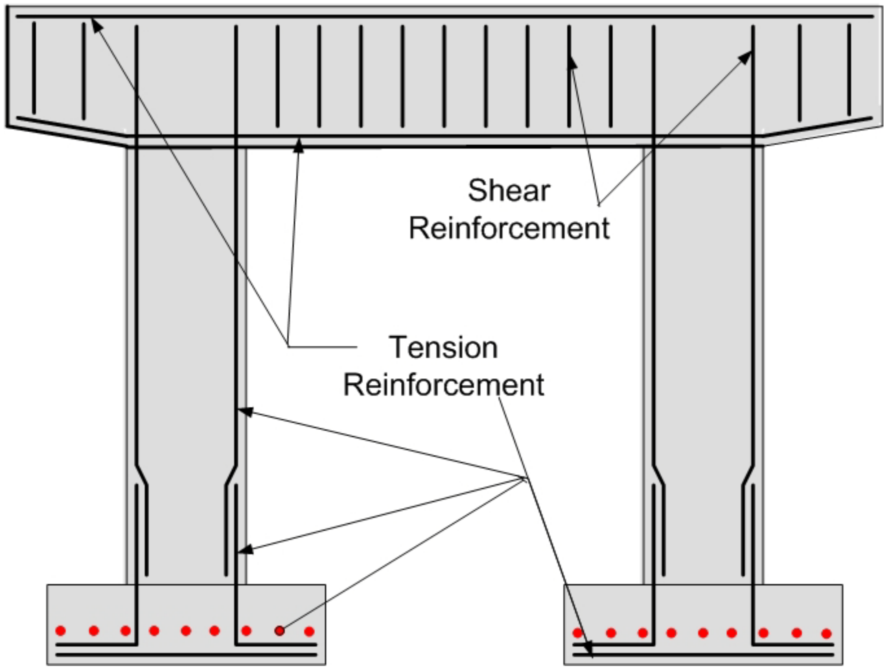

Figure 12.2.20 Primary Reinforcement in Column Bents

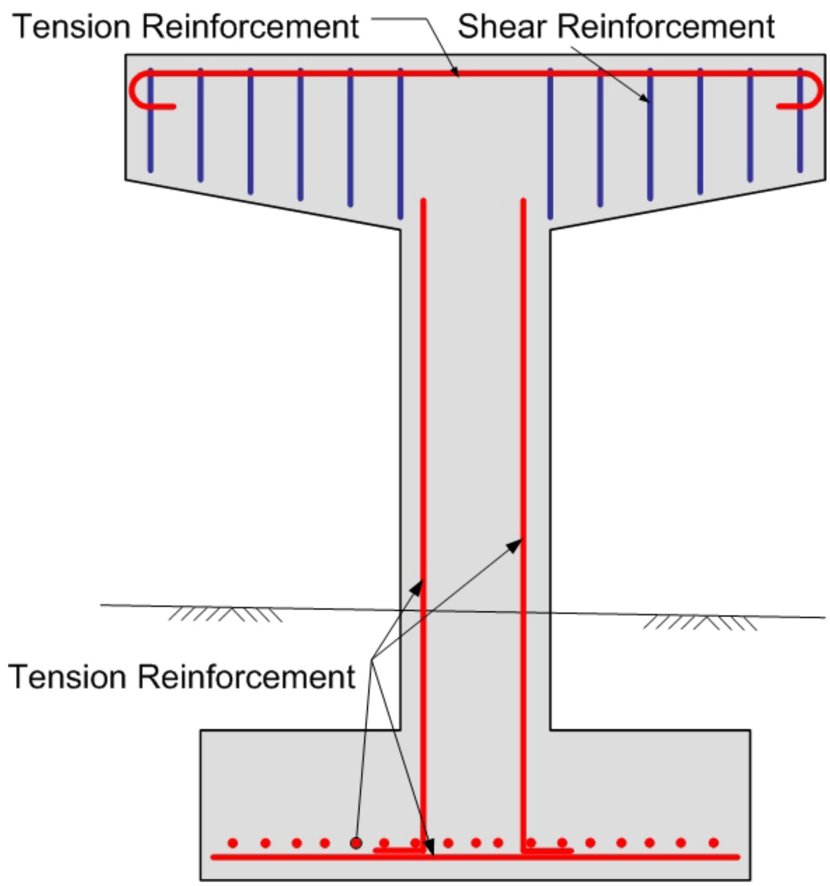

Figure 12.2.21 Primary Reinforcement for a Cantilevered Pier

Pier and Bent Members

The primary pier and bent members are:

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

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

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

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

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

Figure 12.2.22 Cantilevered Piers Joined by a Web Wall

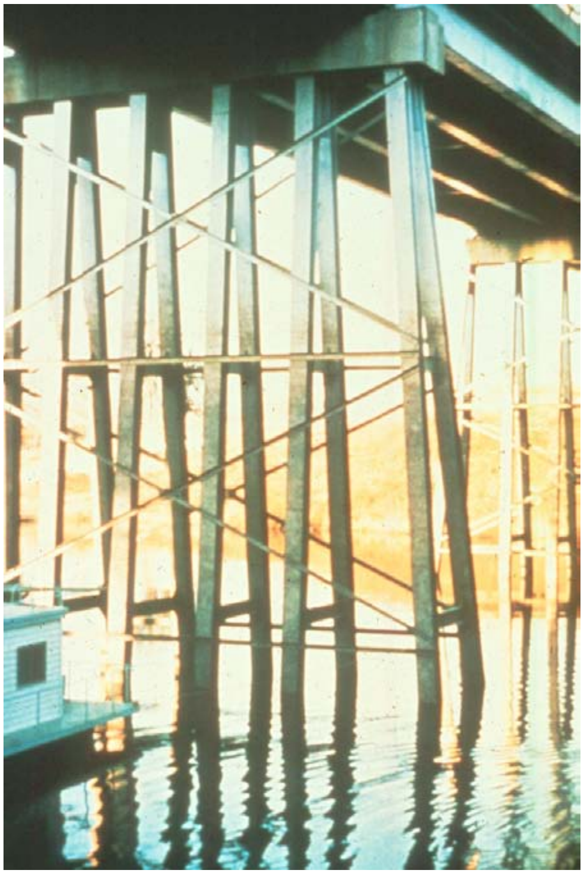

Figure 12.2.23 Pile Bent

Foundation Types

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

- Spread footings
- Deep foundations

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

Pier Protection

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

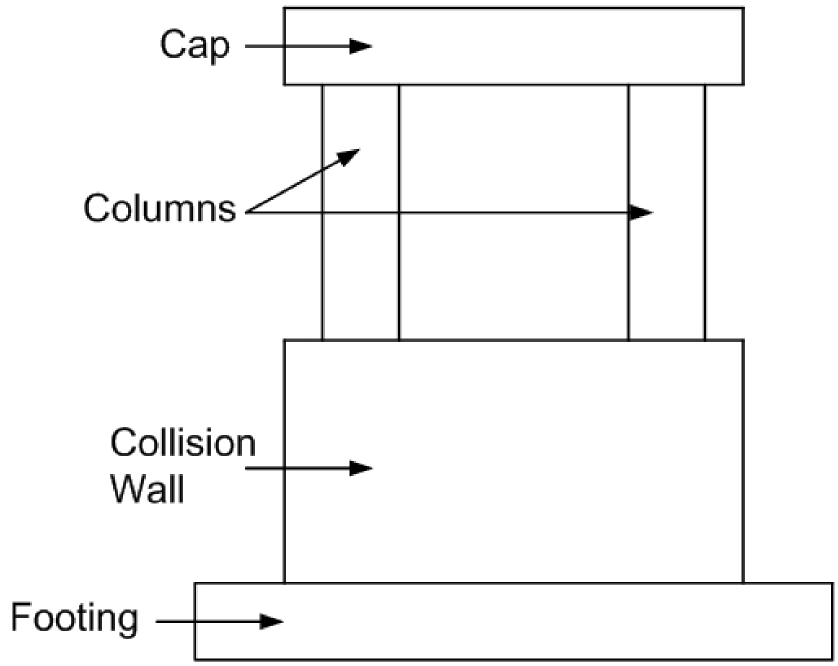

Figure 12.2.24 Collision Wall

Figure 12.2.25 Collision Wall

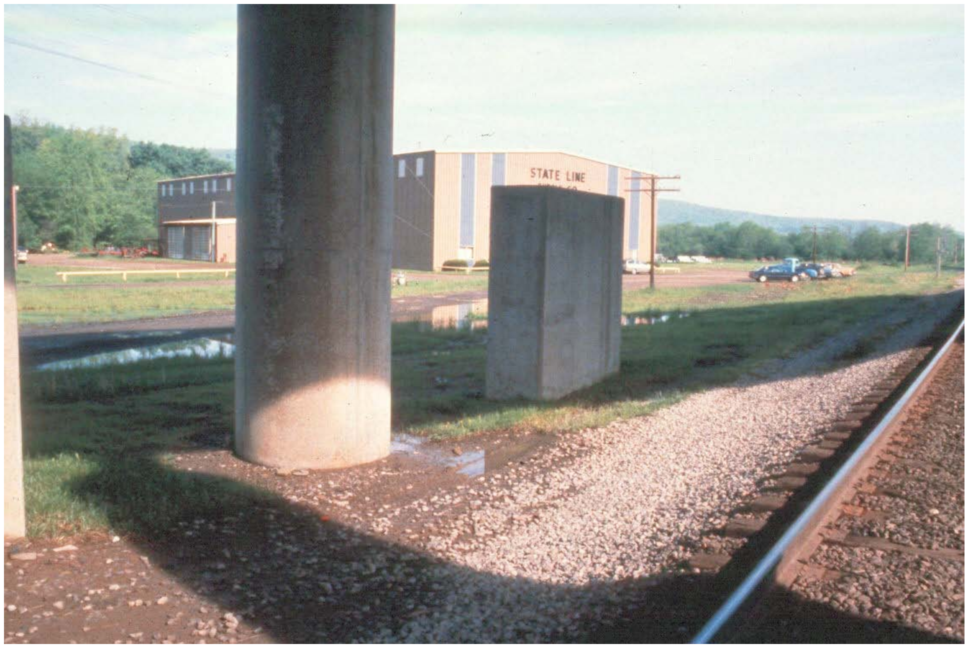

Figure 12.2.26 Concrete Block Dolphin

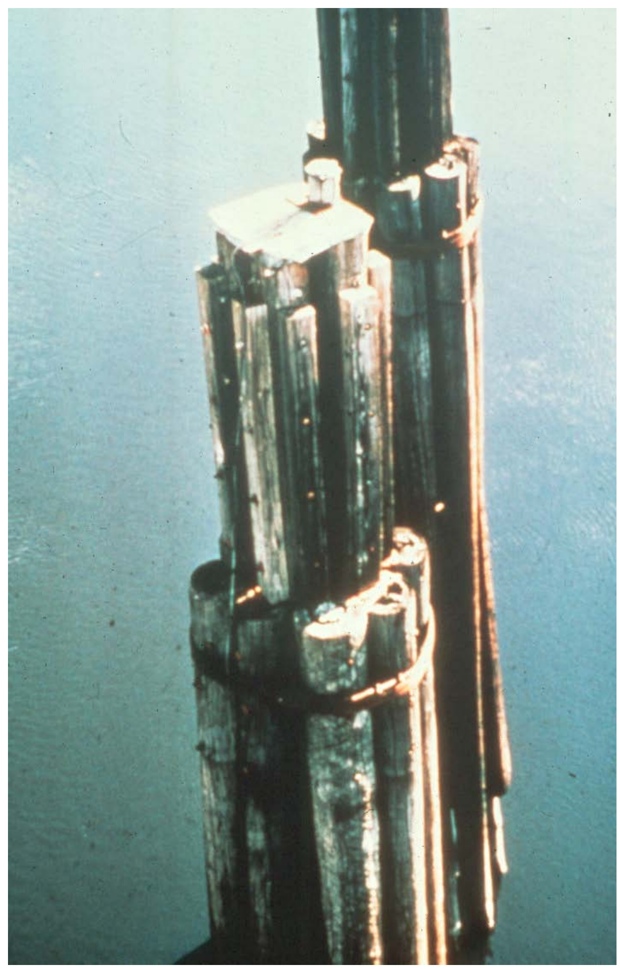

Figure 12.2.27 Timber Dolphin

Figure 12.2.28 Pier Fender

Figure 12.2.29 Fender System

12.2.3 Inspection Methods and Locations

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

Methods

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

- Visual
- Physical
- Advanced inspection methods

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

Visual

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

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

Concrete

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

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

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

Figure 12.2.30 Concrete Spalling due to Contaminated Drainage

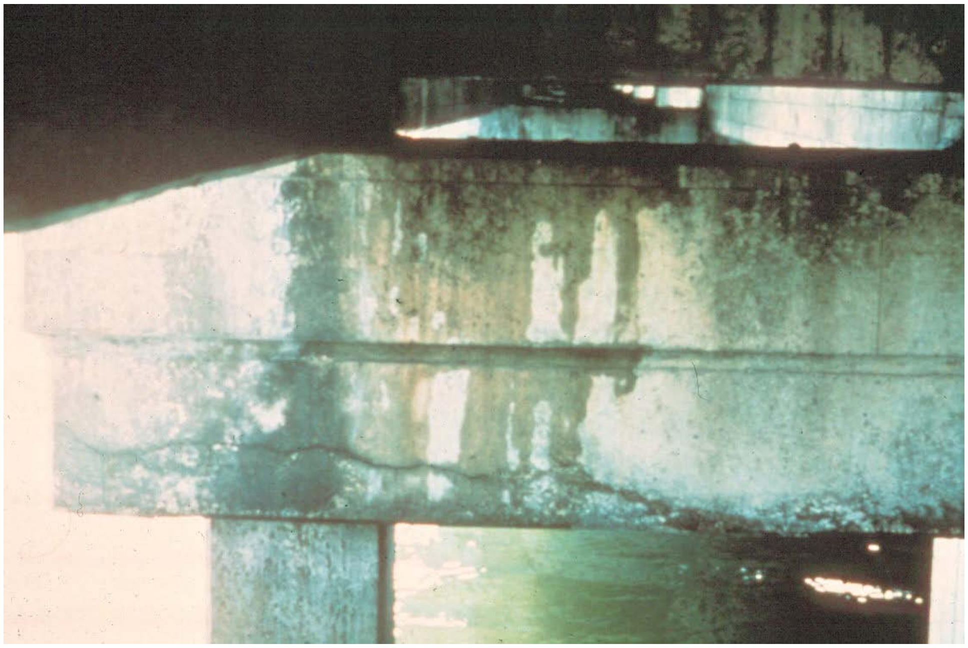

Figure 12.2.31 Crack in Concrete Bent Cap

Figure 12.2.32 Concrete Spalling on Bent Cap

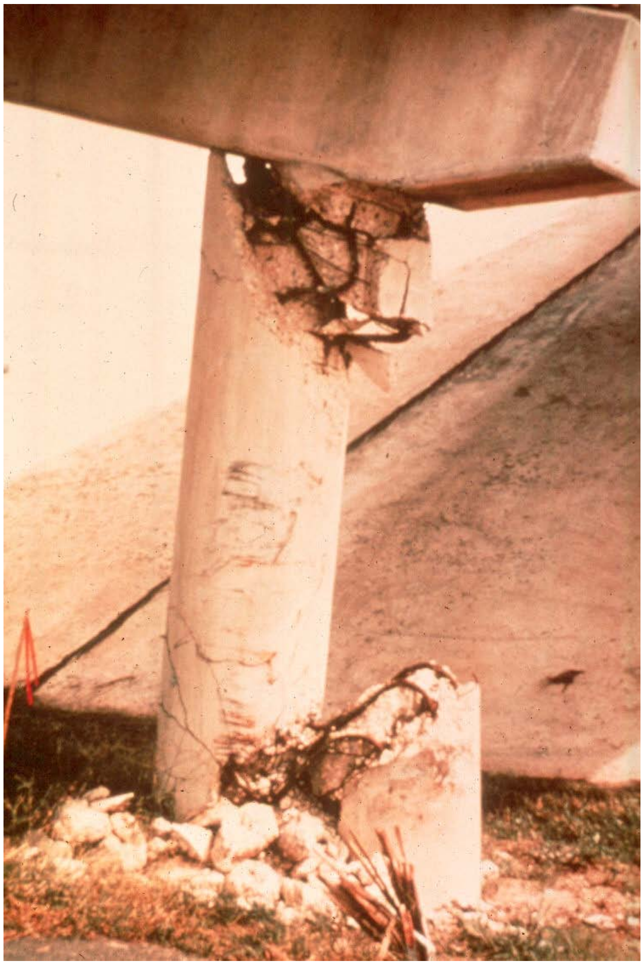

Figure 12.2.33 Collision Damage to Concrete Pier Column

Masonry

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

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

Figure 12.2.34 Deteriorated and Missing Stone at Masonry Pier

Steel

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

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

Figure 12.2.35 Deterioration of Steel Bent Leg

Figure 12.2.36 Corrosion of Steel Pile Bent at Water Surface

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

Timber

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

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

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

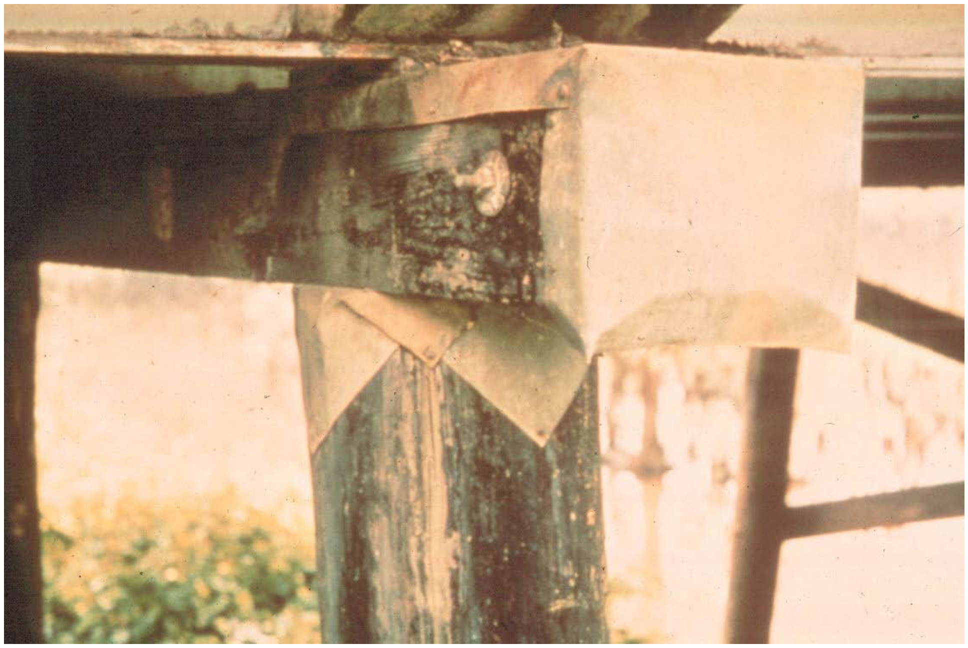

Figure 12.2.38 Decay in Timber Bent Cap (Note "Protective" Cover / Flashing)

Figure 12.2.39 Timber Bent Columns in Water

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

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

Figure 12.2.42 Timber Pile Damage due to Limnoria Marine Borers

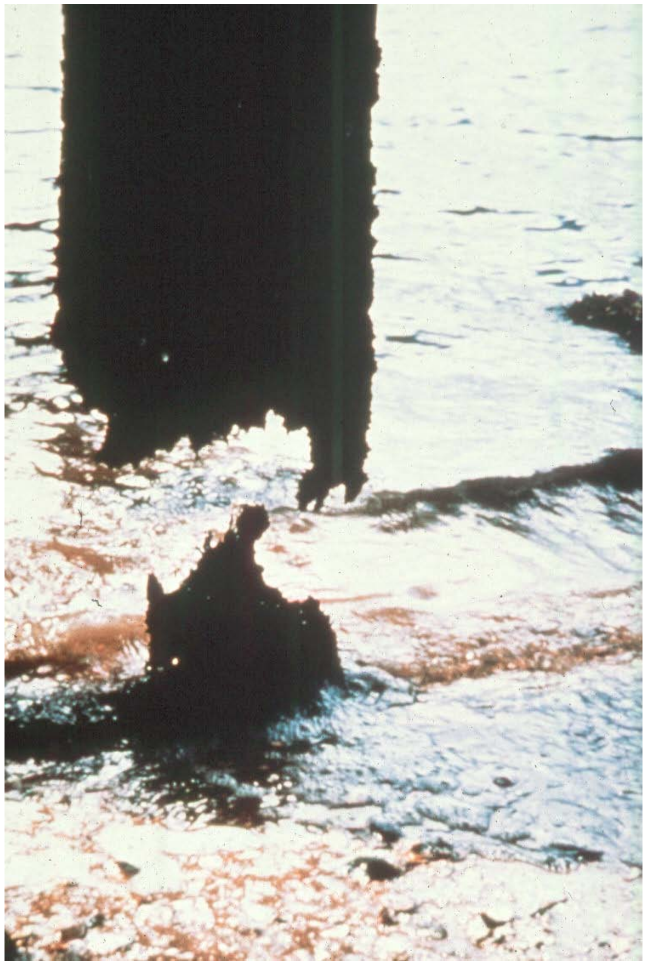

Figure 12.2.43 Timber Bent Damage due to Shipworm Marine Borers

Physical

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

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

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

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

Advanced Inspection Methods

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

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

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

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

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

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

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

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

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

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

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

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

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

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

Locations

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

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

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

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

Areas Subjected to Movement

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

- Vertical movement
- Lateral movement
- Rotational movement

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

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

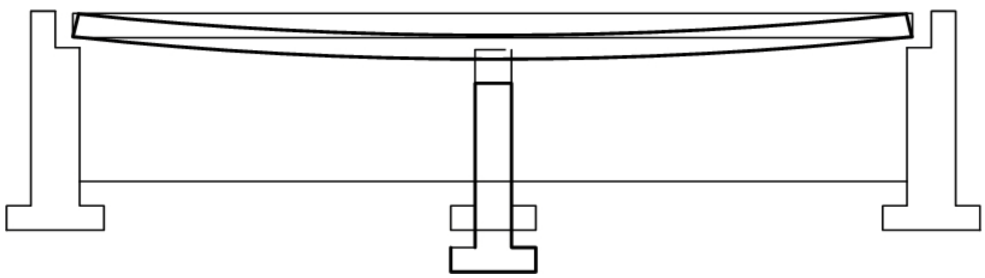

Figure 12.2.44 Differential Settlement Between Different Substructure Units

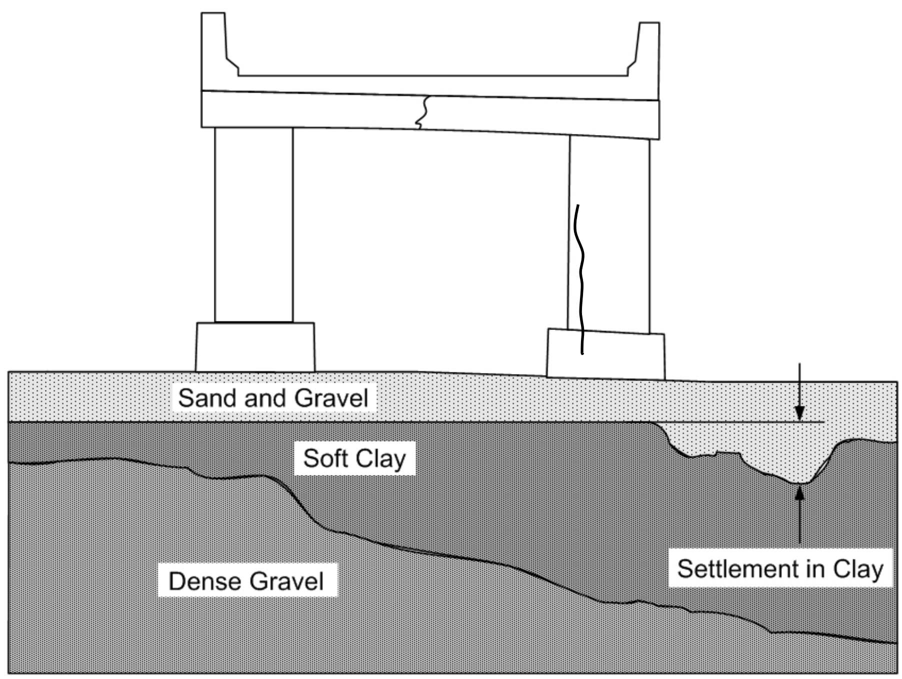

Figure 12.2.45 Differential Settlement Under a Pier

Inspection for vertical movement, or settlement, includes:

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

Figure 12.2.46 Superstructure Evidence of Pier Settlement

Inspection for lateral movement, or sliding, includes:

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

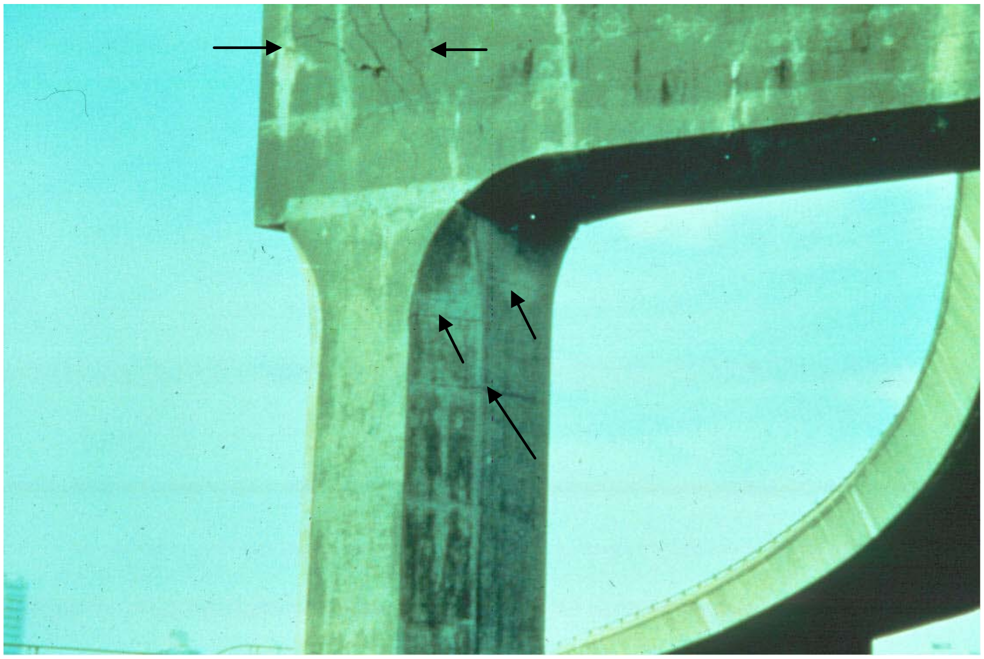

Figure 12.2.47 Cracks in Bent Cap due to Lateral Movement of Bent during Earthquake

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

Figure 12.2.49 Tipping of Bent due to Scour/Undermining

Inspection for rotational movement, or tipping, includes:

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

Bearing Areas

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

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

Shear Zones

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

Flexural Zones

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

Areas Exposed to Drainage

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

Areas Exposed to Traffic

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

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

Areas Previously Repaired

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

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

Figure 12.2.50 Repaired Concrete Column Bent

Scour and Undermining

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

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

Problematic Details and Fracture Critical Members

Steel piers or bents may contain problematic or fatigue prone details. Closely examine these details for section loss due to corrosion and cracking.

Steel piers or bents may be considered to be fracture critical (see Figure 12.2.51). See Topic 6.4 for a detailed description of details and fracture critical members.

Figure 12.2.51 Fracture Critical Steel Bent

Dolphins and Fenders

The condition of dolphins and fenders are checked in a manner similar to that used for inspecting the main substructure elements.

In concrete pier protection members, check for spalling and cracking of concrete or corrosion of the reinforcing steel (see Figure 12.2.52). Investigate for hour-glass shaping of piles due to abrasion at the waterline, and check for structural damage caused by marine traffic.

In steel pier protection members, observe the splash zone (up to two feet above high tide or mean water level) carefully for corrosion. Where there are no tides, check the area from the mean water level to two feet above it. Examine steel members for corrosion, and check for structural damage (see Figure 12.2.53).

In timber pier protection members, observe the portions between the high waterline and the mud line for marine borers, caddisflies, and decay, and check for structural damage (see Figure 12.2.54). Check for hourglass shaping of piles at the waterline.

Figure 12.2.52 Concrete Dolphins

Figure 12.2.53 Steel Fender

Figure 12.2.54 Timber Fender System with Deteriorated Piles

12.2.4 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of substructures. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment method.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the entire substructure including abutments and piers. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 60) for additional details about NBI component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating. Recognize that piers may be affected by scour or other conditions that may only be able to be accessed and evaluated by a separate underwater inspection. Therefore, the results of both the routine and underwater inspection, if applicable, are integrated and evaluated together to arrive at the correct component condition rating for the substructure. Note the findings of the underwater inspection in the narrative portion of the routine inspection report as documentation and justification for the determined substructure component condition rating code.

Element Level Condition State Assessment

In an element level condition state assessment of a pier or bent structure, possible AASHTO Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Substructure	
202	Steel Column/Pile Extension
207	Steel Column Tower (Trestle)
225	Steel Submerged Pile
231	Steel Pier Cap
204	Prestressed Concrete Column/Pile Extension
226	Prestressed Concrete Submerged Pile
233	Prestressed Concrete Pier Cap
205	Reinforced Concrete Column/Pile Extension
210	Reinforced Concrete Pier Wall
220	Reinforced Concrete Pile Cap/Footing
227	Reinforced Concrete Submerged Pile
234	Reinforced Concrete Pier Cap
206	Timber Column/Pile Extension
208	Timber Column Tower (Trestle)
228	Timber Submerged Pile
212	Timber Pier Wall
235	Timber Pier Cap
213	Masonry Pier Wall
211	Other Pier Wall

BME No.	Description
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating
521	Concrete Protective Coating

The unit quantity for the pier cap elements is feet, measured horizontally across the pier cap. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for columns and piles is each, and the total quantity is placed in one of the available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coatings is square feet, with the total area distributed among the four available conditions states depending on the extent and severity of the deficiency. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of the piers and bents..

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
358	Concrete Cracking
359	Concrete Efflorescence
360	Settlement
361	Scour
363	Steel Section Loss
364	Steel Out-of-Plane (Compression Members)
367	Substructure Traffic Impact (load capacity)

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

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Inspection and Evaluation of Waterways

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Chapter 13
Inspection and Evaluation of Waterways

Topic 13.1 Waterway Elements

13.1.1 Introduction

Rivers are the most dynamic geomorphic system that engineers have to cope with in the design and maintenance of bridges. The geomorphic features of the river can change dramatically with time. During major floods, significant changes can occur in a short period of time. While rivers are dynamic or can move locations, bridges do not move locations.

There are several ways in which channels can change and thereby jeopardize the stability and safety of bridges. The channel bed can scour (degrade) so that bed elevations become lower, undermining the foundation of the piers and abutments. Deposition of sediment on the channel bed (aggradation) can reduce conveyance capacity through the bridge opening. Flood waters are then forced around the bridge, attacking roadway approaches, channel banks, and flood plains. Another consequence of aggradation is that the river stage may be increased to where it exerts lateral thrust and lift on the deck and girders of the bridge (see Figures 13.1.1 and 13.1.2). The other primary way in which bridges can be adversely affected by a waterway is through bank erosion or avulsion, causing the channel to shift laterally. These phenomena of aggradation, degradation or scour, bank erosion, and lateral migration can be a result of natural or induced causes and can adversely affect the bridge (see Figure 13.1.3). Topic 13.2 presents detailed descriptions of waterway deficiencies.

Of all the bridges in the National Bridge Inventory (NBI), approximately 86% are built over waterways. Bridge inspectors need to understand the relationship between the bridge and waterway elements. This understanding involves being able to recognize and identify the streambed, embankments, floodplain, and streamflow so that an accurate assessment and record of the present condition of the bridge and waterway can be determined.

Figure 13.1.1 Failure Due to High Water Levels During Hurricane: Aerial View

Figure 13.1.2 Failure Due to High Water Levels During Hurricane: Close-Up View

Figure 13.1.3 Pier Foundation Failure

13.1.2 Properties Affecting Waterways

Safety is a major concern in the inspection of bridges over active waterways. Various properties can affect waterways and structures.

- The size, shape and orientation of the bridge superstructure and foundation units.
- The physical characteristics such as channel sinuosity, slope, streambed and bank material classification and bank geometry and vegetative cover.
- The geomorphic history of the waterway (history of changes in the location, shape, and elevation of the channel).
- The hydraulic forces imposed on the bridge by the streamflow.
- Changes in the river channel or flow due to development projects (such as dams, diversions, urbanization and channel stabilization) or natural phenomena.
- The condition of hydraulic control structures that have been utilized to help protect the bridge and adjacent channel.
- Changes in the sediment balance in the stream due to nearby streambed gravel mining or landslides.

13.1.3 Purpose of Waterway Inspections

There are three major purposes for conducting waterway inspections.

- Identify critical damage
- Record existing channel conditions
- Monitor channel changes

Identify Critical Damage

Waterway inspections are needed to identify conditions that cause structural collapse of bridge structures. Deficient piling along with damage or deterioration to foundation members can only be detected during a waterway inspection. Entering the water and probing around the foundations is necessary to detect loss of foundation support.

Record Existing Channel Conditions

Waterway inspections are conducted to create a record of the existing channel conditions adjacent to the bridge. Conditions such as channel opening width, depth at substructure elements, channel cross-section elevations, water flow velocity, and channel constriction and skew are noted and compared to previously recorded conditions.

Accessing the waterway to measure and record channel conditions may be restricted by several factors including channel width and depth, flow velocity, or pollution. These factors may require the bridge inspector to return to the site during a period of low flow. Alternatively the inspector may need to consider using an alternate means of waterway access, such as a boat, or an alternative inspection technique, such as underwater diving inspection.

Monitor Channel Changes

Current waterway inspection data should be compared to previous inspection data in order to identify channel changes. This "tracking" of channel change over time is an important step in ensuring the safety of the bridge. Over time, vertical changes, due to either degradation or aggradation processes, or horizontal alignment changes, due to lateral migration of the channel, could result in foundation undermining, bridge overtopping, or even collapse of the structure. If major changes are found, a formal scour analysis of the site, involving a multi-disciplinary team of engineers, may be needed to estimate floodwater elevations, velocities, angle of attack, and potential scour depths. Potential threats to bridge members caused by channel changes can thus be dealt with before damage actually occurs. See Topic 13.2 for the inspection and evaluation of waterways.

13.1.4 Channel Characteristics

According to the Hydraulic Design Series Number 6 (HDS-6) Highways in the River Environment, channels are typically well-defined and confine the streamflow during normal flow conditions (see Figure 13.1.4).

Elements of a Channel

- Streambed - the bottom or floor of the channel.
- Streambank - the sloped sides of the channel, which extend from the streambed to the surrounding ground elevation (floodplain).
- Streamflow - the water, suspended sediment, and any debris moving through the channel.
- Thalweg elevation – lowest elevation of the stream.

Elements of floodplains are presented in Topic 13.1.5.

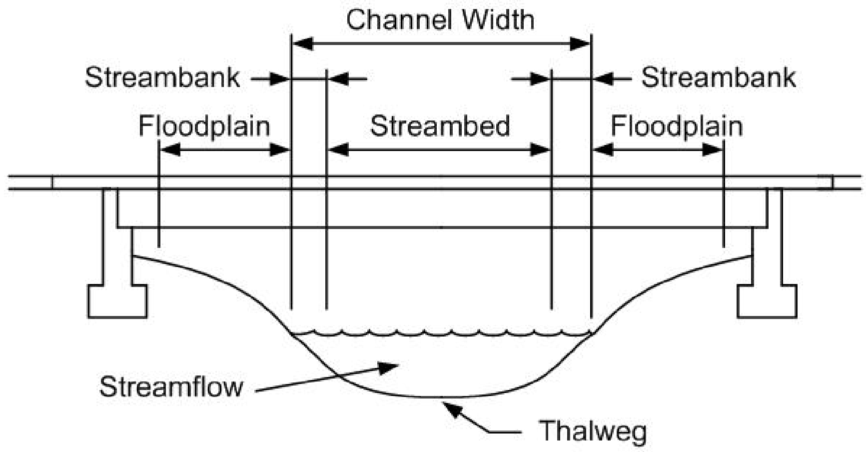

Figure 13.1.4 Typical Waterway Cross Section Showing Well Defined Channel Depression

Types of Channels

Knowledge of the type and profile of a waterway or river channel is essential to understand the hydraulics of the channel and its potential for change. The type of river may dictate certain tendencies or responses that may be more adverse than others. To aid in this understanding, various key river classes are briefly explained. Rivers can be broadly classified into four categories:

- Meandering rivers
- Braided rivers
- Straight rivers
- Steep mountain streams

Meandering Rivers

Meandering rivers consist of a series of bends connected by crossings. In general, pools exist in the bends. The dimensions of these pools vary with the size of the river, flow conditions, radius of the curvature of the bends, and type of bed and bank material. Such rivers are fairly predictable and experience relatively slow velocities. Figure 13.1.5 shows some differences between the various river categories. Figure 13.1.6 illustrates the major characteristics of a meandering river.

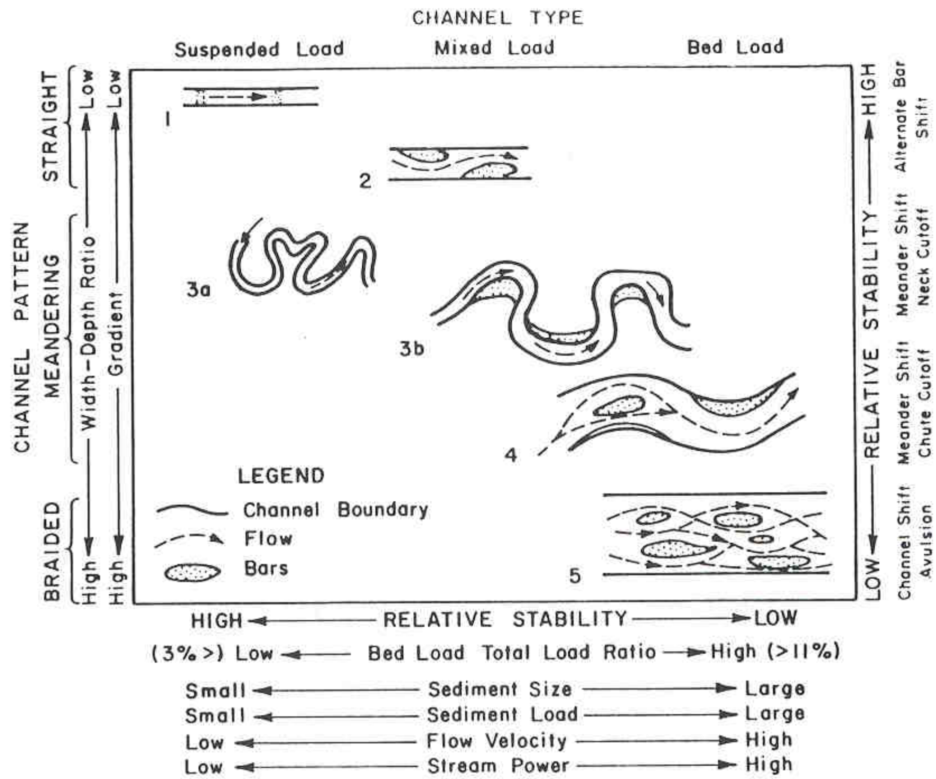

Figure 13.1.5 Plan View of Rivers

Braided Rivers

Braided rivers consist of multiple channels that are intertwined in braided form. At flood stages, the appearance of braiding is less noticeable. The bars dividing the multiple channels may become submerged, and the river will appear to be relatively straight. Braided rivers have steeper slopes and experience higher streamflow velocities which may cause larger scour or undermining problems.

Braided rivers can change rapidly, causing different velocity distributions, partial blockages of portions of the waterway beneath bridges, and larger quantities of debris that can be a hazard to bridges and cause accelerated scour. Figure 13.1.4 illustrates the plan view of typical rivers, including meandering, straight, and braided. This figure also relates form of river to channel type based on sediment load and relative stability of river type.

Straight Rivers

Straight rivers are something of an anomaly. Most straight rivers are in a transition between meandering and braided types. In straight rivers, any development that would flatten the gradient would accelerate change from a straight system to a meandering system. Conversely, if the gradient were increased, the channel may become braided. Therefore, in order to maintain the straight alignment over a normal range of hydrologic conditions, it may become necessary to utilize channel hydraulic control structures (Topic 13.1.7). The characteristics of straight rivers are identified in Figure 13.1.5.

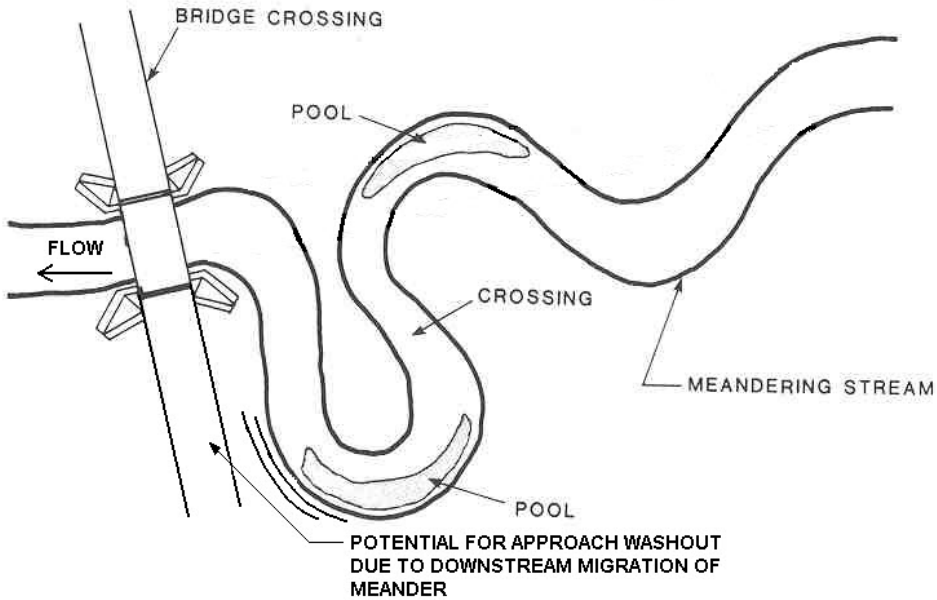

Figure 13.1.6 Meandering River

Steep Mountain Streams

Steep mountain streams are controlled by geologic formations, rock falls, and waterfalls. They experience very small changes in either plan form or profile when subjected to the normal range of discharges. The bed material of such river systems can consist of gravel, cobbles, boulders, or some mixture of these different sizes. Even though these rivers are relatively stable, they can experience significant velocity and flow changes during episodic flood events.

13.1.5 Floodplain Characteristics

The floodplain is the overbank area outside the channel that carries flood flows in excess of channel capacity (see Figure 13.1.7). It is common to find bridges built within the floodplain. For many structures, the floodplain is quite large, as compared to the channel. Observations made during periods of high water can help the inspector identify the floodplain.

Elements of a Floodplain

- Freeboard – the vertical distance between the design flood water surface and the lowest point of the superstructure to account for waves, surges, drift and other contingencies (see Figure 13.1.5)
- Normal stage – the streamflow stage prevailing during the greater part of the year (between low and high water levels)
- Waterway area – the entire are beneath the bridge which is available to pass flood flows.

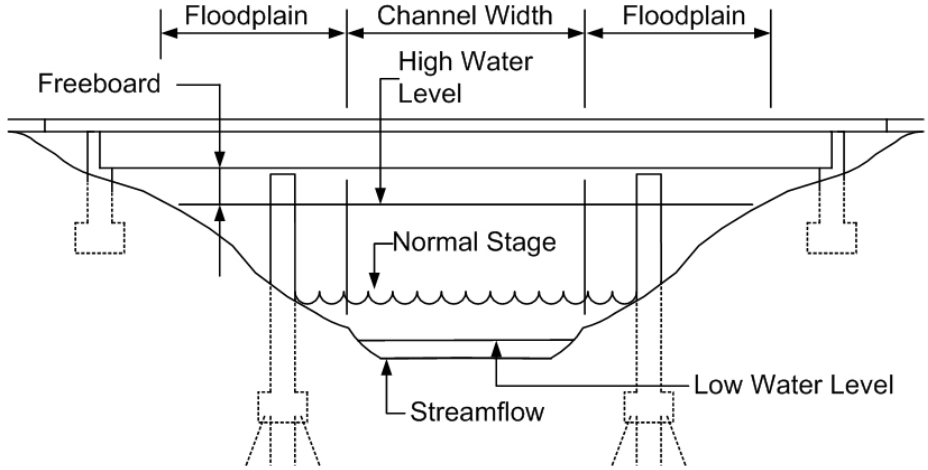

Figure 13.1.7 Typical Floodplain

Channel characteristics are presented in detail in Topic 13.1.4.

13.1.6 Hydraulic Opening

The hydraulic opening is the entire area beneath the bridge which is available to pass flood flows (see Figure 13.1.8). The bottom of the superstructure, the two bridge abutments, and the streambed or ground elevation bounds the hydraulic, or waterway, opening. For multiple spans, intermediate supports such as piers or bents restrict the hydraulic or bridge waterway opening.

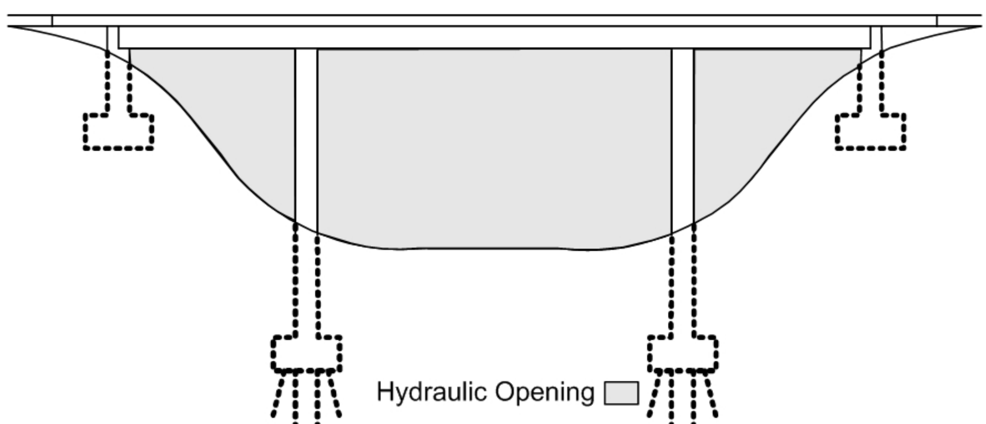

Figure 13.1.8 Hydraulic Waterway Opening

13.1.7 Hydraulic Countermeasures

Hydraulic countermeasures are often utilized to provide protection for bridges against lateral migration of the channel and against high velocity flows and scour. A hydraulic countermeasure is a man-made or man-placed device designed to direct streamflow and protect against lateral migration or scour. These flow hydraulic control countermeasures may be utilized either at the bridge, upstream from the bridge, or downstream from the bridge. Countermeasures are designed by hydraulic and geotechnical engineers and are installed to redirect streamflow and flood flows within the watercourse and through the bridge waterway opening. Hydraulic countermeasures are broken into two distinct categories which are river training structures and armoring countermeasures.

River Control Structures

River control structures are countermeasures designed to modify the flow to help prevent. A couple examples of river training structures are spurs and guide banks. A complete list of the various types of river training structures is located in HEC-23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, 3rd edition.

Spurs

Spurs are linear structures, designed with properly sized and placed rocks, that projects into a channel and placed on the outside bends of the bank to protect the streambank by reducing flow velocity, inducing deposition of sediment or redirecting the flow (see Figure 13.1.10). Common applications occur on meandering streams where they are placed on the outside of the bends to redirect the flow and minimize lateral stream migration.

Guide banks

Guide banks are dikes which extend upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening (see Figure 13.1.11). Scour hole formation occurs at the upstream ends of the guide banks if left unprotected. Common scour prevention devices for guide banks include riprap.

Armoring Countermeasures

Armoring countermeasures tend not to alter the flow significantly, but are design to resist hydraulic stresses of the design flood events. Some examples of armor countermeasures include riprap, gabions, slope stabilization, channel linings and footing aprons. A complete list of the various types of armoring countermeasures is located in HEC-23.

Riprap

Layers or facings of properly sized and graded rock or broken concrete, placed or dumped to protect an abutment, pier or embankment from erosion (see Figure 13.1.9). Riprap has also been used to almost all kinds of armor which include wire-enclosed riprap partially grouted riprap, sacked concrete and concrete slabs. Riprap should be protected against subsurface erosion by filters formed either of properly graded sand/gravel or of synthetic fabrics developed and utilized to replace the natural sand/gravel filter system. It must be placed on an adequately flat slope to be able to resist the anticipated forces of the flowing flood waters. Proper design and placement of riprap is essential. This generally requires placement of the riprap on side-slopes no steeper than 1.5 to 1 vertical (1.5H:1V). Flatter side-slopes of such as 2H:1V to 3H:1V are preferable. Proper design and placement of riprap is essential. Inappropriate installations can aggravate or cause the conditions they were intended to correct or prevent.

Gabions

Rectangular rock- or cobble- filled wire mesh baskets or compartmented rectangular containers, anchored together and generally anchored to the surface they are protecting (see Figure 13.1.12). Gabions may be placed on steeper slopes than riprap or may even be stacked vertically, depending upon the design procedure and site conditions.

Slope Stabilization Methods

Slope stabilization methods consist of the placement of geotextiles, wire mesh, riprap, paving, revetment, plantings or other materials on channel embankments, intended to protect the slope from erosion, slipping or caving or to withstand external hydraulic pressure (see Figure 13.1.13). It is anticipated the various stabilization methods will fill-in with sediment and help sustain plant growth. The roots from the plants contribute to stabilize the embankment or flood plain.

Channel Lining

Channel lining is a concrete pavement that extends across the streambed. Channel linings also may be revetment mats or some other form of bed armoring. A typical revetment mat is formed by interlocking precast concrete blocks linked by cable (polyester or steel) placed on a geotextile fabric. The interlocking matrix allows for use over varying land contours and grades (see Figure 13.1.14). Channel linings may also consist of formed concrete. This type is less flexible and versatile than revetment mats and other bed armoring (see Figure 13.1.15).

Footing Aprons

Footing aprons are protective layers of material surrounding the footing of a substructure unit. Footing aprons usually consist of cast-in-place concrete (see Figure 13.1.16 and 13.1.17). Footing aprons protect footings from undermining. The aprons are not a structural element of the abutment or pier footings and are considered a structural countermeasure instead of a hydraulic countermeasure.

Figure 13.1.9 Crushed Stone Riprap

Figure 13.1.10 Spurs

Figure 13.1.11 Guide banks Constructed on Kickapoo Creek Near Peoria, Illinois

Figure 13.1.12 Gabion Basket Serving as Slope Protection

Figure 13.1.13 Slope Stabilization

Figure 13.1.14 Concrete Revetment Mat

Figure 13.1.15 Formed Concrete Channel Lining

Figure 13.1.16 Concrete Footing Apron on a Masonry Abutment

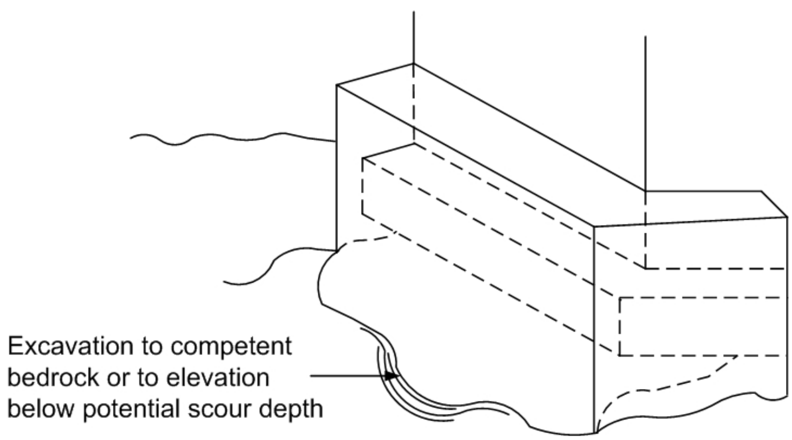

Figure 13.1.17 Concrete Footing Apron to Protect a Spread Footing from Undermining

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Chapter 13
Inspection and Evaluation of Waterways

Topic 13.2 Inspection of Waterways

13.2.1 Introduction

The bridge inspector needs to be able to correctly identify and assess waterway deficiencies when performing a bridge waterway inspection. Accurate bridge waterway inspections are vital for the safety of the motoring public. For this to happen, have a thorough understanding of the different types of waterway elements and deficiencies, as well as the various inspection techniques. See Topic 13.1 for detailed descriptions of various waterway elements.

Waterway deficiencies are properties of the waterway or substructure members that work to act negatively on the structural integrity of the bridge. They are mostly interrelated and when a change in one of these properties occurs, others are also often affected.

13.2.2 Waterway Performance Factors

Waterway Alignment

In general, bridges are designed so that the flow passes through the waterway parallel to the axes of the abutments and the piers. If the path of flow shifts in direction as a result of continued lateral movement so that it approaches the abutments and the piers at a significant skew angle, the capacity of the waterway can be reduced. More significantly, local scour will be increased and may lead to the failure of the structure. This depends upon the original design conditions and the degree of change resulting in misalignment in the flow with the critical elements supporting the structure. Carefully note any change in direction of the approach of the flow to the bridge and any change in the angle at which the flow hits or impinges on the abutments and piers. Also make observations of local change in flow directions and surveys of changes in bed and bank elevations. Evaluation of aerial photographs over time is extremely useful in assessing changes in waterway alignment. All of this information may be utilized to rate the severity of increasing misalignment in the flow on bridge safety.

Example of channel misalignment: If the approaching flow impinges on rectangular piers at an angle of 45 degrees versus flowing parallel to the axis of the piers, the depth of scour may be increased by a factor of two or more. The actual factor of increase depends upon the characteristics of the bed material, the pier type, and the duration of the flood.

For bridges spanning over wide floodplains, the approach angle of the low flow channel may not be significant. In these cases it is the alignment of the floodplain flow during the larger floods that will determine the magnitude of local scour.

Streamflow Velocity

Streamflow velocity is a major factor in the rate and depth of scour. During flood events, the streamflow velocity is increased, which produces accelerated scour rates and depths. At high streamflow velocities, bridge foundations have the greatest chance to become undermined (see Figure 13.2.1).

Figure 13.2.1 Flood Flow Around a Pier Showing High Streamflow Velocity

The streamflow velocity depends on many variables. One of these variables is the stream grade. A steep stream grade will produce high streamflow velocities, while a flat stream grade produces low streamflow velocities. Other variables that affect the streamflow velocity include the waterway alignment, the hydraulic opening, any natural or man-made changes to the stream, flooding, etc.

Hydraulic Opening

It is necessary to consider the adequacy of the hydraulic opening (the cross-sectional area under the bridge) to convey anticipated flows, including the design flood, without damage to the bridge. It is essential to maintain a bridge inspection file comparing original conditions in the waterway at the time the bridge was constructed to changes in the cross-sectional area of the channel under the bridge over time.

The primary method of assessing loss of cross-sectional area of the hydraulic opening is to determine channel bed elevation changes. This can be determined by a periodic survey of the channel bed or by taking soundings from the bridge. Typically, a number of survey or sounding points spaced across the bridge opening are established to determine changes in cross-sectional area. Note the lateral location of these surveyed points so that as subsequent inspections are conducted, the survey points can be repeated to maintain consistency. Photographs from key locations can be used to document debris and vegetation that can block the bridge opening.

Stream gages in the vicinity of the bridge may be useful in evaluating the adequacy of the waterway in relationship to changing hydraulic conditions. For example, stage-discharge curves based on discharge measurements by the United States Geological Survey (USGS) or other agencies and shifts in rating curves may indicate changes in channel bed elevation and cross section.

Streambed Material

The size, gradation, cohesion, and configuration of the streambed material can affect scour rates. When comparing sands and cohesive soils, such as clays, the size of the streambed material has little effect on the depth of scour, but can affect the amount of time needed for this depth to be attained. Cohesive streambed materials that are fine usually have the same ultimate depth of scour as sand streambeds. The difference is that the cohesive streambeds take a longer period to reach this ultimate scour depth. For these reasons, the streambed type is important and correctly evaluated by the bridge inspector. Streambed rates of scour for different types of material are described later in this topic.

Substructure Shape

Substructure members on old bridges were not necessarily designed to withstand the effects of scour. Wide piers and piers skewed to the flow of the stream can contribute to an increase the depth of scour. Due to increased awareness of bridge waterway scour, recent substructure members have been designed to allow the stream to pass through with as little resistance as possible. Many newer piers have rounded or pointed noses, which can decrease the scour depth by up to 20%.

Foundation Type

Footings that are undermined, but founded on piles are not as critical as spread footings that are undermined. Determine the substructure foundation type, in order to properly evaluate the substructure and the waterway. The foundation type may often be determined from design and/or construction drawings. In some older bridges, the foundation type is not known. In this case, advanced inspection techniques by a trained professional may be required to verify the foundation type.

13.2.3 Waterway Deficiencies

Total Scour

The most common bridge waterway deficiency is scour, which may adversely impact bridge substructure units. Scour is the removal of material from the streambed or embankment as a result of the erosive action of streamflow.

The rate of scour will vary for different streambed materials, and for different streamflow rates. For a given streamflow rate, a streambed material will scour to a maximum depth in a given time. The following are examples for different types of streambeds and their corresponding scour rate:

- Dense granite: centuries
- Limestone: years
- Glacial tills, sandstone and shale: months
- Cohesive soils (clay): days
- Sand and gravel: hours

There are three forms of scour considered in evaluating the safety of bridges:

- Aggradation and degradation
- General scour (which includes contraction scour)
- Local scour

Aggradation and Degradation

Aggradation and degradation are long-term streambed elevation changes. Aggradation is the general and progressive buildup of the longitudinal profile of a channel bed due to the sediment deposition. (see Figure 13.2.2). Degradation is the general and progressive (or long-term) lowering of the channel bed due to erosion, over the relatively long channel length (see Figure 13.2.3).

Aggradation and degradation may be a result of the natural erosion and downcutting process that rivers experience through the years. This scour type may be accelerated by natural cutoffs in a meandering river, which steepens the channel gradient, increasing both the velocity of flow and hence scour. These changes may also be accelerated by various types of development or river modification, such as:

- Upstream dam construction
- Dredging
- Straightening or narrowing of the river channel
- Upstream development resulting in an increase of precipitation into the channel

Since aggradation and degradation of the channel bed is along some considerable distance of channel, major facilities are sometimes used to control scour. These facilities can include a series of drop structures (small dam-like structures) or other scour protection of the riverbed. Presence of such structures may be indicative that the channel is experiencing scour.

Factors that may cause changes in the elevation of the streambed include:

- Water resources development, such as upstream diversions and upstream dams
- Changes in channel alignment or dimensions
- Urbanization of the watershed (conversion of a more natural or agricultural area to a city)

Headcut migration is the degradation of the channel that is associated with abrupt changes in the bed elevation and then migrates upstream. Headcutting tends to form in more cohesive materials in a streambed. Cohesive materials are discussed on page 13.2.18.

Figure 13.2.2 Streambed Aggradation

Figure 13.2.3 Streambed Degradation

General Scour

General scour can occur in a short time with the right conditions (see Figure 13.2.4 and 13.2.5). It is the lowering of the streambed across the waterway at the bridge which may or may not be uniform. This means it could be deeper in some parts than in others. General scour could be the result of contraction of the flow, which will result in the removal of the streambed material across all or most of the channel width or from other general scour conditions, such as flow around a bend where the scour will be concentrated near the outside of the bend.

Figure 13.2.4 General Scour

Figure 13.2.5 Close-up of General Scour of a Pier

Changes in downstream elevation, such as at the confluence with another river which is undergoing scour of its own, can cause general scour in the upstream river. Weather events such as hurricanes can also cause general scour (see Figures 13.2.4 and 13.2.5).

General scour may reduce the degree of safety experienced by the substructures, because of the changed hydraulic conditions and the changed channel geometry. In this case, it is essential to refer to the bridge inspection file and study historical changes that have occurred in the bed elevation through the waterway. If possible, these changes are related to specific causes to assess the present safety of the bridge. These changes also provide insight as to future conditions that may be imposed by changed flow conditions, watershed development, or other conditions affecting the safety of the bridge.

Contraction Scour

Contraction scour results from the acceleration of flow due to a natural contraction, a bridge contraction, or both (see Figures 13.2.6 and 13.2.7). When the available area for stream flow at the bridge is reduced compared with the available area upstream from the bridge, velocity will increase at the bridge. Less area for flow results in faster moving water. The lowering of the streambed under the bridge due to this accelerated stream velocity is known as contraction scour. A bridge length may be shortened to reduce the initial cost of the superstructure. However, this shortened bridge results in a smaller hydraulic opening which can lead to contraction scour (see Figure 13.2.8).

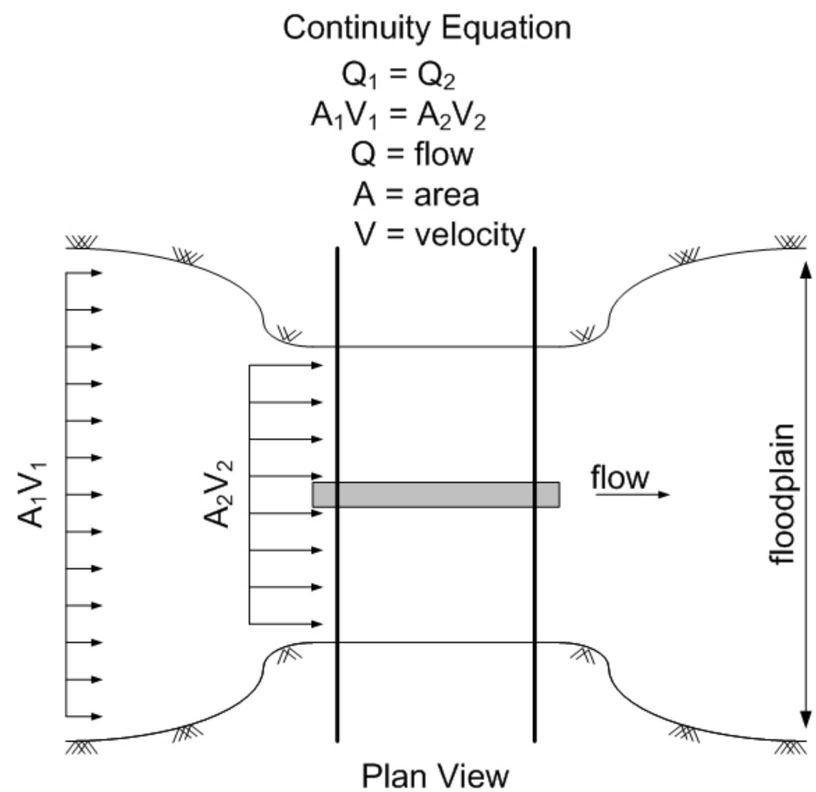

Figure 13.2.6 Stream Contraction Schematic

Figure 13.2.7 Contraction Scour Photograph

Figure 13.2.8 Large number of Piers Combine to Reduce the Hydraulic Opening

Some common causes that can lead to contraction scour include:

- A natural stream constriction such as hard rock on embankment slopes.
- Excessive number of piers in the waterway (see Figure 13.2.8)
- Heavy vegetation in the waterway or floodplain (see Figure 13.2.9).
- Bridge roadway approach embankments built in the floodplain constricting the waterway opening. The overbank area of the floodplain is restricted by the bridge approach embankments extending partially across the floodplain.
- Formation of sediment deposits within the waterway along the inside radius of curved waterways (sandbars), and along embankments that constrict or reduce the available waterway opening (see Figure 13.2.10).

- Ice formation or ice jams that temporarily reduce the waterway opening and produce contraction (see Figure 13.2.11).
- Flow under an ice sheet or flow contracting down under bottom of the superstructure.
- Debris buildup, which often reduces the waterway opening (see Figure 13.2.12).

The effects of contraction scour can be very severe.

Figure 13.2.9 Vegetation Constricting the Waterway

Figure 13.2.10 Sediment Deposits Within the Waterway Opening

Figure 13.2.11 Ice in Stream Resulting in Possible Contraction Scour

Figure 13.2.12 Debris Build-up in the Waterway

Other General Scour

Other general scour conditions result from erosion due to streams which are meandering, braided, or straight, variable downstream control, flow around a bend or any other changes which may cause a decrease in the bed elevation. This could also result from a short-term change in downstream water surface elevation which can control the velocity through the bridge. This may occur at bridges located upstream or downstream from a confluence.

Local Scour

Local scour occurs around an obstruction that has been placed within a stream, such as a pier or an abutment which causes an acceleration of the flow and results in induced by the obstruction. Local scour can either be clear-water scour or live-bed scour.

Clear-water scour occurs when there is no bed material transport upstream of the bridge. It occurs in streams where the bed material is coarse, the stream grade is flat, or the streambed is covered with vegetation except in the location of substructure members.

Live-bed scour occurs when local scour at the substructure is accompanied by bed material transport in the upstream waterway.

The cause of local scour is the acceleration of streamflow resulting from vortices induced by obstructions (see Figure 13.2.13). Some common obstructions are:

- Abutments – floodplain overbank flow is collected along and forced around abutments at high velocities (see Figure 13.2.14).
- Wide Piers - scour depth is proportional to width (see Figure 13.2.15).
- Long Piers - can produce multiple vortices and greater scour depth if the pier is at an angle to the flow direction (see Figure 13.2.16).Unusually Shaped Piers - can increase vortex magnitude. A square-nosed pier will have maximum scour depth, about 20 percent deeper than a sharp-nosed pier and 10 percent deeper than a cylinder or round-nosed pier.
- Bridge Piers Skewed to the Direction of Streamflow - can increase both contraction scour and local scour because of increased (projected) pier width effects. This skew can be dramatically different during low flow versus high flows.
- Depth of Streamflow - increases vortex effect on the streambed. An increase in flow depth can increase scour depth by a factor of 2 or more (see Figure 13.2.13).
- Streamflow Velocity - as streamflow velocity increases vortex action can be magnified considerably.
- Unstable Streambed Material - can contribute to the occurrence of local scour.
- Irregular Waterway Cross Section - can result in local scour at substructure units in the waterway.
- Debris Accumulation - and ice piled up against piers can produce the same effect as a wider pier, increasing both contraction and local scour effects. Debris needs to be removed as a safety precaution to prevent pier failure

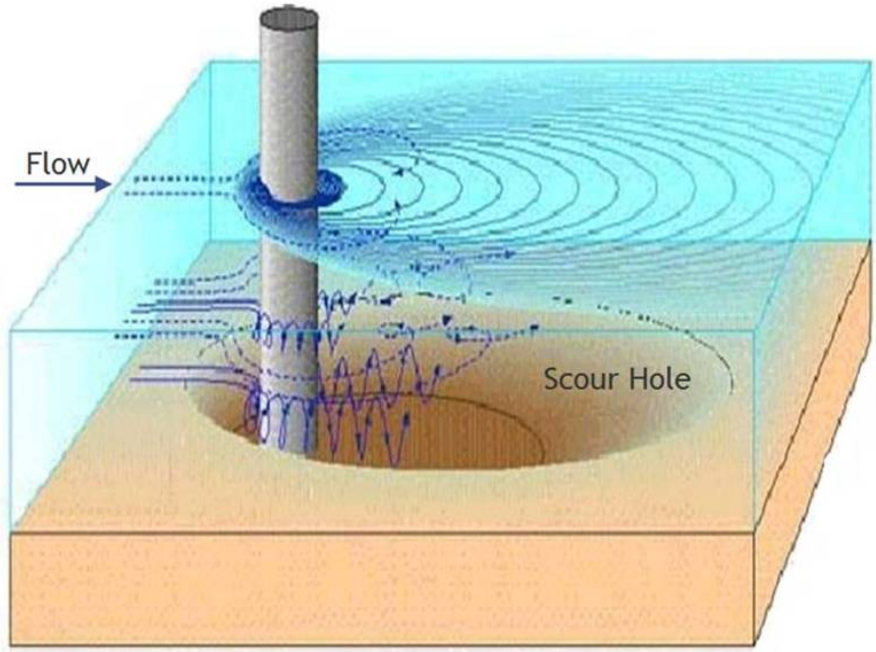

Figure 13.2.13 Local Scour at a Pier

Figure 13.2.14 Local Scour at a Pier

Figure 13.2.15 Wide Pier

Figure 13.2.16 Long Pier

Scour depths resulting from local scour are normally deeper than those from general scour, often by a factor of ten. However, if there are major changes in hydrologic conditions resulting from such factors as construction of large dams and water resources development, the general scour can be the larger element in the total scour.

Bridges in tidal situations are particularly vulnerable to local scour. A strong tidal current whose direction reverses periodically causes a complex local scour phenomenon around a bridge substructure. This local scour is caused by an imbalance between the input and output sediment transport rates around the pier, and it has a negative influence on the stability of the bridge.

To properly evaluate local scour and impacts of changes in hydrologic and hydraulic conditions on local scour, it is essential to develop and refer to that component of the bridge inspection file which deals with local scour. With each inspection, subject the critical supporting elements of the bridge to careful survey to determine the degree of local scour that has developed over time. By referring to this history of change in local scour, it can be determined whether or not the maximum local scour has occurred and the relationship of this maximum local scour to bridge safety.

If the survey of the magnitude of local scour indicates increased local scour with time and furthermore verifies that the local scour exceeds the anticipated maximum local scour when the bridge was designed, take remedial measures to protect the bridge. Surveys of local scour along the abutments and around the piers are most often done during periods of low flow when detailed measurements can be made, either by wading and probing, by probing from a boat, by the use of divers, or by sonic methods. The pattern of survey has to be established and remain the same during the life of the bridge, following either a fixed radial or a rectangular grid. Changes in magnitude of local scour can then be compared at specific points over time.

The greatest problem associated with determining the magnitude of local scour relates to maximum local scour occurring at flows near flood peak followed by a period of deposition of sediments in the scour hole after the flood peak has passed and during low-flow periods. Consequently, base the bridge rating upon maximum scour that occurred during floods but not based upon examination of bed levels around abutments and piers during low-flow periods. Hence, it is necessary to use a variety of techniques to differentiate between maximum scour that may have occurred during flood periods and apparent scour after periods of low flow.

Consider utilizing straight steel or aluminum probing rods to probe loose sediments deposited along abutments and around footings; if sediment is finer than average bed material sizes or if the sediment is easily penetrated by the rod, it is indicative that the present sediment has accumulated in the scour hole and local scour is more severe than indicated by present accumulations of sediments. Core samples may also be used to differentiate between backfill in the scour hole and the bottom of the scour hole. It may be possible to use geotechnical means as another alternative to differentiate between materials that have deposited in the scour hole and the bottom of the scour hole. It may also be necessary to use underwater surveys using divers, or perhaps to even divert water away from critical elements to allow removal of loose backfill material. The inspector can then determine the true level of maximum scour in relationship to the bridge's supporting structural elements.

The problem of accurately determining maximum local scour and rate of change of local scour over time is one of the most difficult aspects of bridge inspection and is one of the most important aspects of evaluating bridge safety. Additional research is being conducted to provide better guidelines for investigating local scour in relationship to bridge safety.

Lateral Stream Migration Lateral stream migration or horizontal change in the waterway alignment is another type of erosion that can also threaten the stability of bridge crossings. Embankment instability typically results from lateral stream movement at a bridge opening and has often been the primary cause in a number of bridge collapses around the country. Bridge abutments and piers are often threatened by this type of erosion (see Figure 13.2.17).

Figure 13.2.17 Lateral Stream Migration Endangering an Abutment

Lateral stream migration often threatens bridge abutments, piers and approach roadways, particularly those that are along upstream banks at the bridge opening. Lateral stream migration can occur in four modes of bank failure:

- Streambank damage – onset of lateral stream migration. The toe of the slope of the embankment will exhibit lateral scour and the streambank protection will be failing. (see Figure 13.2.18)
- Sloughing streambank – next level of streambank damage where lateral scour has removed enough of the slope that the streambank slides down into the channel. This occurs most often when streambanks are unprotected. (see Figure 13.2.19)
- Undermined streambank – an advanced state of lateral scour where the overbank area is undercut. The original embankment slope is gone. This occurs because the streambank and/or overbank protection at the surface is able to support itself without the underlying streambank material. (see Figure 13.2.20)
- Channel misalignment – an adverse channel offset where the stream flow now impacts one of the bridge abutments or flows through the under bridge waterway at a skew angle incompatible with the span opening(s).

This results when earlier stages of lateral stream migration are allowed to advance unchecked, and leads to local scour conditions that result in undermining and substructure distress. (see Figure 13.2.21)

Figure 13.2.18 Streambank Damage

Figure 13.2.19 Sloughing Streambank

Figure 13.2.20 Undermined Streambank

Lateral stream migration is very common and can result from a variety of causes. Channel changes contributing to lateral stream migration include:

- Stream meander changes due to slope instability, cuts or additional exposure that was not visible before (see Figure 13.2.21)
- Channel widening (see Figure 13.2.22)

Series of aerial photographs over time could be a way to check for lateral stream migration.

Figure 13.2.21 Stream Meander Changes

Figure 13.2.22 Channel Widening

When inspecting for lateral stream instability, some visual indicators are:

- Steep eroding banks on the outside of bends
- Tension cracks in the soil at the top of the bank
- Active undercutting of trees and riparian vegetation along the banks
- Bank sloughing due to undercutting of the toe
- Wide point bars on the inside of meander bends
- Alternate point bars developing in an otherwise straight channel
- Piers that were originally on the floodplain are now in the main channel
- Oxbow lakes or evidence of recent meander cutoffs in the floodplain

The resistance that a streambank has to erosion is closely related to several characteristics of the bank material. The bank material that is deposited in the stream can be classified as:

- Noncohesive bank material - can be removed grain by grain from the streambank. The rate of the streambank erosion are affected by factors which include the particle size, streambank slope, the direction and magnitude of the velocity adjacent to the streambank, turbulent velocity fluctuations, the magnitude of and fluctuations in the shear stress exerted on the streambanks, seepage force, piping and wave forces (see Figure 13.2.23).
- Cohesive bank material - Cohesive bank material is more resistant to erosion than noncohesive bank material. It has low permeability which will reduce the effect of seepage, piping, frost heaving and subsurface flow on the stability of the streambanks. However, if the streambank is undercut and/or saturated, they are more likely to fail due to the mass wasting processes (see Figure 13.2.24).

- Composite bank material - Composite bank material consists of layers of various sizes, permeability and cohesive material. The noncohesive layers will be subjected to surface erosion, but may be partially protected by adjacent layers of cohesive materials. However, this type of bank material is vulnerable to erosion and sliding due to subsurface flows and piping (see Figure 13.2.25).

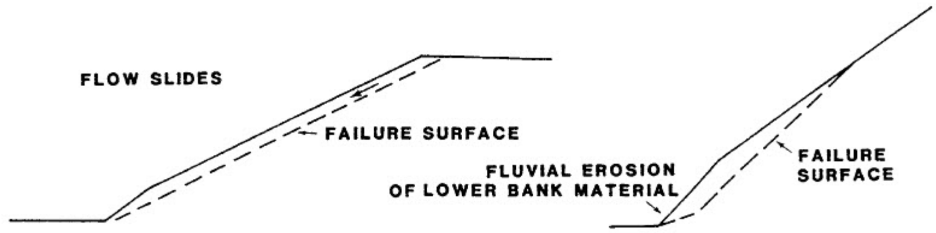

Figure 13.2.23 Schematic of Noncohesive Bank Material

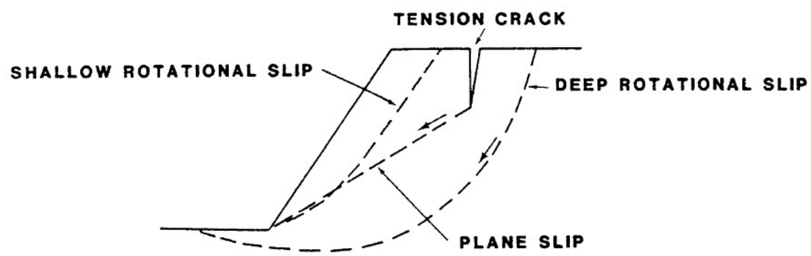

Figure 13.2.24 Schematic of Cohesive Bank Material

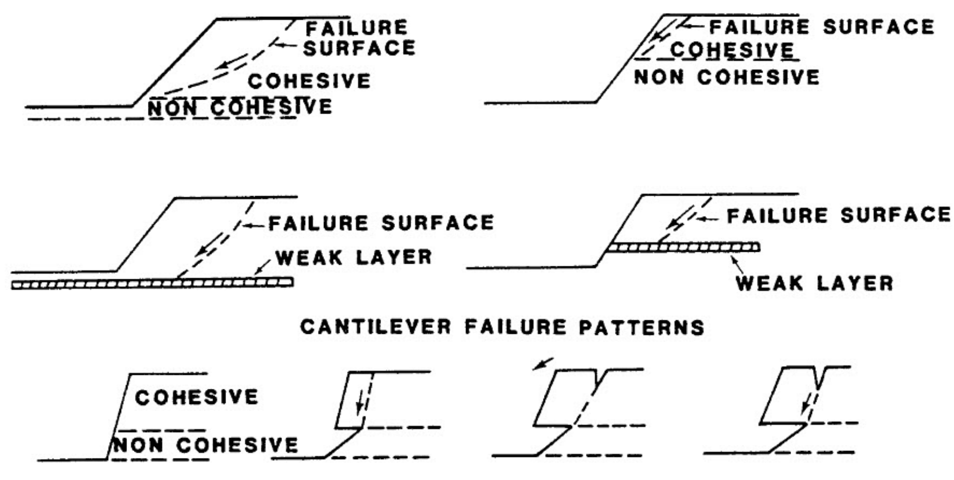

Figure 13.2.25 Schematic of Cohesive Bank Material

13.2.4 Effects of Waterway Deficiencies

Material Defects

Material defects that can be caused by waterway deficiencies include the deterioration and damage (i.e. abrasion, corrosion, scaling, cracking, spalling, and decay) to channel protection devices and substructure members.

As an integral part of the waterway inspection, give careful consideration to the identification of material defects. A loss of quality and quantity of materials required to provide bridge safety may occur in a variety of ways. Carefully record the changes in characteristics of materials in the bridge inspection file. Changes over time can be compared and any decision concerning maintenance requirements or replacement becomes more straightforward with historic information available.

Bridge Damage

Waterway deficiencies that are severe have the capability to cause damage to bridges. Effects of waterway deficiencies on bridge members include undermining, settlement, and failure.

Undermining

Undermining is the scouring away of streambed and supporting foundation material from beneath the substructure (see Figure 13.2.26). Excessive scour often produces undermining of both piers and abutments. Such undermining is a serious condition, which requires immediate correction to assure the stability of the substructure unit. Undermining is especially serious for spread footings, but may also be cause for concern for pile foundations because loss of supporting soil around piling can reduce pile capacity. Substructure stability may be compromised, potentially leading to total bridge collapse.

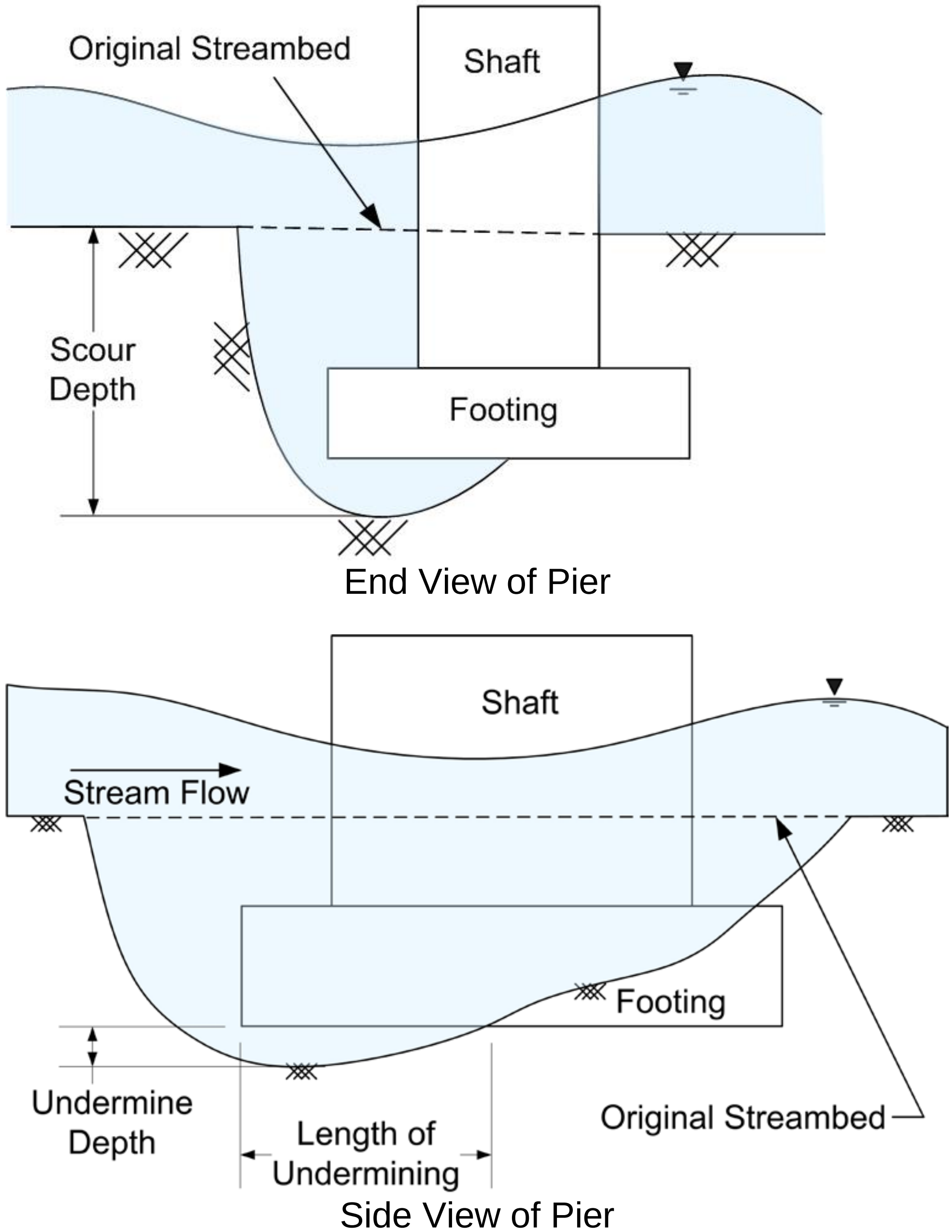

Figure 13.2.26 End and Side View of Scour and Undermining

The undermining of structural elements is basically an advanced form of scour. It is essential to determine whether or not undermining has a potential to develop, as well as whether it has already occurred. Address undermining immediately since it can pose an immediate threat to safety.

With small bridges, L-shaped rods can be used to probe at the base of footings to determine possible undermining. On the other hand, undermining may be very difficult to identify due to the redeposition of sediments during periods of low flow after undermining has occurred. However, in those channels where the bed is formed of coarse rock and the sediment supply to the bridge crossing is small, it is possible to inspect the footings because the backfill with fine sediments during periods of low flow generally does not occur.

For areas not accessible to effective probing from above water, it is essential to employ underwater inspection techniques utilizing divers. Whenever possible, take detailed measurements, showing the height, width, and penetration depth of the undermined cavities. Refer to Topic 13.3 for a more detailed description of underwater inspections.

Settlement

Local scour and undermining is typically most severe at the upstream end of the substructure and, if not corrected, may result in differential settlement (see Figure 13.2.27).

Figure 13.2.27 Pier Settlement due to Undermining

Failure

When undermining and settlement go undetected for some length of time, the bridge may become unstable, and be subject to failure or collapse. Failure may occur over a period of time, or it may be a very rapid process occurring during a flood event.

13.2.5 Inspection Preparation

It is necessary to identify and assemble the documentation and equipment required to conduct the waterway inspection. The required equipment will depend upon the characteristics of the river, the characteristics of the bridge, and the accessibility of the site.

Information Required

Necessary information is required for a comprehensive, well-organized inspection of waterways.

Examine any previous hydraulic engineering scour evaluation studies on the bridge. These studies provide theoretical ultimate scour depths for the bridge substructure elements. Review original drawings and previous inspection report data taken from successive inspections to determine the foundation type and streambed material. Establish whether the waterway is stable, degrading or aggrading.

Become familiar with site conditions and channel protection installations. Verify if there is a change in the hydraulic opening by reviewing previous channel cross sections and profiles. Examine the photographs to determine any changes in the channel alignment.

Considering the complexity of the inspection and the equipment and materials needed to execute the inspection, develop a detailed plan of investigation, as well as forms for recording observations. Use a systematic method each time the bridge is surveyed to provide a means of accurately identifying changes that have occurred at the bridge site, which may affect the safety of the bridge.

Inspection Methods

Prior to beginning the inspection, the bridge inspector needs to understand the type and extent of the inspection required. Waterway inspections are typically accomplished by either surface inspection or underwater diving inspection.

Surface or “wading” inspection is conducted on shallow depth foundations. Submerged substructure, streambed and embankments are often accessible by inspectors using hip boots or chest waders and probing rods (see Figure 13.2.28). Additionally, boats are often used as a surface platform from which to gather waterway data, including channel cross-sections, pier soundings, etc.

Underwater diving inspection is required when the foundations are deep into water. Site conditions often require waterway and submerged substructure units to be evaluated using underwater divers, in order to obtain complete, accurate data. This is especially true when water depths are too great for wading inspection, and/or undermining of substructure elements is suspected.

Equipment

Equipment required to inspect bridges is listed and described in Topic 2.4. Additional equipment may be required for the inspection of waterways. The type of equipment needed for a waterway inspection is dependent on the type of inspection. The following is a list that represents the most common waterway inspection equipment.

- Probing rods and waders (see Figure 13.2.28)
- Sounding line (lead line to measure depths of scour)
- Fathometer to determine water depth

- Diving equipment (see Figure 13.2.29 and Topic 13.3)
- Boat, oars, motor, and anchor
- Surveying equipment (level or transit)
- Survey tapes and chains
- Level rod
- Compass
- Underwater camera and video recorder
- Underwater to surface communication equipment
- Past climatic and hydrologic records
- Stopwatch to time stream velocity and record diver durations under water

Refer to Topic 13.3 for additional information on underwater inspection equipment.

Figure 13.2.28 Probing Rod and Waders

Figure 13.2.29 Surface Supplied Air Diving Equipment

Special Considerations

Give special considerations to the site conditions and the navigational controls that may adversely affect the safety of the bridge inspector and others.

Site conditions such as rapid stream flow velocity, pollution levels, safety concerns, and conditions requiring special attention need to be accounted for during a waterway inspection (see Figure 13.2.30).

Navigational control is necessary when inspecting large waterways. Notify the Coast Guard in advance of inspections where navigational controls are needed. Other navigational controls include boat traffic, operational status and condition of dolphins and fenders, dam releases (see Figure 13.2.31).

Figure 13.2.30 Rapid Flow Velocity

Figure 13.2.31 Navigable Waterway

13.2.6 Inspection Methods and Locations

Methods

Visual

The primary method used to inspect waterways is visual. Look at the site in the vicinity of the bridge. Also, look at the floodplain. This observation may have to be done during periods of high water flow.

Physical

After the inspector gets the general condition by visually inspecting the bridge site, the next step is to probe for any scour or undermining. Take care to adequately press the probing rod into the soil in the streambed. Sometimes scour holes are loosely filled with silt. This silt may be washed away quickly during the next period of high stream flow velocity, permitting additional scour.

Advanced Inspection Methods

Take measurements to obtain the cross section and profile. These measurements are used to analyze the area of the hydraulic opening and help determine need for and design of mitigation measures. The cross section under the bridge can be measured with a surveyor's tape or rod. The stream profile can be measured with a hand level, survey tape and surveying rod (see Figures 13.2.32 and 13.2.33). Compare the streambed profile and hydraulic opening to previous inspections.

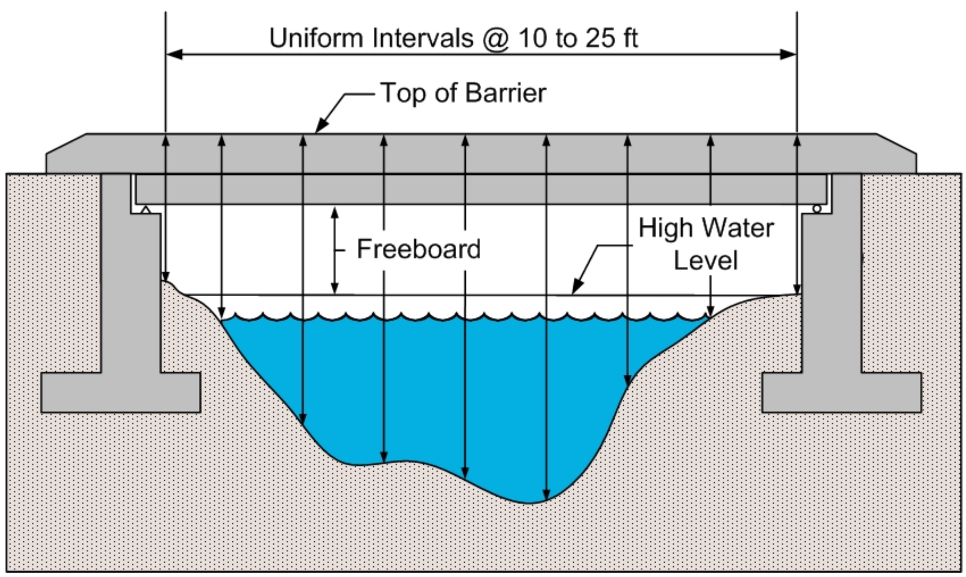

Figure 13.2.32 Streambed Cross-Section

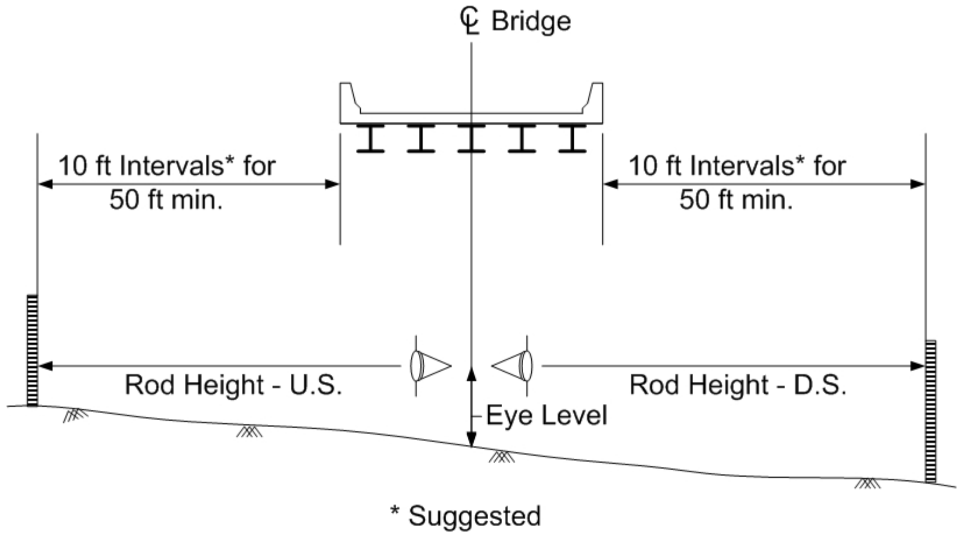

Figure 13.2.33 Streambed Profile

An alternative to the sounding and scour sensing devices used during inspections is to permanently install fixed instrumentation directly on the bridge substructure. With fixed instrumentation, local scour is continuously monitored and recorded as it occurs, unaffected by washing back of silts and sands, and making information readily available to the bridge owner by setting off a beacon-type alarm on the bridge deck (or relayed back to an office). One such instrument consists of a steel rod inside of a conduit attached to the substructure unit. The rod acts as a probe, resting on the vulnerable soil supporting the substructure. As local scour occurs the soil is washed away and the rod drops a measured distance.

Other fixed instrumentation includes fixed sonar units, sliding magnetic collars, and buried “float-out” buoys, which float to the water surface after being uncovered by local scour, activating an electronic alarm system (see Figure 13.2.34).

Researchers are studying a new method for scour detection and monitoring. The new method is based on time domain reflectometer (TDR) technology, which uses pulse transmissions to show changes in a particular environment. The TDR bridge scour monitoring system consists of a probe, which is completely buried in the sediment at appropriate locations around and near the bridge pier and footings. As erosion occurs, part of the probe is exposed to water. Then, the probe reflects a specific pulse back to the TDR box, which is on the surface, indicating how much of the probe is exposed and producing wave forms to show scour depth. The probes are designed to be left at bridge sites to detect/monitor scour.

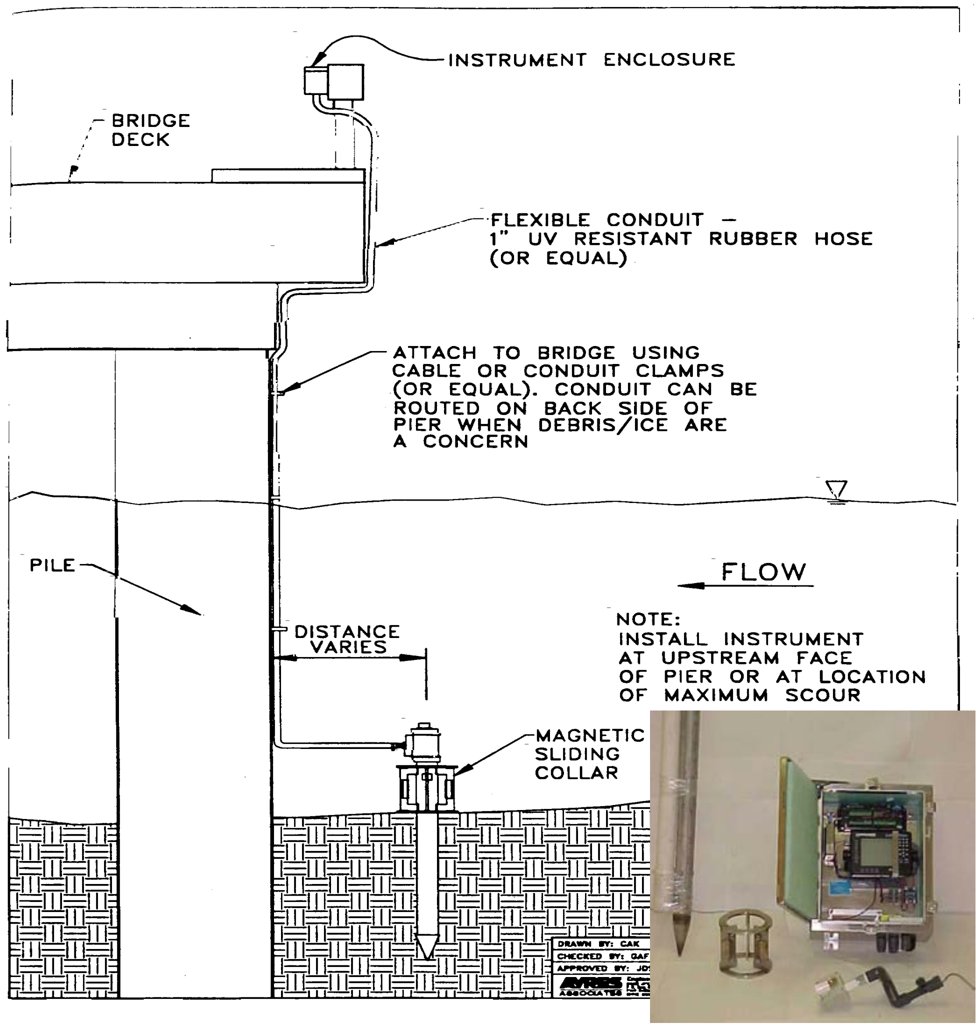

Figure 13.2.34 Scour Monitoring Collar

Locations

When inspecting the bridge waterway, three main areas are of concern. These areas include the channel under the bridge, the upstream channel, and the downstream channel.

Channel Under the Bridge

Substructure

- Inspect substructure units below water level for defects, damage and foundation condition (see Figure 13.2.35).
- Measure heights and lengths of foundation element exposures, and dimensions of foundation undermining (opening height, width, and penetration depth), as applicable. Document with sketches and photos.
- Note location of high water mark on abutments and piers.
- Plumb face of abutments and piers for local settlement (see Figure 13.2.36).
- Check abutments and piers for accumulations of debris (drift).

- In case of damage to scour countermeasures, check condition and function of channel protection devices adjacent to substructure units.
- In case of changes in streambed elevations generate streambed profile.
- In case of changes in streambed cross section generate streambed cross-sections for typical upstream, downstream, and under structure waterway configurations.
- Locate and contour large scour holes at the substructure.
- Establish a grid system for depth soundings at substructure elements, which can be repeated in subsequent inspections.
- Take photographs to document conditions of abutments, piers, and channel features.
- Check bridge seats and bearings for transverse movement.

Figure 13.2.35 Pile Bent Deterioration Normally Hidden Underwater

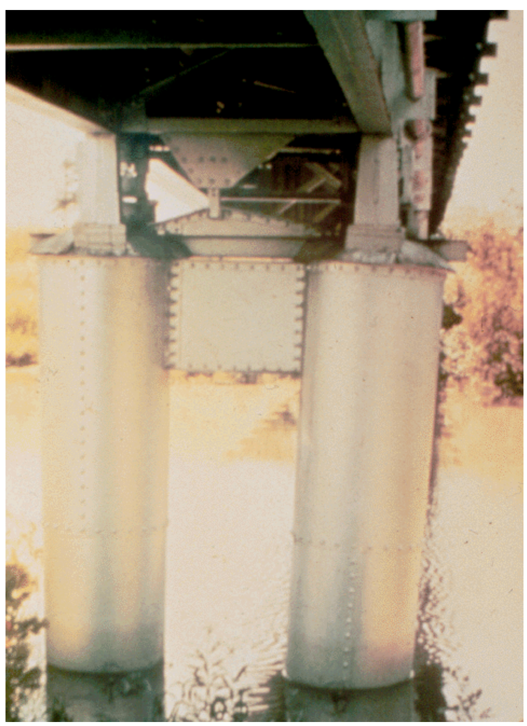

Figure 13.2.36 Out of Plumb Pier Column

Superstructure

During a waterway inspection, the superstructure can be a good indicator of existing waterway deficiencies.

The following items need to be reviewed:

- Check to see if the superstructure is tied to the substructure to prevent washout.
- Sight along the superstructure to reveal irregularity in grade or horizontal alignment caused by settlement (see Figure 13.2.37).
- Check to see if debris is lodged in superstructure elements or tree limbs above the superstructure (see Figure 13.2.38).
- Check for high watermarks or ice scars on trees.
- Talk to local residents about high water during previous flood events.
- Check any hydraulic engineering scour evaluation studies for overtopping flow elevation and frequency.
- Check to see if the superstructure is below the design flood level elevations.
- Check to see if the superstructure presents a large surface of resistance during floods.
- Note if the superstructure is vulnerable to collapse in the event of excessive foundation movement (i.e., simple span and non-redundant vs. continuous) (see Figure 13.2.39).

Figure 13.2.37 Superstructure Misalignment

Figure 13.2.38 Drift Lodged in a Superstructure

Figure 13.2.39 Multi-Span Simply Supported Bridge

Channel Protection and Scour Countermeasures

- Examine any river training and bank protection devices to determine their stability and condition.
- Check for any gaps or spreading that have occurred in the protective devices.
- Check for separation of slope pavement joints.
- Check for exposure of underlying erodible material.
- Inspect for steepening of the protective material and the surface upon

which these materials are placed.

- Check for evidence of slippage of protective works.
- Check the condition and function of riprap as well as changes in size of riprap.
- Check for evidence of failed riprap in the stream (see Figure 13.2.40).
- Check for the proper placement, condition, and function of guidebanks, or spurs.
- Check the streambed in the vicinity of the channel protection for evidence of scour under the device.
- Check to see if the streamflow is impinging behind the protective devices.

Figure 13.2.40 Failed Riprap

It is essential to identify any change that is observable, including changes in the gradation of riprap. It is also essential to carefully inspect the integrity of the wire basket where gabions have been used.

Disturbance or loss of embankment and embankment protection material is usually obvious from close scrutiny of the embankment. Unevenness of the surface protection is often an indicator of the loss of embankment material from beneath the protective works. However, loss of embankment material may not be obvious in the early stages of failure. Also look for irregularities in the embankment slope.

It is difficult to determine conditions of the protective works beneath the water surface. In shallow water, evidence of failure or partial failure of protective works can usually be observed. However, with deeper flows and sediment-laden flows, it is necessary to probe or sound for physical evidence to identify whether failure or partial failure exists.

Waterway Area

- Check the hydraulic opening with respect to the floodplain. If the width is small compared to the floodplain and return flow is expected to be large, there could be high potential for contraction scour and abutment scour
- Determine the type of streambed material.
- Check for degradation (see Figure 13.2.41).
- Check for local scour around piers and abutments and record data.
- Inspect during drought conditions when applicable.
- Check for contraction scour due to abutment placement, sediment build-up, and vegetation.
- Check for debris underwater, which may constrict flow or create local scour conditions.
- Check to see if the approach roadways are located in the floodplain (see Figure 13.2.42).
- Examine approaches for signs of overtopping.
- Determine if the hydraulic opening is causing or has the potential to cause scour under the bridge.

Figure 13.2.41 Severe Streambed Degradation Evident at Low Water

Figure 13.2.42 Approach Roadway Built in the Floodplain

Upstream and Downstream of the Bridge

Streambanks

- Stable - gradually sloped, grass covered with small trees. Streambanks are still basically in their original locations. Slope stabilization measures are in place and intact (see Figure 13.2.43).
- Unstable - streambank is sloughing due to scour, evidence of lateral movement or erosion, damage to slope stabilization measures (see Figure 13.2.19).

Figure 13.2.43 Stable Streambanks

Main Channel

- Record the flow conditions (e.g. low or high).
- Estimate velocities using floats.
- Check for sediment buildup and debris, which may alter the direction of stream flow (see Figure 13.2.44).
- Check for cattle guards and fences, which may collect debris. The results may be sediment buildup, channel redirection, or an increase in velocity and contraction scour (see Figure 13.2.45).
- Determine the streambed material type.
- Check for aggradation or degradation. Check several hundred feet upstream and downstream of the bridge.
- Check the basic alignment of the waterway with respect to the structure and compare it to its original alignment (lateral stream migration) (see Figure 13.2.46).
- Record the direction and distribution of flow between piers and abutments.
- Make sketches and take pictures as necessary to document stream alignment, conditions of bank protection works, and anything that appears unusual at each inspection.

Figure 13.2.44 Sediment Accumulation Redirecting Streamflow

Figure 13.2.45 Fence in Stream at Bridge

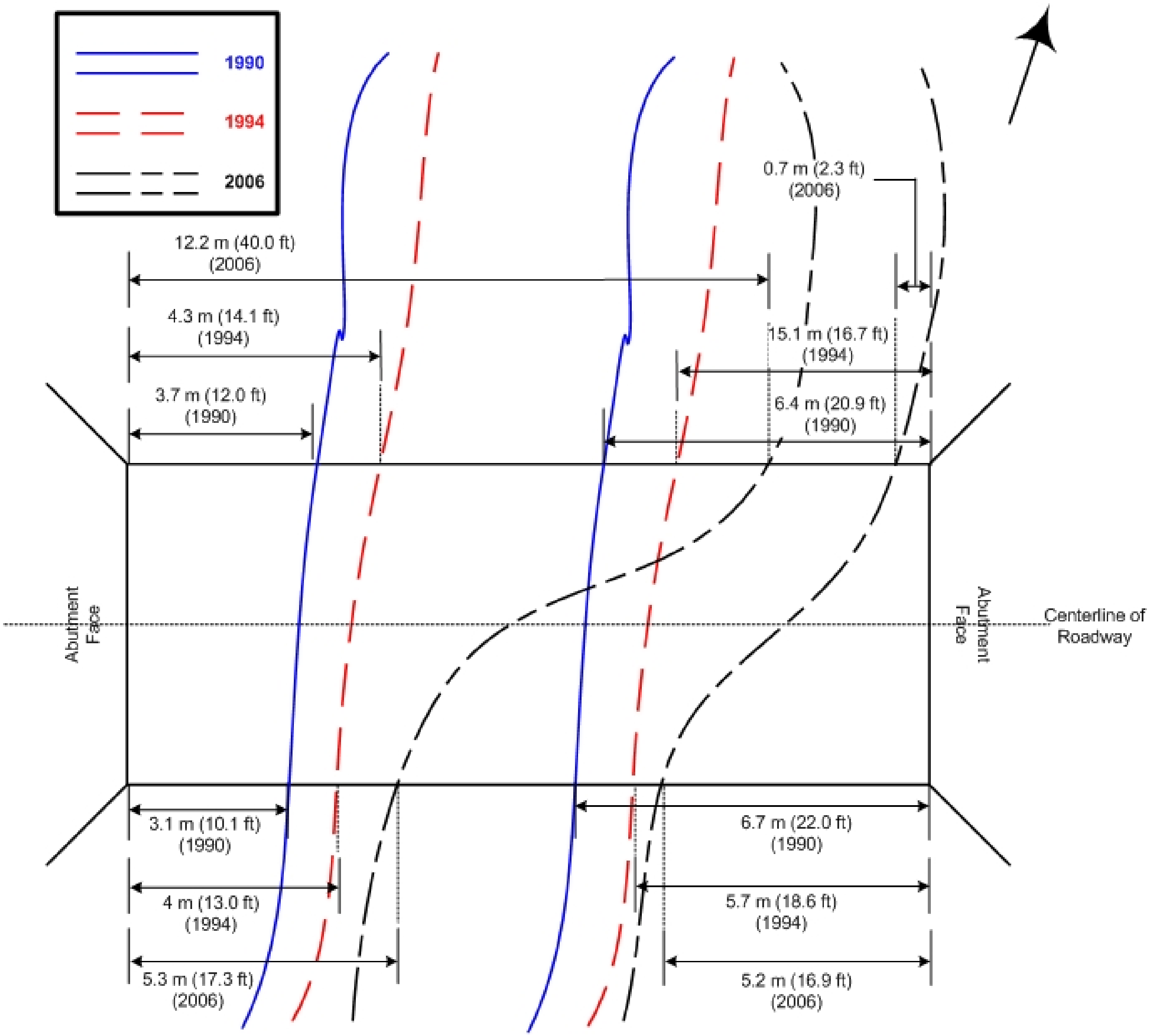

Figure 13.2.46 Waterway Alignment 1990 - 2006

Floodplain

- Check for evidence of embankment sloughing, undermining, and lateral stream migration resulting from significant stream flow (see Figure 13.2.47).
- Check for amounts and locations of debris, sediment accumulations, tree scaring, and amounts of vegetation growth, all of which may indicate the frequency of stream flow on the floodplain.
- Check for accumulations of sediments, debris, or significant vegetation growth in the waterway that may impact sufficient waterway adequacy and adversely affect streamflow under the main channel span.
- Check for damage to the approach pavement, shoulders, and embankments to determine if the stream flow overtops the approach roadway during flood flows or returns to the main channel to flow under the structure.
- Check the extent of structures, trees, and other obstructions that could impact stream flow and adversely affect the bridge site (see Figure 13.2.48).

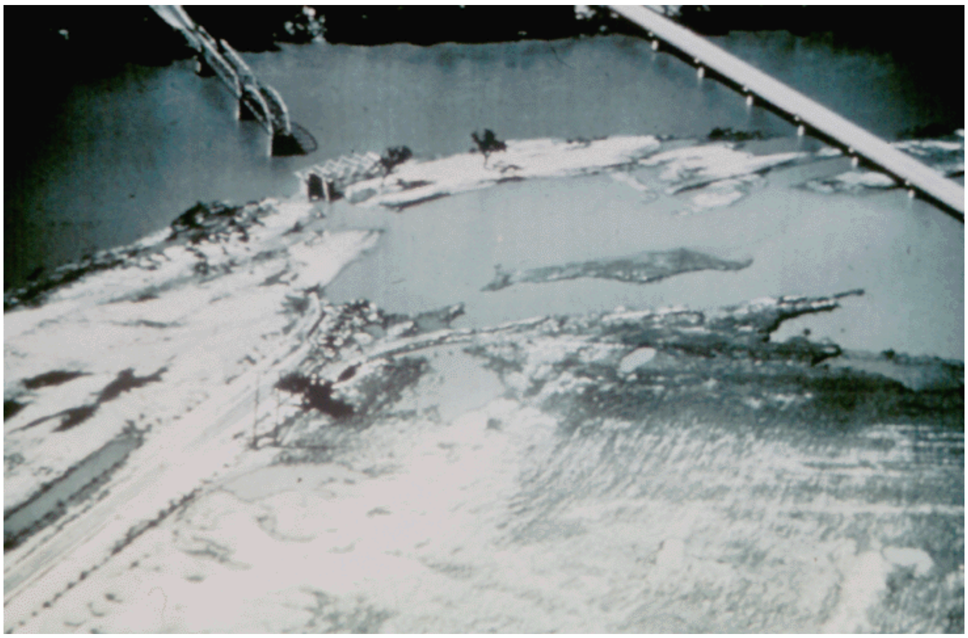

Figure 13.2.47 Approach Spans in the Floodplain

Figure 13.2.48 Debris and Sediment in the Channel

Other Features

- Check for streamflow impact of any other features such as tributaries, confluence of another waterway, dams, and substructure units from other bridges (see Figure 13.2.49). This may create conditions for high stream flow velocity through the bridge.
- Report any recent construction activity (e.g. causeways, fishing piers, and stranded vessels) which may affect stream flow under the bridge.

Figure 13.2.49 Upstream Dam

13.2.7 Evaluation

Scour Plan of Action

A plan of action is prepared to monitor any known and potential deficiencies and address any critical findings for bridges that are determined to be scour critical. Instructions regarding the type and frequency of inspections in regards to monitoring the performance and the closing of the bridge during or after flood events are included in the scour plan of action. A schedule for the design and construction of scour countermeasures if it is determined they are needed for the bridge are also included.

Scour Potential Assessment

Bridges over streams and rivers are subject to scour and are evaluated to determine their vulnerability to floods and to determine whether they are scour critical.

Purpose and Objective

In a scour evaluation, structural, hydraulic and geotechnical engineers have to make decisions on:

- Priorities for making bridge scour evaluations.
- The scope of the scour evaluations to be performed in the office and in the field.
- Whether a bridge is a scour critical bridge.
- Develop a plan of action for each scour critical bridge.
- Which scour countermeasures may reduce the bridge's vulnerability to scour.
- Which scour countermeasures are most suitable and cost-effective for a given bridge site.
- Priorities for installing scour countermeasures.

- Monitoring and inspecting scour critical bridges.

A responsibility of the bridge inspector is to gather on-site data for an assessment of scour potential, that:

- Accurately records the present condition of the bridge and the stream (see Figure 13.2.50).
- Identifies conditions that are indicative of potential problems with scour and stream stability.

To accomplish these objectives, the inspector needs to recognize and understand the potential for scour and its relationship with the bridge and stream. When an actual or potential scour problem is identified by a bridge inspector, further evaluation the bridge is completed by an interdisciplinary team made up of structural, geotechnical, and hydraulic engineers.

Figure 13.2.50 Scour at a Pile Abutment

Recognition of Scour Potential

Identify and record waterway conditions at the bridge, upstream of the bridge, and downstream of the bridge. Indications that could establish a scour potential include waterway, substructure and superstructure.

Waterway

- Stream flow velocity is a major factor in the rate of scour. High velocities produce accelerated scour rates (see Figures 13.2.51 and 13.2.52).
- Streambed materials such as loose cohesive soils, sand or gravel material, are highly susceptible to accelerated scour rates (see Figure 13.2.53).

- Orientation of waterway opening such as misaligned or skewed structure foundation elements, which can frequently generate adverse streamflow conditions, can lead to scouring of the streambed especially during flood flows (see Figure 13.2.53).
- Large floodplains constricted to a narrow hydraulic opening under a structure can result in accelerated scour during flood flow, due to high velocities and changes in local flow direction (see Figure 13.2.54).
- Banks that are sloughing, undermined, or moving laterally are signs of potential scour at a bridge (see Figure 13.2.55).

Figure 13.2.51 Fast Flowing Stream

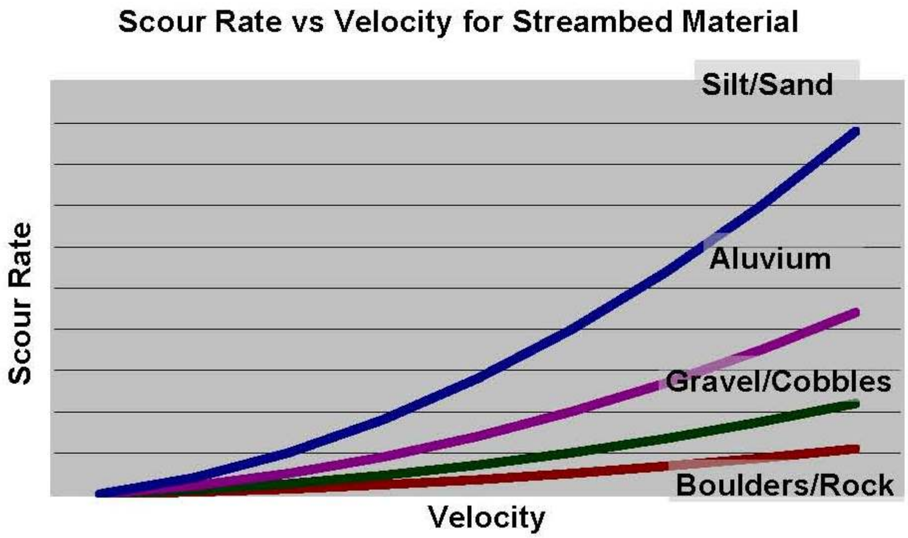

Figure 13.2.52 Scour Rates vs. Velocity for Common Streambed Materials

Figure 13.2.53 Typical Misaligned Waterway

Figure 13.2.54 Typical Large Floodplain

Figure 13.2.55 Lateral Stream Migration

Substructure

Consider the following condition of bridge foundations and substructure units in the scour potential assessment:

- Piers and abutments that are not parallel with the stream flow especially during flood flow conditions, can lead to local scour of foundations (see Figure 13.2.56).
- Rotational, horizontal, or vertical movement of piers and abutments are evidence of undermining (see Figure 13.2.57).
- Spread footing foundation levels above maximum calculated scour depth determined for a particular streambed material are subject to undermining and failure. Exposed piling can be damaged or deteriorated and can lead to failure. Loss of supporting surrounding soil can also diminish pile capacity (see Figure 13.2.58).
- Constriction of the general waterway opening beneath the structure due to numerous large piers or simply an inadequate span length between abutments can increase streamflow velocities and lead to contraction scour (see Figure 13.2.59).

Figure 13.2.56 Stream Alignment Not Parallel with Abutments

Figure 13.2.57 Rotational Movement and Failure Due to Undermining

Figure 13.2.58 Exposed Piling Due to Scour

Figure 13.2.59 Accelerated Flow Due to Constricted Waterway

Superstructure

Consider the following conditions associated with the superstructure in recognizing scour potential:

- Evidence of overtopping indicates insufficient hydraulic opening and excessive flow velocities.
- Insufficient freeboard can trap debris, increasing the potential for a washout.
- Simple span designs are most susceptible to collapse in the event of foundation movement or increased flows during a flood event.

NBI Condition Rating Guidelines

Scour Evaluation

The scour evaluation is an engineering assessment of existing and potential problems and making a sound judgement on what steps can be taken to eliminate or minimize future damage.

In assessing the adequacy of the bridge to resist scour, the inspector and engineer need to understand and recognize the interrelationships between several items. The inspector can expedite the engineers' evaluation by considering the following:

- Substructure Condition Rating (Item 60)
- Channel and Channel Protection Condition Rating (Item 61)
- Waterway Adequacy Appraisal Rating (Item 71)
- Scour Critical Bridges (Item 113)

See Topic 4.2 for a detailed description of NBI Condition Rating Guidelines.

Substructure (Item 60)

Substructure rating is a key item for rating the bridge foundations for vulnerability to scour damage. When a scour problem is found that has already occurred, considered it in the condition rating of the substructure. If the bridge is determined to be scour critical, further evaluate the condition rating for Item 60 to ensure that any existing problems have been properly considered. Be consistent with the rating factor given to Item 60 with the one given to Item 113 whenever a rating factor of 2 or below is determined for Item 113.

Channel and Channel Protection (Item 61)

This item permits rating the physical channel condition affecting streamflow through the bridge waterway. Consider the condition of the channel, adjacent rip-rap, bank protection, guidebanks, and evidence of erosion, channel movement or scour in establishing the rating for Item 61.

Waterway Adequacy (Item 71)

This is an appraisal item, rather than a condition item, and permits assessment of the adequacy of the bridge waterway opening to pass flood flows.

Scour Critical Bridges (Item 113)

This item permits a rating of current bridge conditions regarding its vulnerability to flood damage. A scour-critical bridge is one with abutment or pier foundations that are considered unstable due to:

- Observed scour at the bridge site, or
- Having scour potential as determined by a scour evaluation

When an actual or potential scour problem is identified, the bridge is to be further evaluated by an interdisciplinary team comprised of structural, hydraulic and geotechnical engineers.

In this process, the effects of a 100-year flood (a flood which has a one percent chance of occurring in any year) would be considered, but the effects of a "superflood" or 500-year flood would also be assessed and assigned to one of three conditions.

- Safe condition - if calculations indicate that the likely scour depth of the superflood would be above the top of the footing, the bridge would be considered safe or stable (see Figure 13.2.60).
- Evaluate condition - if calculations indicate a scour depth within the limits of a spread footing or piles, further structural or foundation evaluation may be needed to establish the likely stability of the foundation (see Figure 13.2.61).
- Fix condition - where there are indications that scour depth will lie below the bottom of the spread footing or piles, then the bridge would be considered clearly scour critical and would be at risk to damage or collapse (see Figure 13.2.62).

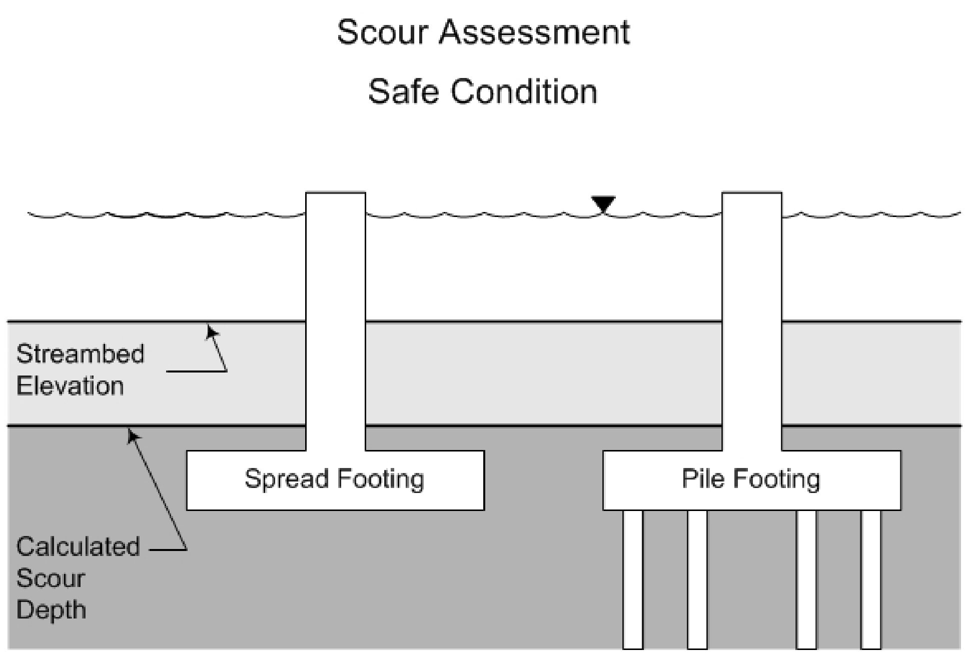

Figure 13.2.60 Scour Assessment - Safe

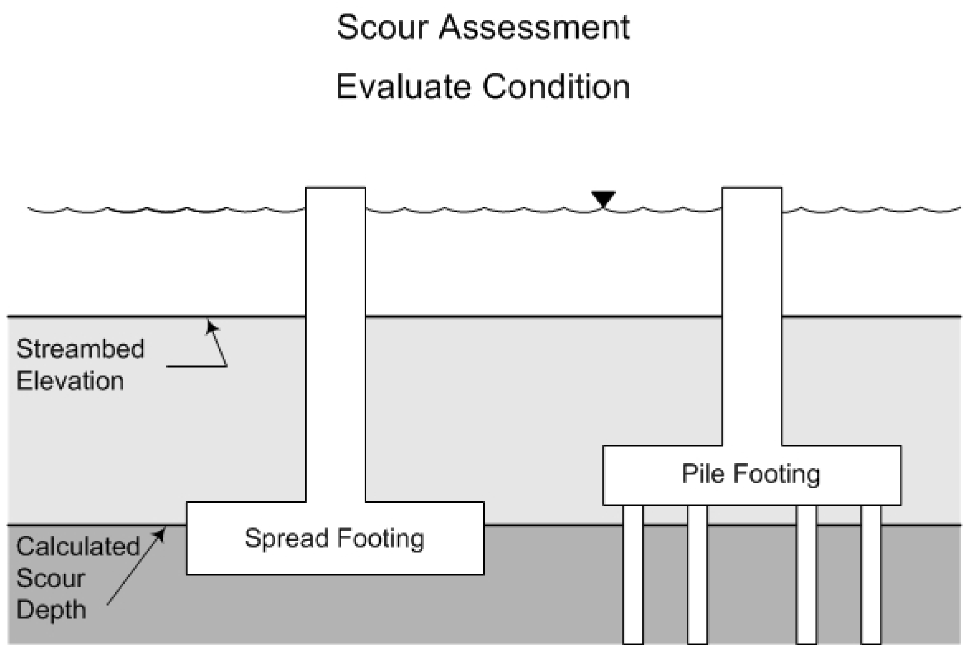

Figure 13.2.61 Scour Assessment - Evaluate

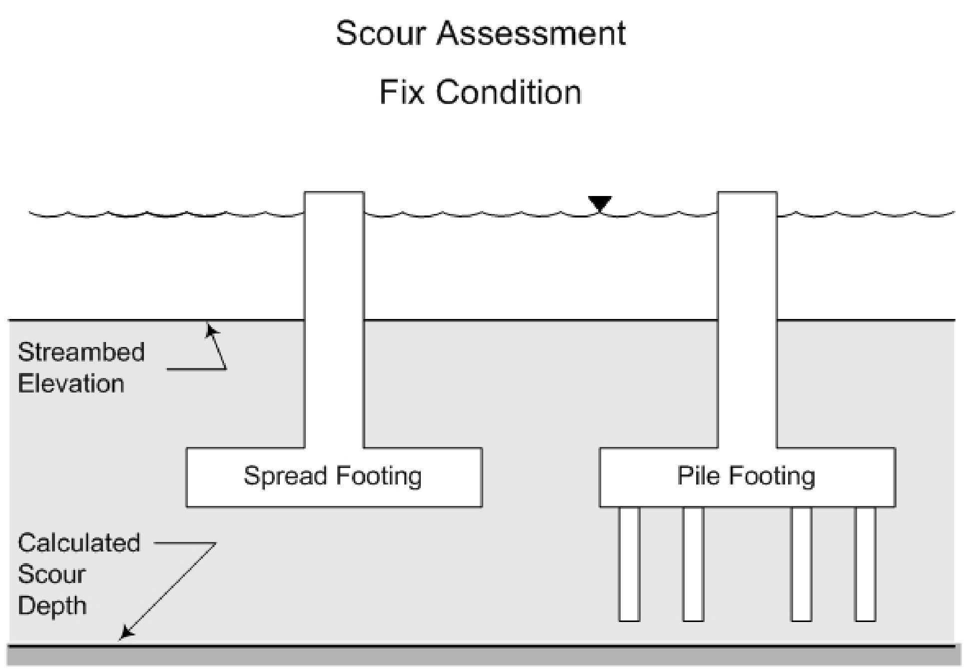

Figure 13.2.62 Scour Assessment - Fix

For scour critical bridges, the NBIS requires that a Plan of Action is developed for monitoring and correcting the scour problem. Monitor, in accordance with the plan, bridges which are scour critical. Such a plan would address the type and frequency of future inspections to be made and would include a schedule of timely design and construction actions for appropriate countermeasures to protect the bridge. The countermeasures might include the possibility of riprap, bed armoring, or flow-control structures or embankments.

Washouts of scour critical bridges, which appeared to be stable in the past, have still occurred. Recognizing potential problems and developing a Plan of Action for scour critical bridges will help reduce the likelihood of washouts.

13.2.8 Culvert Waterway

The following excerpt is from a reproduction of the out-of-print Culvert Inspection Manual (Supplement to Manual 70), July 1986 – Chapter 5, Section 3:

Section 3. WATERWAYS

5-3.0 General.

The primary function of most culverts is to carry surface water or traffic from one side of a roadway embankment to the other side. The hydraulic design of culverts usually involves the determination of the most economical size and shape of culvert necessary to carry the design discharge without exceeding the headwater depth allowable. It is essential that the culvert be able to handle the design discharge. If the culvert is blocked with debris or the stream changes course near the ends of the culvert, the culvert may be inadequate to handle design flows. This may result in excessive ponding, flooding of nearby properties, and washouts of the roadway and embankment. In addition changes in upstream land use such as clearing, deforestation, and real estate development may change the peak flow rates and stream stability. It is therefore important to inspect the condition of the stream channel, SI&A item 61, and evaluate the ability of the culvert to handle peak flows, SI&A item 71.

5-3.1 Stream Channel--What to Look for During Inspection.

The stream channel should be inspected to determine whether conditions exist that would cause damage to the culvert or surrounding properties. Factors to be checked include culvert location (horizontal and vertical alignment), scour, and accumulation of sediment and debris. These factors are closely related to each other. Poor culvert location can result in reduced hydraulic efficiency, increased erosion and sedimentation of the stream channel, and increased damage to the embankment and surrounding properties. A brief discussion of each of these factors is provided.

a. Horizontal Alignment - The inspector should check the condition of the stream banks and any bank protection at both ends of the culvert. He should also check for erosion and indications of changes in the direction of the stream channel. Sketches and photographs should be used to document the condition and alignment at the time of inspection. Abrupt stream alignment changes retard flow and may require a larger culvert; they cause increased erosion along the outside of the curve, damage to the culvert, and increased sedimentation along the inside of the curve. Where sharp channel curves exist at either the entrance or exit of a culvert, the inspector should check for sedimentation and erosion.

b. Vertical Alignment - Vertical alignment problems are usually indicated by scour or accumulation of sediment. Culverts on grades that differ significantly from the natural gradient may present problems. Culverts on flat grades may have problems with sediment build up at the entrance or within the barrel. Culverts on moderate and steep grades generally have higher flow velocities than the natural stream and may have problems with outlet scour. Scour and sediment problems may also occur if the culvert barrel is higher or lower than the streambed.

c. Scour - Erosion generally refers to loss of bank material and a lateral movement of the channel. Scour is more related to a lowering of the streambed due to the removal and transporting of stream bed material by flowing water. Scour may be classified into two types: local scour and general scour.

(1) Local scour is located at and usually caused by a specific flow obstruction or object, which causes a constriction of the flow. Local scour occurs primarily at the culvert outlet.

(2) General scour extends farther along the stream and is not localized around a particular obstruction. General scour can involve a gradual, fairly uniform degradation or lowering of the stream channel. It can also result in abrupt drops in the channel that move upstream during peak flows. This type of scour is referred to as head cutting. Head cutting may be a serious problem if it is occurring in the channel downstream from the culvert, since it may threaten the culvert as it moves upstream. Head cutting may also occur in the stream channel immediately upstream from depressed inlets. Where upstream head cutting is usually not as serious a problem for the culvert, it can affect upstream structures and properties.

The upstream channel should be checked for scour that may undermine the culvert or erode the embankment. Scour that is undermining trees or producing sediment that could block or reduce the culvert opening should also be noted. The stream channel below the culvert should be checked for local scour caused by the culvert's discharge and for general scour that could eventually threaten the culvert.

d. Accumulation of Sediment and Debris - Deposits of debris or sediment that could block the culvert or cause local scour in the stream channel should be noted. Accumulations of debris or sediment in the stream may cause scour of the streambanks and roadway embankment, or could cause changes in the channel alignment. Debris and sediment accumulations at the culvert inlets or within the culvert barrel reduce the culvert's capacity and may result in excessive ponding. It also increases the chances for damage due to buoyant forces. Downstream obstructions, which cause water to pond at the culvert’s outlet, may also reduce the culvert's capacity. Debris collectors are used in some culverts so that the opening is not blocked by floating materials.

5-3.2 Waterway Adequacy - What to Look for During Inspection.

The preceding paragraphs dealt with evaluating the condition of the stream channel and identifying conditions that could cause damage to the culvert or reduce the hydraulic efficiency of the culvert. A closely related condition that must be evaluated is the waterway adequacy or ability of the culvert to handle peak flows, changes in the watershed, and changes in the stream channel which might affect the hydraulic performance. Guidelines for rating SI&A item 71, Waterway Adequacy, are presented in the Coding Guide.

a. High Water Marks - The high water elevation will vary with each flood but should still be checked to evaluate waterway adequacy. Ideally, culverts should be checked during or immediately after peak flows to determine whether water is being ponded to excessive depths, flooding adjoining properties, or overflowing the roadway, as shown in Exhibit 63. High water marks are needed to define the upstream pond elevation and the downstream tailwater elevation. Several high water marks should be obtained, if possible, to insure consistency. High water marks in the culvert barrel, in the drain down area near the inlet, or near turbulent areas at the outlet are generally misleading. An inspection can also determine high water levels for peak flows by looking for debris caught on fences, lodged in trees, or deposited on the embankment. Information may also be obtained by interviewing area residents. Indications of excessive ponding, flooding, or overtopping of the roadway should be investigated to determine the cause. If the cause is apparent, such as a blocked inlet, it should be reported for scheduling of appropriate maintenance. If the cause is not apparent, the culvert should be reported for evaluation by a hydraulic specialist.

b. Drainage Area - The inspector should be aware that changes in the drainage might have an effect on the discharge that culverts must handle. Replacement of an upstream culvert with a larger structure may eliminate upstream ponding, causing more water to reach the culvert sooner. Land clearing, construction, channel improvements, or removal of upstream dams or sediment basins may also affect discharge rates. Similarly, changes in land use may increase or decrease the amount of rainfall that infiltrates the ground and the amount that runs off. The inspector should note in the inspection report any apparent changes that are observed and be aware that changes a considerable distance upstream may affect the performance of downstream structures. Obstructions downstream from a culvert that back water up to the culvert may also affect the performance of the culvert.

c. Scour - As previously discussed, scour that changes the stream alignment at the ends of the culvert can reduce the hydraulic efficiency.

d. Sedimentation and Debris - Accumulation of debris and sediment at the inlet or within the culvert barrel reduces both the size of the opening and the culvert's capability to handle peak flows. Severe drift and sediment accumulations are illustrated in Exhibits 64 and 65. However, culverts are occasionally designed with fill in the bottom to create a more natural streambed for fish.

Figure 13.2.63 (Exhibit 63) Culvert Failure Due to Overtopping

Figure 13.2.64 (Exhibit 64) Culvert Almost Completely Blocked by Sediment Accumulation

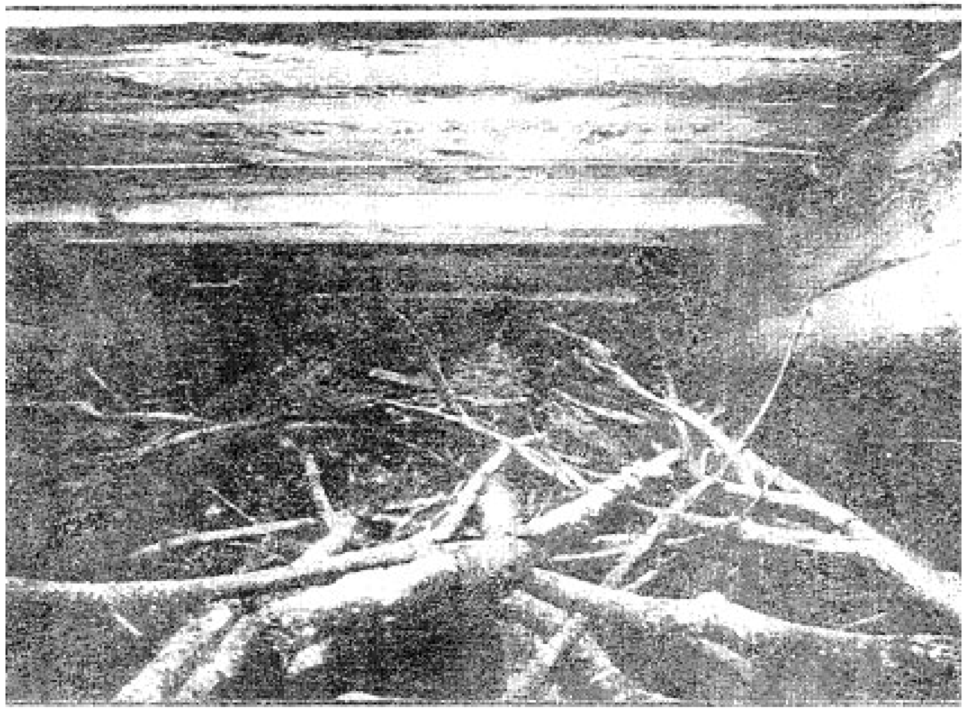

Figure 13.2.65 (Exhibit 65) Drift and Debris Inside Timber Box Culvert

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Table of Contents

Chapter 13

Inspection and Evaluation of Waterways

Topic 13.3 Underwater Inspection

13.3.1 Introduction

The need for underwater inspections is great. Approximately 83 percent of the bridges in the National Bridge Inventory (NBI) are built over waterways. While many of these bridges do not have foundation elements actually located in water, a great many do and most bridge failures occur because of underwater issues. Inspect underwater members to the extent necessary to determine with certainty that their condition has not compromised the structural safety of the bridge.

Several bridge collapses during the 1980's, traceable to underwater deficiencies, have led to revisions in the National Bridge Inspection Standards (NBIS) (see Figure 13.3.1). As a result, bridge owners are required to develop a master list of bridges requiring underwater inspections.

Figure 13.3.1 Schoharie Creek Bridge Failure

According to the NBIS, underwater inspection is the inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.

The expense of such inspections necessitates careful consideration of candidate bridge, since underwater inspection is a hands-on inspection requiring underwater breathing apparatus and related diving equipment.

Bridge Selection Criteria

Bridges that cross waterways often have foundation elements located in water to provide the most economical total design. Where these elements are continuously submerged (see Figure 13.3.2), use underwater inspection and management techniques to establish their condition so that failures can be avoided.

Figure 13.3.2 Liberty Bridge over Monongahela River

In many cases, a multi-disciplinary team including structural, hydraulic and geotechnical engineers evaluate a bridge located over water that is a candidate for underwater inspection. Underwater inspection is therefore only one step in the total investigation of a bridge.

Selection Criteria

Various factors influence the underwater bridge inspection selection criteria. In accordance with the *Code of Federal Regulations* (23 CFR Part 650) and the *AASHTO Manual for Bridge Evaluation (MBE)*, all structures receive routine underwater inspections at intervals not to exceed 60 months, or 72 months with FHWA approval. This is the maximum interval permitted between underwater inspections for bridges which are in excellent condition underwater and which are located in passive, nonthreatening environments. More frequent routine and in-depth inspections may be desirable for many structures and necessary for critical structures. The bridge owner determines the inspection interval that is appropriate for each individual bridge. Factors to consider in establishing the inspection frequency and levels of inspection include:

- Age (*CFR* and *MBE*)
- Traffic volume (*MBE*)
- Size (*MBE*)
- Susceptibility to collision (*MBE*)
- Extent of deterioration (*MBE*)
- Performance history of bridge type (*MBE*)
- Load rating (*MBE*)
- Location (*MBE*)
- National defense designation (*MBE*)

- Detour length (*MBE*)
- Social and economical impacts due to bridge being out of service (*MBE*)
- Type of construction materials (*CFR*)
- Environment (*CFR*)
- Scour characteristics (*CFR*)
- Condition ratings from past inspections (*CFR*)
- Known deficiencies (*CFR*)

Selected Bridges

Note those bridges that require underwater inspection on the bridges' individual inspection and inventory records. For each bridge requiring underwater inspection, include the following information as a minimum:

- Type and location of the bridge
- Type and frequency of required inspection
- Location of members to be inspected
- Inspection procedures to be used
- Dates of previous inspections
- Maximum water depth and velocity (if known)
- Special equipment requirements
- Findings of the last inspection
- Follow-up actions taken on findings of the last inspection
- Type of foundation
- Bottom of foundation elevation or pile tip elevation
- Include each bridge on a master list for the Agency

13.3.2 Diving Inspection Intensity Levels

Originating in the offshore diving industry and adopted by the United States Navy, the designation of standard levels of inspection has gained widespread acceptance. Three diving inspection intensity levels have evolved as follows:

- Level I: Visual, tactile inspection
- Level II: Detailed inspection with partial cleaning
- Level III: Highly detailed inspection with non-destructive testing (NDT) or partially destructive testing (PDT)

Routine underwater inspections normally include a 100 percent Level I inspection and a 10 percent Level II inspection, but it may include a Level II and Level III inspection to determine the structural condition of any submerged portion of the substructure with certainty.

Level I

Level I inspection consists of a close visual inspection at arm's length with minimal cleaning to remove marine growth of the submerged portions of the bridge. This level of inspection is used to confirm the continuity of the members and to detect any undermining or elements that may be exposed that would normally be buried. Although the Level I inspection is referred to as a "swim-by" inspection, it needs to be detailed enough to detect obvious major damage or deterioration. A Level I inspection is normally conducted over the total (100%) exterior surface of each underwater element, involving a visual and tactile inspection with limited probing of the substructure and adjacent streambed. In areas where light is minimal, handheld lights may be needed. If the water clarity is poor enough that the inspector cannot inspect the member visually, a tactile inspection may be performed by making a sweeping motion of the hands and arms to cover the entire substructure.

The results of the Level I inspection provide a general overview of the substructure condition and verification of the as-built drawings. The Level I inspection can also indicate the need for Level II or Level III inspections and aid in determining the extent and the location of more detailed inspections.

Level II

Level II inspection is a detailed inspection that requires that portions of the structure be cleaned of marine or aquatic growth. In some cases, cleaning is time consuming, particularly in salt water, and needs to be restricted to critical areas of the structure. However, in fresh water, aquatic coatings can be removed by just wiping the structural element with a glove.

Generally, the critical areas are near the low waterline, near the mud line, and midway between the low waterline and the mud line. On pile structures, horizontal bands, approximately 6 to 12 inches in height, preferably 10 to 12 inches, need to be cleaned at designated locations:

- Rectangular piles - the cleaning includes at least three sides
- Octagonal piles - at least six sides
- Round piles - at least three-fourths of the perimeter
- H-piles - at least the outside faces of the flanges and one side of the web

Figure 13.3.3 Level II Cleaning of a Steel Pile

On large elements, such as piers and abutments, clean areas at least 1 square foot in size at three or more levels on each face of the element (see Figure 13.3.4). For a structure that is greater than 50 feet in length, clean an additional three levels on each exposed face. It is important to select the locations to clean to help minimize any potential damage to the structure and to target more critical locations. Measure and document deficient areas, including both the extent and severity of the damage.

It is intended to detect and identify high stress, damaged and deteriorated areas that may be hidden by surface growth. A Level II inspection is typically performed on at least 10% of all underwater elements. Govern the thoroughness of cleaning by what is necessary to determine the condition of the underlying material. Complete removal of all growth is generally not required.

Figure 13.3.4 Diver Cleaning Pier Face For Inspection

Level III

A Level III inspection is a highly detailed inspection of a critical structure or structural element, or a member where extensive repair or possible replacement is contemplated. The purpose of this type of inspection is to detect hidden or interior damage and loss in cross-sectional area. This level of inspection includes extensive cleaning, detailed measurements, and selected nondestructive and other testing techniques such as ultrasonics, sample coring or boring, physical material sampling, and in-situ hardness testing. The use of testing techniques is generally limited to key structural areas; areas that are suspect; or areas that may be representative of the entire bridge element in question.

13.3.3 Types of Inspection

A comprehensive review of all bridges contained in an agency's inventory will indicate which bridges require underwater inspection. Many combinations of waterway conditions and bridge substructures exist. For any given bridge, the combination of environmental conditions and structure configuration can significantly affect the requirements of the inspection. It is generally accepted that there are five different types of inspections used for underwater inspections:

- Routine underwater (periodic)
- Initial (inventory)
- Damage
- In-depth
- Special (interim)

Underwater inspections are typically either routine or in-depth inspections.

Routine Underwater (Periodic) Inspections

A routine underwater (or periodic) inspection is a regularly scheduled, intermediate level inspection consisting of sufficient observations and measurements to determine the physical and functional condition of the bridge, to identify any change from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements. A routine underwater inspection will incorporate Level I, Level II and a scour evaluation.

The summary guidelines for a routine underwater inspection include:

- A Level I inspection will be conducted on 100% of the underwater portion of the structure to determine obvious problems.
- A Level II inspection will be conducted on at least 10% of underwater units selected as determined by the Level I inspection.
- Scour evaluations which help give the cross-section of the channel by sounding and probing near the elements underwater

The dive team may also conduct a scour evaluation at the bridge site, including:

- Inspect the channel bottom and sides for scour.
- Cross sections of the channel bottom will be taken and compared with as-built plans or previously taken cross sections to detect lateral channel movement or deepening (see Figure 13.3.5).
- Soundings are to be made in a grid pattern (see Figure 13.3.6) about each pier and upstream and downstream of the bridge, developing contour

elevations of the channel bottom, to detect areas of scour. Permanent reference point markers can be placed on each abutment/pier (see Figure 13.3.7). Data obtained from the soundings will be correlated with the original plans (if available) of the bridge foundations and tied to these markers for reference during future underwater inspections.

- Local scour and undermining can be determined with probes in the vicinity of piers and abutments (see Figure 13.3.8). In streams carrying large amounts of sediment, reliable scour depth measurements may be difficult at low flow due to scour hole backfilling.

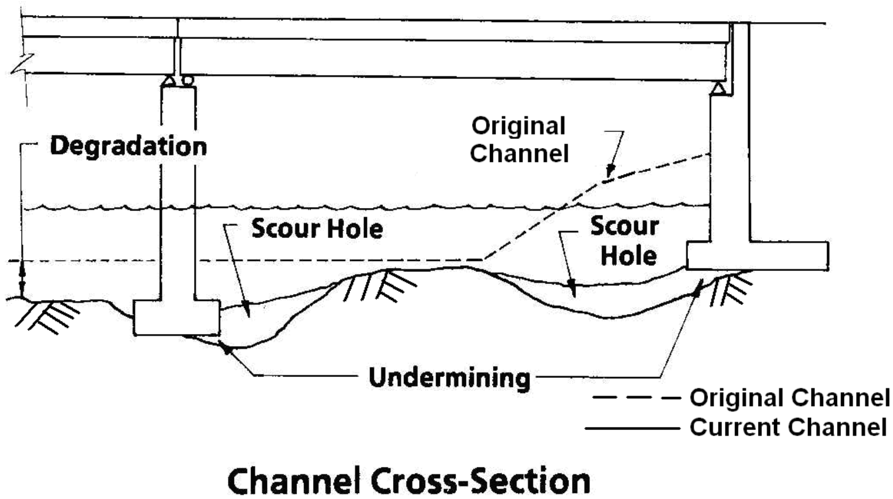

Figure 13.3.5 Channel Cross-Section (Current Inspection Versus Original Channel)

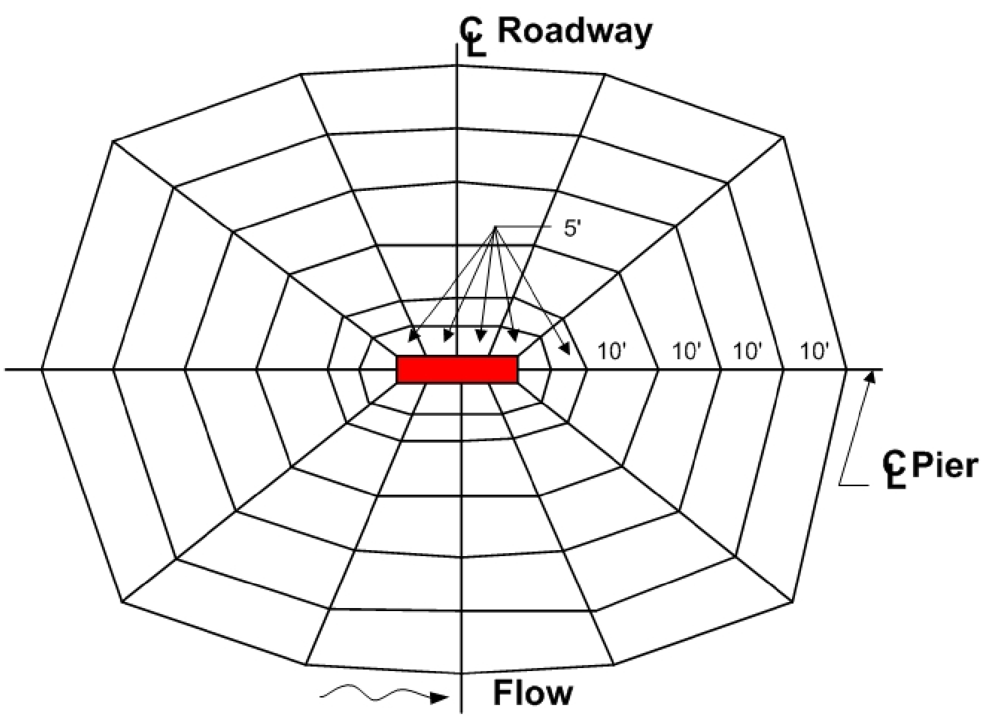

Figure 13.3.6 Pier Sounding Grid

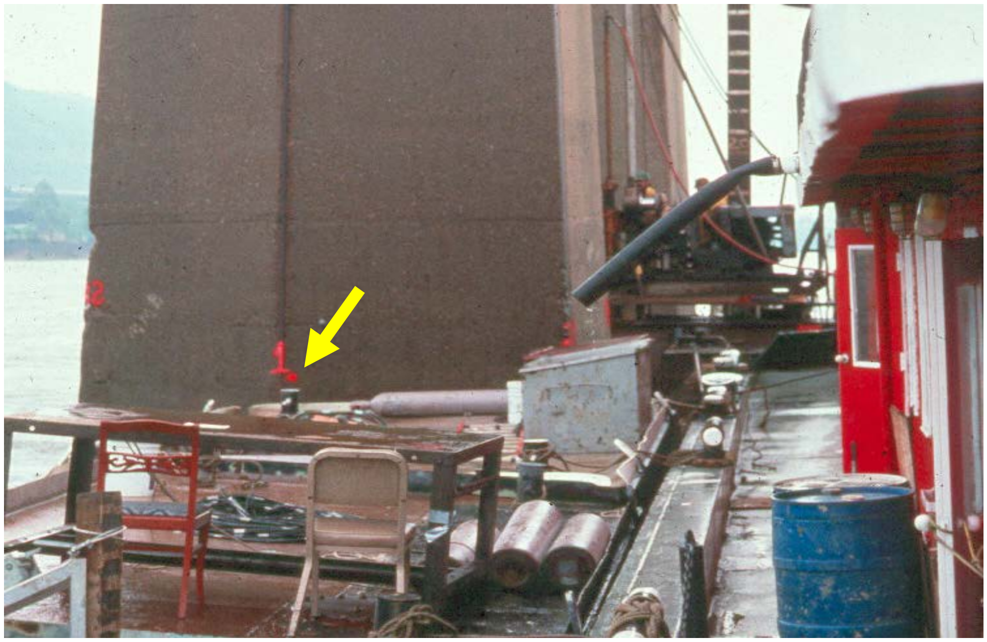

Figure 13.3.7 Permanent Reference Point (Bolt Anchored in Nose of the Pier, Painted Orange)

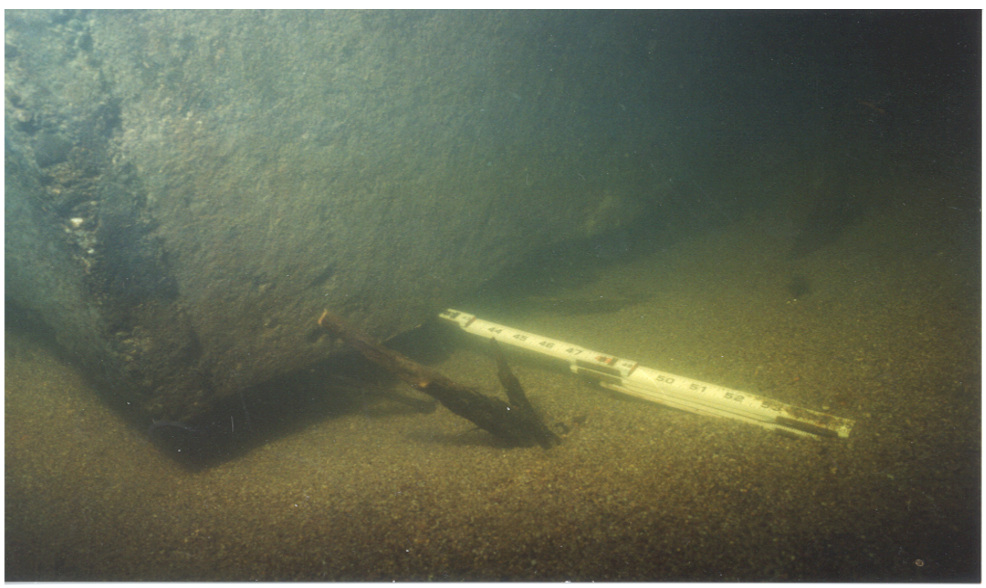

Figure 13.3.8 Local Scour; Causing Undermining of a Pier

Initial (Inventory) Inspections

An initial (or inventory) inspection is the first inspection of a bridge as it becomes a part of the bridge inventory. An initial inspection is a fully documented investigation that will typically incorporate Level I and Level II inspections and a scour evaluation as required for a routine inspection. In addition, this type of inspection will provide all of the Structure Inventory and Appraisal (SI&A) and other relevant data to determine the baseline structural conditions. It also identifies and lists the existing problems and locations of existing problems or locations in the structure that may have potential problems. Aided by a prior detailed review of plans, it is during this inspection that any underwater members (or details) are noted for subsequent focus and special attention (see Figure 13.3.9).

An initial inspection may also be required when there has been a change in the configuration of the structure such as widening, lengthening, bridge replacement, or change in ownership.

Figure 13.3.9 Bascule Bridge on the Saint Croix River

Damage Inspections

Certain conditions and events affecting a bridge may require more frequent or unscheduled inspections to assess structural damage resulting from environmental or accident related causes.

The scope of the inspection is to be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic and to assess the level of effort necessary to repair the damage. The amount of effort expended on this type of inspection will vary significantly depending upon the extent of the damage. If major damage has occurred, evaluate section loss, make measurements for misalignment of members, and check for any loss of foundation support.

In-Depth Inspections

An in-depth inspection is a close-up, hands-on inspection of one or more members below the water level to detect any deficiencies not readily apparent using routine inspection procedures. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiencies, Level III, nondestructive tests may need to be performed.

The in-depth inspection typically includes Level II inspection over extensive areas and Level III inspection of limited areas. Nondestructive testing is normally performed, and the inspection may include other testing methods, such as extracting samples for laboratory analysis and testing, boring, and probing.

One or more of the following conditions may dictate the need for an in-depth inspection:

- Inconclusive results from a routine inspection
- Critical structures, whose loss would have significant impact on life or property

- Unique structures, whose structural performance is uncertain
- Prior evidence of distress
- Consideration of reuse of an existing substructure to support a new superstructure or planned major rehabilitation of the superstructure

The distinction between routine and in-depth inspections is not always clearly defined. For some bridges, such as steel pile supported structures in an actively corrosive environment, it may be necessary to include Level III, nondestructive testing inspection techniques as part of routine inspections.

Special (Interim) Inspections

A special (or interim) inspection is scheduled at the discretion of the individual in charge of bridge inspection activities. A special inspection is used to monitor a particular known or suspected deficiency (e.g., foundation settlement or scour).

Conditions for Inspections

Situations that may warrant a damage, in-depth or a special underwater inspection include:

- Unusual floods - inspect bridge elements after floods. Inspect bridge elements located in streams, rivers, and other waterways with known or suspected scour potential after every major runoff event to the extent necessary to ensure bridge foundation integrity (see Figure 13.3.10).
- Vessel impact - inspect bridges underwater if there is visible damage above water from vessel impact. This is to be done in order to determine the extent of damage and to establish the extent of liability of the vessel owner for damages.
- Unusual ice floes - ice floes can damage substructure elements, and accumulations of ice on the elements can cause scouring currents or increase the depth of scour.
- Prop wash from vessels - prop wash from vessels (i.e., turbulence caused by the propellers of marine vessels) can cause scouring currents and may propel coarse-grained bottom materials against substructure elements in a manner similar to that of blast cleaning operations.
- Adverse environmental conditions - rapid and severe deterioration of substructure materials may be caused by brackish water, polluted water, and water with high concentrations of chemicals. Some waterways may promote microbial induced corrosion (MIC) on steel submerged substructure elements.
- Floating and build-up of debris - buildup of debris at piers or abutments effectively widens the unit and may cause scouring currents or increase the depth of scour (see Figure 13.3.11).
- Above water evidence of deterioration or movement - evidence of deterioration or movement will require underwater inspection. Many underwater deficiencies only become apparent above water when the distress extends above the waterline or is manifested by lateral movement or settlement. Inspect bridges underwater following significant earthquakes (see Figure 13.3.12).

Figure 13.3.10 Flood Conditions: Pier Settlement.

Figure 13.3.11 Buildup of Debris At Pier

Figure 13.3.12 Movement of a Substructure Unit

Frequency of Inspection

Conduct routine inspections of substructures in water at least once every 60 months. This is only applicable to substructures that are in excellent condition and for substructures that have current conditions that are considered acceptable for that timeframe and without any concerns that may require more frequent monitoring.

Structures having underwater members which are partially deteriorated or are in unstable channels may require shorter inspection intervals. Establish criteria for determining the level and frequency to which these underwater elements will be inspected base on such factors as:

- Structure age
- Type of construction materials
- Configuration of the substructure
- Adjacent waterway features such as dams, dikes or marinas
- Susceptibility of streambed materials to scour
- Maintenance history
- Saltwater pollution
- Damage due to waterborne traffic, debris or ice

Certain underwater structural elements may be inspected at greater than 60-month intervals, not to exceed 72 months, with written FHWA approval. This may be appropriate when past inspection findings and analysis justifies the increased inspection interval.

Some bridge owners, however, may shorten the underwater inspection interval to 24 or 48 months to coincide with a regular routine bridge inspection (24 to 48 months) or a fracture critical member inspection (24 months).

13.3.4 Qualifications of Diver-Inspectors

An underwater bridge inspection diver needs to complete an FHWA-approved comprehensive bridge inspection training course or other FHWA-approved underwater bridge inspection training course.

The underwater inspector needs to have knowledge and experience in bridge inspection. Conduct all underwater inspections under the direct supervision of a qualified bridge inspection team leader. A diver not fully qualified as a bridge inspection team leader is to be used under close supervision.

The ability of the underwater inspector to safely access and remain at the underwater work site is paramount to a quality inspection. The individual is to possess a combination of commercial diving training and experience as a working diver. This allows the inspector to meet the particular challenges of the underwater working conditions for that inspection.

Team leader requirements for those in charge of underwater inspection are the same as their top-side counterparts. See Topic 1.2 Responsibilities of the Bridge Inspector and *Title 23 of the Code of Federal Regulations, Part 650, Subpart C*.

Federal Commercial Diving Regulations

Underwater bridge inspection, using either self-contained or surface-supplied equipment, is a form of commercial diving. In the United Sates, commercial diving operations are federally regulated by the Occupational Safety and Health Administration (OSHA). OSHA regulates all commercial diving operations performed inland and on the coast (through *Title 29 of the Code of Federal Regulations, Part 1910, Subpart T, Commercial Diving Operations*). Consult this reference for details on commercial diving procedures and safety.

Diver Training and Certification

OSHA Safety Requirements

The OSHA Commercial Diving Operations standard applies to all diving and related support operations that are conducted in connection to diving. The OSHA delineates diving personnel requirements, including general qualifications of dive team members. The standard also provides general and specific procedures for diving operations, and provides requirements and procedures for diving equipment and recordkeeping:

- Personnel requirements - all divers are to be trained, which includes dive physiology, first aid, and cardiopulmonary resuscitation (CPR)
- General and specific operating procedures - employers are to develop a safe diving practices manual for diving operations which includes the following:
  - Designate a person to be in charge of the operation who is qualified by training and experience
  - SCUBA gear is not to be used for depths greater than 130 feet sea water
  - SCUBA gear is not to be used in currents that are exceeding 1 knot unless diver is line tended
  - Surface-supplied air is not allowed for depths greater than 220 feet sea water
  - A recompression chamber needs to be on-site and ready for use for dives that exceed 100 feet sea water and for dives that are outside the no-decompression limits,
  - Minimum size dive team for both SCUBA and surface-supplied diving operations is to be three, but more personnel may be required
- Equipment procedures - minimum equipment requirements are specified for diver and diving equipment, equipment testing and requirements
- Record keeping requirements - requirements are set for the recording and retaining of documents related to diving related illness, injuries, or fatalities, diving exposure, decompression evaluations, medical treatment, equipment inspection and testing, and depth-time profiles

U.S Army Corp Army of Engineers Safety and Health Requirements is another safety standard that may be used and is similar to OSHA standards, except it provides more specific guidance as to the minimum dive team personnel required for various diving conditions. It also provides a more definitive requirement for diving qualifications and requires that divers be certified in the emergency administration of oxygen.

Visit www.osha.gov for more information.

ANSI Standards for Commercial Diver Training

American National Standards Institute (ANSI) Standards exist, which define minimum training standards for both recreational SCUBA and commercial divers. These standards provide clear-cut distinctions between recreational and commercial diver training. While not federal law, these standards constitute the consensus of both the recreational and commercial diving communities, following ANSI's requirements for due process, consensus, and approval.

The American National Standard for Divers- Commercial Diver Training- Minimum Standard (ANSI/ACDE-01-1998) requires a formal course of study, which contains at least 625 hours of instruction. This training may come from an accredited commercial diving school, military school, or may be an equivalent degree of training achieved prior to the effective date of the Standard, which includes a documented combination of field experience and/or formal classroom instruction. Visit www.ansi.org/ for more information.

ADC International Requirements

The Association of Diving Contractors International (ADC) is a non-profit organization representing the commercial diving industry. The ADC publishes "Consensus Standards For Commercial Diving Operations", which have been developed to present the minimum standards for basic commercial diving operations conducted either offshore or inland. The Consensus Standards, in part, duplicate the ANSI standard for commercial diver training, but subdivide the minimum 625 hours of training into both a formal course of study (317 hours, minimum), and on the job training (308 hours, minimum). The ADC also formally issues OSHA-recognized Commercial Diver Certification Cards to individuals meeting minimum training standards. Visit www.adc-int.org/ for more information.

Dive Team Requirements

The Federal Highway Administration's main concern is whether the diver has knowledge and experience in underwater bridge inspection. The individual employers are in the best position to determine the specific requirements of their dive teams.

13.3.5 Planning an Underwater Inspection

The primary goal for an underwater inspection is for the dive team to complete the work safely and to perform a complete and accurate inspection. Planning for underwater bridge inspections is particularly important because of:

- The complexity and potential hazards involved in conducting the inspection
- Unknown factors which may be discovered during the diving
- The difficulty for the bridge owner to verify the thoroughness of the inspection
- The cost of conducting underwater inspections

These factors are most influential for first-time (initial) underwater inspections that set a benchmark for future inspections. Therefore, it is important to distinguish between the first-time and follow up inspections.

The effectiveness of an underwater inspection depends on the agency's ability to properly consider the following factors:

- Method of underwater inspection (i.e., Dive mode)
- Diving inspection intensity level
- Type of inspection
- Qualifications of diver-inspectors
- Specific bridge site conditions, including access requirements, and waterway and climate conditions

With these factors considered, an agency may opt for a lower level of inspection. Depending on conditions and the type of damage found, a higher level may then be necessary to determine the actual bridge condition. It is also possible that different levels may be required at various locations on the same bridge.

The steps in planning an underwater inspection include:

- Preliminary planning - This determines the goal of the inspection and decides what information is to be gathered and the amount of detail.
- Data collection and research - The next step is to obtain design and as-built drawings of the bridge to determine the configuration of the structure, construction materials and foundation type. It is also important to include any past records of repairs as well as past inspection reports. This may help indicate the progression of any deficiencies, deterioration of repairs, waterway conditions and access points. Also, any scour data or plan of action is to be reviewed.
- Hazard analysis - When planning an inspection, it is recommended that the site be examined to identify all potential hazards and ways to work around these hazards. Planning may include the avoidance or removal of the hazard, the selection of appropriate operational methods, the choice of the appropriate inspection and diving equipment and the use of special protective equipment. Common hazards may include swift current, deep water, high altitudes, extreme water temperatures, limited or no visibility, marine wildlife, contaminated water, ice floes or fixed ice, floating or accumulated debris, watercraft operations or construction operations.
- Dive inspection operations plan - This will include team member assignments and responsibilities, inspection procedures and objectives, equipment requirements, emergency information and procedures, and a review of potential hazards and mitigation techniques that will be used.
- Risk assessment - This is a qualitative process that evaluates the hazardousness of the proposed underwater inspection based on the parameters that relate the inspection team characteristics and the demands the diving operation will take.

Quality Control and Quality Assurance

To aid with quality control (QC) and Quality Assurance (QA) and to ensure procedures are in place and followed, check lists have been developed to aid the bridge owner and the dive team. See Figure 13.3.3.

Bridge Owner's Underwater Inspection Plan Checklist

Bridge Identification

- [ ] Bridge Name
- [ ] Structure ID
- [ ] Owner
- [ ] Route
- [ ] Milepoint
- [ ] Latitude
- [ ] Longitude
- [ ] Underwater inspection interval
- [ ] Points of Contact (name and phone number) for immediate action such as closing the bridge based on findings:____________________________

____

Marine Information

- [ ] Waterway Name
- [ ] Navigable? (Y/N)__________

  If so:
  - [ ] Waterway river point
  - [ ] Inspection coordination contacts (names, agencies, phone #s, required lead time for notification)
- [ ] Type of water - salt/fresh/brackish
- [ ] Anticipated dive depths: _______
- [ ] Anticipated current: _______
- [ ] Anticipated water visibility: ________
- [ ] Other waterway concerns or items to note, i.e., presence near military facility, tribal fishing, water quality concerns, historic presence of logjams, etc.

Scour Information

- [ ] Is bridge Scour Critical? _______
- [ ] Current Bridge Scour Code (Item 113): ____
- [ ] Is Plan of Action (POA) in place?
- [ ] Are scour mitigation/countermeasures present? (Locations, types and significance)
- [ ] Are scour monitoring devices present? (Locations and types)

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Figure 13.3.13 Bridge Owner's Underwater Inspection Plan Checklist

Underwater Bridge Inspection—NHI Course 130091 Reference Manual

Structure Information

- ☐ Type of Superstructure
  - Main Spans
  - Approach Spans
- ☐ Type of Substructure
  - Abutments
  - Piers
  - Foundations

Inspection Information

- ☐ Date of last inspection
- ☐ Findings and necessary follow-up from previous inspection
- ☐ Routine Inspection codes for:
  - Substructure Condition ______
  - Superstructure Condition ______
  - Channel and Channel Protection ______
  - Waterway Adequacy ______
- ☐ Special equipment necessary

Dive Team Certification Requirements

- ☐ Team Leader
  - ☐ NBIS requirements
  - ☐ Professional Engineer
  - ☐ Successful completion of underwater bridge inspection course
  - ☐ OSHA qualified diver
- ☐ Team Members
  - ☐ Engineer-diver
  - ☐ Successful completion of comprehensive bridge inspection course
  - ☐ Successful completion of underwater bridge inspection course
  - ☐ OSHA qualified diver

Inspection Requirements (Directions to dive team)

- ☐ Specify level of inspection (I, II or III) and amount of coverage of in-water elements

  Ex: Level I, 100%

  Level II, at three elevations on 10% of piles, and four locations at three elevations per substructure unit

  Level III,__________________

8-16

Figure 13.3.13 Bridge Owner's Underwater Inspection Plan Checklist (cont'd.)

Underwater Bridge Inspection—NHI Course 130091 Reference Manual

☐ Specify Scour Inspection
- Soundings around substructure units
- Cross sections at upstream and downstream fascias
- Cross sections at ___ ft and ____ft upstream and downstream of fascias
- Profile along thalweg of waterway for ___ ft upstream and downstream of fascias
- Fathometric survey with plotted contour lines extending ___ft upstream and downstream of fascias

☐ Substructure elements to be inspected by divers____________________
____
____

☐ If an element is identified to be inspected underwater is not in water,
____

☐ Required Dive Mode
- ☐ Scuba
- ☐ Scuba with communication
- ☐ Surface supplied air with communication

☐ Reference Datum ____

☐ Criteria for Underwater Photographs
- Minimum number per substructure unit
  - Typical conditions
  - Conditions rated less than ____

☐ Check and document condition of structural members looking for cracks, spalling, abrasion, corrosion, exposed reinforcing steel, and undermining

☐ Document depth, length, height, and location of exposed or undermined portions of the foundations. Record number of exposed piles for footings supported by piles.

☐ Photography requirement

☐ Video requirement

☐ Check for and document presence and effectiveness of scour mitigation/countermeasures

☐ Sounding requirement

☐ Acoustic imaging

☐ Check for and document presence, condition, and operability of scour monitoring devices

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Figure 13.3.13 Bridge Owner's Underwater Inspection Plan Checklist (cont'd.)

Underwater Bridge Inspection—NHI Course 130091 Reference Manual

☐ Examine streambed and channel for stability, especially around substructure units

☐ Note presence of debris build-up

Criteria for Communications from Dive Team

☐ In event of critical/emergency condition

☐ Frequency otherwise: start, finish, daily?

____

____

Report Requirements

☐ Document findings from inspection in report

☐ Record pertinent inspection environment—current, depths, visibility, equipment used, etc.

☐ Comment on any recommendations and justification for changes that need to be made to Superstructure, Substructure, Channel & Channel Protection, Scour and/or Waterway Adequacy codes based on this inspection.

☐ Document recommendations for needed repairs and their urgency

☐ Include plan and elevation of the structure with important features highlighted

☐ Include definitions referred to in this document for Levels of Inspection, degree of corrosion, urgency of repairs, etc.

☐ Other ____

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Figure 13.3.13 Bridge Owner's Underwater Inspection Plan Checklist (cont'd.)

13.3.6 Substructure Units and Elements

The underwater portions of bridge structures can be classified into the following categories: bents, piers, abutments, caissons, cofferdams, protection devices and culverts. Proper identification is important since various elements may require different inspection procedures, levels of inspection, or inspection tools.

Bents

Bents can be divided into two groups:

- Column bents
- Pile bents

Column bents have two or more columns supporting the superstructure and may in turn be supported by piling below the mud line. The column bents are typically constructed of concrete, but the piling may be timber, concrete, or steel.

Pile bents carry the superstructure loads through a pile cap, into the piles and directly to the underlying soil or rock. The piles (and pile cap) can be constructed of timber, steel, or concrete. Pile bents are generally distinguished from piers by the presence of some battered piles and also bracing which provides stability for the individual piles. See Figures 13.3.14 through 13.3.16 for photographs of pile bents of different material types.

See Topic 12.2. for detailed description of the two bent types.

Figure 13.3.14 Timber Pile Bent

Figure 13.3.15 Steel Pile Bent

Figure 13.3.16 Concrete Pile Bent

Important items to be noted by the inspector are collision damage, and material deficiencies. Scour of the river bottom material at the bottom of the piles can result in instability of the piles. The underwater inspector compares present scour and resultant pile length with that observed in previous inspections.

Piers

Piers consist of three basic elements, which are the pier cap, shaft, and footing. Piers carry superstructure loads from the pier cap to the footing, which may be a spread footing or may be supported on a deep foundation. Piers can be constructed of steel, timber, concrete, or masonry and are usually distinguished by two to four large columns or a single large shaft. As with pile bents, collision damage, material deterioration, and scour are important items to look for in an underwater inspection. It is also important for the inspector to note if the pier shaft or columns are vertical. There are four common types of piers the inspector is likely to encounter:

- Column pier
- Column pier with solid web wall (see Figure 13.3.17)
- Cantilever or hammerhead pier (see Figure 13.3.18)
- Solid shaft pier (see Figure 13.3.19)

Figure 13.3.17 Column Pier with Solid Web Wall

Figure 13.3.18 Cantilever or Hammerhead Pier

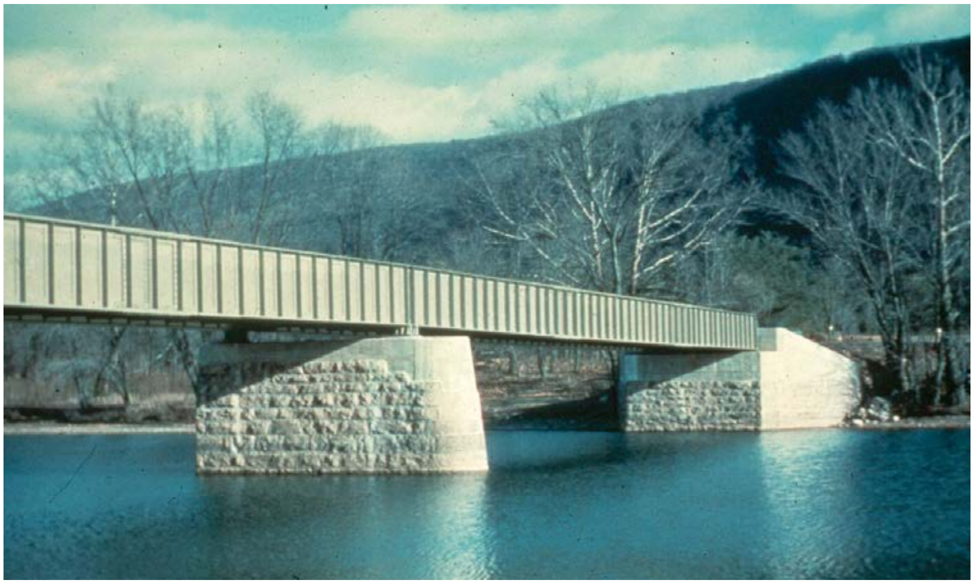

Figure 13.3.19 Solid Shaft Pier

Abutments

Abutments carry the superstructure loads to the underlying soil or rock and also retain the earth at the end of the structure. In most cases, the abutments are dry during low water periods and do not require a diving inspection. However, occasionally the abutments remain continually submerged and will require an underwater inspection (see Figure 13.3.19). Abutments can be constructed from concrete, masonry, or timber and may be supported by spread footings, piles, caissons, or pedestals. The most common abutment types include:

- Full height or closed
- Stub, semi-stub or shelf type
- Open or spill-through type
- Integral

Scour is probably the most critical item to be aware of when performing an underwater abutment inspection. Extreme local scour (undermining) could result in a forward tilting or rotation of the abutment, especially on those abutments without deep foundations (see Figure 13.3.20).

Figure 13.3.20 Severe Flood-Induced Abutment Scour

Caissons

Caissons, or drilled shafts, are enclosures which are used to build a substructure's foundation and carry loads from the bridge through the unsound soil and water to sound soil or rock. When it is in place, a caisson can act as a pier's footing. Caissons are made from timber, reinforced concrete, steel plates, or a combination of the above materials.

Cofferdams and Foundation Seals

Cofferdams and foundation seals are used to maintain a dry work area when constructing piers and abutments in water. Cofferdams are constructed from steel sheet piling. Once the foundation is complete, the sheeting may be removed or cut-off at the bottom of the channel.

Before a cofferdam is dewatered, a concrete seal needs to be placed below the water on top of the soil and to prevent any uplift and flooding of the dewatered cofferdam.

Protection Devices

Dolphins, fenders, and shear fences are often placed around substructure units to protect them from impact damage (see Figure 13.3.21). They are designed to absorb some of the energy from a direct hit from a vessel. Since these systems are usually at least partially underwater, conduct a diving inspection in concert with the substructure unit inspection.

Dolphins are a group of timber piles, but may also be a group of steel or composite piles. Fenders usually consist of timber or steel members attached directly to a substructure unit or piles adjacent to the substructure unit. Shear fences are generally an extension of a fender system which consists of a series of timber piles supporting timber wales and sheeting.

Figure 13.3.21 Damaged Protective System

Culverts

A culvert is a hydraulic structure normally constructed entirely below the ground and may be constructed of concrete, steel, timber, or stone masonry. Culverts that may not be inspected while dry will be inspected by diving. The underwater inspection of culvert structures present unique challenges to the inspection team, as culverts exist in a wide range of sizes, shapes, lengths, materials, and environments. Areas of special concern to the dive team when conducting culvert inspections include confined space, submerged drift and debris, and animal occupation.

Physically confined space issues arise when inspecting culverts containing individual pipes, barrels, or cells with small interior dimension, or non-linear layout. Additionally, many culverts are continually either completely submerged, or exhibit limited freeboard. In northern environments, winter inspections may also include ice as a contributing factor (see Figure 13.3.22). Conduct diving operations in physically confined space in compliance with Federal commercial diving regulations, as well as the individual agency's Safe Practices Manual. The Occupational Safety and Health Administration (OSHA) also offers guidance for work requiring confined space entry.

Submerged drift and debris is a persistent threat to the underwater inspection team, combining with the physically confining nature of most culvert structures to greatly increase the threat of diver entanglement. The diver may be completely unaware of the presence of drift until fouled. Use surface-supplied air diving equipment when conducting diving operations in physically confined and/or debris-laden culverts.

Another threat to the diver involves animals living or seeking shelter inside the culvert. Snakes are often found in and around accumulations of sediment and drift, while, in the southeast United States, alligators often reside inside culvert structures. When inspecting a structure exhibiting debris accumulations, which partially or fully constrict one end of a culvert, approach with caution, as excited animals may try to leave the culvert in haste, while the inspector is entering.

Figure 13.3.22 Inspection of Culvert With Limited Freeboard and Ice Cover

13.3.7 Underwater Inspection for Material Deficiencies

The materials typically used in bridge substructures are concrete, timber, steel, and masonry. An estimated 75% of all underwater elements are concrete. The balance consists of timber, steel, and masonry, in descending order of use.

Concrete

Plain, reinforced, and prestressed concrete are used in underwater elements. Since the majority of substructures are basically compression units, concrete is a nearly ideal material choice. Some concrete damage tends to be surface damage that does not jeopardize the integrity of the system. However, concrete deterioration that involves corrosion of the reinforcement may lead to a reduction in load carrying capacity (see Figure 13.3.23).

Cracking, delamination, spalling and chemical attack are typical for concrete substructures exposed to water. Reinforcement exposed to water and air is subjected to section loss. Scaling occurs above the water surface while abrasion occurs in the area near the water surface.

See Topic 6.2 for detailed descriptions of concrete deficiencies.

Figure 13.3.23 Concrete Deterioration

Masonry

Masonry can be used in substructure units, but is seldom used as a material in newer bridges. Masonry substructures can experience cracking and delamination of the stones. Cracking of mortar joints at the normal waterline is a result of freeze-thaw damage.

See Topic 6.5 for detailed descriptions of masonry deficiencies.

Timber

Timber pile bents are typical for short span bridges in many parts of the country, particularly for older bridges. The primary cause of timber deterioration is decay, abrasion, collision, and biological organisms, such as fungi, insects, bacteria, and marine borers. The ingredients for a biological attack include suitable food, water, air, and a favorable temperature. The waterline of pile structures offers all of these ingredients during at least part of the year. Since water, oxygen, and temperature generally cannot be controlled in a marine environment, the primary means to prevent a biological attack is to deny the food source through treatment to poison the wood as a food source. Timber piles are particularly vulnerable if the treatment leaches out (which happens with age) or if the core is penetrated. Therefore, it is important to carefully inspect in the vicinity of connectors, holes, or other surface blemishes (see Figure 13.3.24).

Figure 13.3.24 Deteriorated Timber Piling

Piles used in older bridges quite often were not treated if the piles were to be buried below the mud line (eliminating the source of food and oxygen). However, in some cases, streambed scour may have exposed these piles. Take special care in differentiating between treated and untreated piles to ensure a thorough inspection of any exposed, untreated piles. With each inspection, note the diameter or circumference for each timber pile. As a minimum, make these measurements at the waterline and mud line. Make comparisons with the original pile size.

Another primary caution for inspecting underwater timber piles is that the damage is frequently internal. Whether from fungal decay or borers, timber piles may appear sound on the outside shell but be completely hollow inside. While some sources recommend hammer soundings to detect internal damage, this method is unreliable in the underwater environment. One way to inspect for such damage is to take core samples. Plug all bore holes. Ultrasonic techniques for timber piling are also available.

See Topic 6.1 for detailed descriptions of timber material deficiencies.

Steel

Underwater steel structures are highly sensitive to corrosion, particularly in the low to high water zone (see Figure 13.3.25). Whenever possible, measure steel to determine if section loss has occurred. Ultrasonic devices are particularly useful to determine remaining steel thicknesses.

Figure 13.3.25 Deteriorated Steel Piles at Splash Zone

Connections such as bolts, rivets and welds are examined for corrosion. If the steel members have a coating, check the condition of the coating and its ability to protect the steel. In addition to protecting a concrete deck, cathodic protection has been used to protect steel piles in harbor settings. These cathodic protection systems may become more popular in the future. Check to see if the system appears to be working and check the connections and power source.

See Topic 6.3 for detailed descriptions of steel material deficiencies.

Composite Materials

One composite material, known as fiber reinforced polymer (FRP) is a mixture of fibers and resin. FRP is becoming more popular throughout the transportation community and can be used for substructure units. This material is more resistant to marine borers than timber members.

Composite materials have mechanical deficiencies similar to traditional materials, which are due to impact, abrasion or construction related events. Environmental deficiencies in composite materials include fires and ultra-violet ray degradation.

See Topic 6.6 for detailed descriptions of fiber reinforced polymer material deficiencies.

Vessel Damage

Bridges that are located in water are susceptible to damage from any vessel on the water. Damage that happens from a vessel collision may be visible on top of the water, but the extent of the underwater damage cannot be properly assessed without a detailed underwater inspection.

Damage below normal water level caused by prop wash may not be visible above the water. Examples of vessels that rotate their propellers at high speeds and may cause prop wash are ferry's leaving terminals or tugboats moving barges from their moorings. The movement may pick up bottom material and discharge it against the foundations, essentially sandblasting the material which, in time, can cause the erosion of steel and concrete surfaces.

Hands-on Inspection of Material Underwater

When visibility permits, the diver visually observes all exposed surfaces of the substructure. Scraping over the surface with a sharp-tipped probe, such as a knife or ice pick, is particularly useful for detecting small cracks. With limited visibility, the diver "feels" for damage. Because orientation and location are often difficult to maintain, the diver will be systematic in the inspection. Establish regular patterns from well-defined reference points.

Typical inspection patterns include:

- Circular or semicircular horizontal sweeps around piers or abutments beginning at the base, moving upward a specified increment, and repeating until complete
- Probing zones of undermining of piers by moving uniform increments from start to finish and recording the undermined penetration
- Down one side and up the other for piles (or inspecting in a spiral pattern)
- For scour surveys, record depths at regular increments adjacent to substructure (e.g., at each pile or 10 foot increments around piers), and then at each measured point extend radially from the substructure a uniform distance and repeat depth measurements

Major advantages of surface-to-diver communications are that the diver can be guided from the surface with available drawings, and that immediate recording of observations can be made topside along with the clarification of any discrepancies with plans.

Measuring Damage

Measure any damage encountered in detail. As a minimum for a Level II or III inspection include:

- Location of the damage zone both horizontally and vertically from a fixed reference point
- A good vertical reference point is the waterline, provided that the waterline is measured with respect to a fixed reference point on the bridge prior to the dive
- For undermining of foundations, take enough measurements to define the zone no longer providing soil bearing
- If plans are not available, measure the basic dimensions of damaged members (it is also usually prudent to spot check dimensions of damaged members even if plans are available)
- Check for displacements of major elements and whether they are plumb
- Locate the beginning and ends of cracks and intermediate points as needed to define the pattern
- Measure the maximum crack width and penetration depth
- Measure the length, width, and penetration of spalls or voids, making note of exposure and condition of any reinforcing steel
- Note the degree of scaling on concrete
- Measure the thicknesses of all four flange tips on steel H-piles at distressed areas, and specify the vertical location
- Locate buckles, bulges, and significant loss of section in steel members -

accurately measure the thickness of remaining sound material when significant section loss is found

- Note damage at connections
- Measure the diameter of timber piles – note extent and width of checks, and extent of any decay, if found.

Recordkeeping and Documentation

Because of the effort spent in conducting underwater inspections, combined with the time between inspections, it is particularly important to carefully document the findings. On-site recording of all conditions is essential:

- It is recommended that sketches be used as much as possible; providing enough detail is critical since it is difficult to go back to check items once the diving is completed. Contour and plan view sketches of the area surrounding the substructure elements allow the inspector to track any scour or streambed movement. A profile of the streambed can also provide information for tracking the development of scour.
- In addition to sketches, keep written notes or logs, documenting the inspection.
- When significant damage is encountered, a tape recording of the diver's observations can also prove helpful.
- Underwater photographs and/or underwater videotapes can be used to support the inspection report.
- If repairs were recommended in previous inspection reports, verify the repairs were made and that they have addressed the deficiencies

Include the results in an inspection form or report. Drawings and text need to describe all aspects of the inspection and any damage found. Include recommendations on condition assessment, repairs, and time interval for the next inspection in the report. See Figure 13.3.26 for a sample underwater inspection form.

See Topic 4.4 for detailed descriptions of record keeping and documentation.

CONDENSED UNDERWATER BRIDGE INSPECTION REPORT

BRIDGE NUMBER	COUNTY NAME	ROAD NUMBER	ROAD NAME	DATE INSPECTED

BODY OF WATER: ____________________ **DIVE MODE:**

DIVING CONDITIONS

MAXIMUM CURRENT: AIR TEMPERATURE:

AVERAGE VISIBILITY: WATER TEMPERATURE:

BOTTOM MATERIAL: MAXIMUM DEPTH: ____________________

ITEMS INSPECTED: ____________________

ITEM OF INSPECTION	NCR**	REMARKS
1. PILING/SHAFTS		
2. FOOTINGS/CAISSONS/PEDESTALS		
3. COLUMNS/WALL PIERS		
4. BRACING/STRUTS/WEB WALLS		
5. ABUTMENTS/END BENTS		
6. RETAINING WALLS/WING WALLS		
7. FENDER SYSTEM/PIER PROTECTION		
8. EMBANKMENTS/SLOPES/BULKHEADS		
9. DEGRADATION/AGGRADATION		
10. OBSTRUCTION/FLOW		
11. MOVABLE BRIDGE PIERS (PIVOT, BASCULE, REST)		
12. CULVERT BARRELS		
13. CULVERT HEADWALLS		
14. SUBMARINE CABLE (S) ***		

* Deficiencies exist in this element that warrant written and/or sketched description which are provided in the "Comprehensive Report of Deficiencies" section of this report.

** NCR is an acronym for numerical condition rating, the definitions of which can be found on the back of this page.

*** Submarine Cables(s) rated using Non-Structural Features rating system [1 (Poor) to 4 (Good) or N]

INSPECTION PARTY

Name: Name:
Name: Name:

Figure 13.3.26 Sample Underwater Inspection Form

NUMERICAL CONDITION RATING DEFINITIONS FOR STRUCTURAL ITEMS

CODE	DESCRIPTION
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION-No problems noted.
7	GOOD CONDITION-Some minor problems. Minor maintenance may be needed.
6	SATISFACTORY CONDITION-Structural elements show some minor deterioration. Major maintenance is needed.
5	FAIR CONDITION-All primary structural elements are sound but may have minor section loss, cracking, spalling. Minor rehabilitation may be needed.
4	POOR CONDITION-Advanced section loss, deterioration, spalling. Major rehabilitation may be needed.
3	SERIOUS CONDITION-Loss of section, deterioration, spalling have seriously affected primary structural elements. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present. Repair or rehabilitation required immediately.
2	CRITICAL CONDITION-Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	AIMMINENT@ FAILURE CONDITION-Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION-Out of Service-beyond corrective action

NUMERICAL CONDITION RATING DEFINITIONS FOR DEGRADATION/AGGRADATION

CODE	DESCRIPTION
N	NOT APPLICABLE-Use when bridge is not over a waterway.
9	EXCELLENT CONDITION-No noticeable or noteworthy deficiencies, which affect the condition of the channel.
8	VERY GOOD CONDITION-Banks are protected or well vegetated. River control devices, such as spur dikes and embankment protection, are not required or are in stable condition. Some minor scour has occurred near bridge.
7	GOOD CONDITION-Bank protection is in need of minor repairs. River control devices and embankment protection have minor damage. There is minor streambed movement evident. Minor local scour developing near substructure.
6	SATISFACTORY CONDITION-Bank is beginning to slump. River control devices and embankment protection have considerable minor damage. There is minor streambed movement evident. Debris is restricting the waterway slightly. Scour holes deepening.
5	FAIR CONDITION-Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel. Scour holes are becoming more prominent, affecting the stability of the substructure.
4	POOR CONDITION-Bank and embankment protection undermined with corrective action required. River control devices have severe damage. Large deposits of debris in the waterway. The streambed has changed its location but is causing no problem.
3	SERIOUS CONDITION-Bank protection has failed completely. Scour holes forming in embankment. River control devices have been destroyed. Streambed aggradation or degradation has changed the waterway to now threaten the bridge and/or approach roadway.
2	CRITICAL CONDITION-Abutment has failed (portion has settled) due to undermining of footing. The waterway has changed and now threatens the bridge and/or embankment. Scour is of sufficient depth beneath footing that substructure is in near state of collapse.
1	AIMMINENT@ FAILURE CONDITION-Bridge closed. Corrective action may put the structure back into light service.
0	FAILED CONDITION-Bridge closed. Replacement necessary.

Figure 13.3.26 Sample Underwater Inspection Form (Continued)

13.3.8 Special Considerations for Underwater Inspections

Once a diver enters the water, their environment changes completely. Visibility decreases and is often reduced to near zero, due to muddy water and depth. In many cases, artificial lighting is required. There are times when tactile (by feel) inspections are all that can be accomplished, significantly compromising the condition evaluation of the element(s) being inspected.

The diver not only has reduced perceptual capabilities but is less mobile as well. Maneuverability is essential for underwater bridge inspections. With either self-contained or surface-supplied equipment, the diver may find it useful to adjust his/her underwater weight to near buoyancy and use swim fins for propulsion.

It is important for the diver to be able to adapt to the environment and be familiar with the diving equipment. They are to feel safe and comfortable while working underwater to be able to do an effective job on the inspection and to remain safe while performing the inspection.

Dealing with Current

Most waterways have low flow periods when current will not hinder an inspection. Plan diving inspections with this consideration in mind. Divers can work in current below 1.0 knots with relatively little hindrance. Currents may vary in direction or velocity when inspecting around submerged obstacles such as cofferdams (see Figure 13.3.27).

Figure 13.3.27 Diving Inside a Cofferdam

Waterway conditions may sometimes be too swift to allow safe diving operations (see Figure 13.3.28). For these conditions, other appropriate procedures must be used to evaluate the condition of underwater elements.

Figure 13.3.28 Excessive Current

Dealing with Drift and Debris

The drift and debris that often collects at bridge substructures can be extensive (see Figure 13.3.29). This type of buildup typically consists of logs and limbs from trees that are usually matted or woven either against or within the substructure elements. Often this debris is located on the lower parts of the substructure and cannot be detected from the surface. The buildup can be so thick as to prevent access to major portions of the underwater substructure.

Address concerns such as removal, past history, and safety when dealing with the presence of drift and debris.

Figure 13.3.29 Debris

Since drift and debris are often under the water surface, it is difficult to estimate the time and cost required to remove and gain access. The removal of the drift and debris is required if a hands-on inspection of the underwater elements is to proceed. While in some cases debris can be removed by the inspection divers, heavy equipment, such as a hoist or underwater cutting devices, are often required.

Generally, such buildup occurs in repetitive patterns. If previous underwater inspections have been conducted, the presence of drift can be estimated based on past history. Also, certain rivers and regions tend to have a history of drift problems, while others do not. Knowledge of this record can help predict the likelihood of drift and debris accumulation. A separate drift removal team, working ahead of the dive inspection team, could possibly be utilized.

Debris build-up near a bridge creates unique safety concerns for the dive team. Occasionally, debris can be quite extensive and can lead to entanglements or sudden shifts which might entrap the diver. Divers normally approach debris from the downstream side to avoid entanglements (see Figure 13.3.29).

Cleaning

Bridges on many inland waterways are relatively clean and free of marine growth. In such cases, the inspection can be conducted with little extra effort from the diver other than perhaps light scraping.

In coastal waterways, the marine growth can completely obscure the substructure element and may reach several inches or more in thickness (see Figure 13.3.30). The cost of cleaning heavily infested substructures may be completely impractical. In such cases, spot cleaning and inspection may be the only practical alternative.

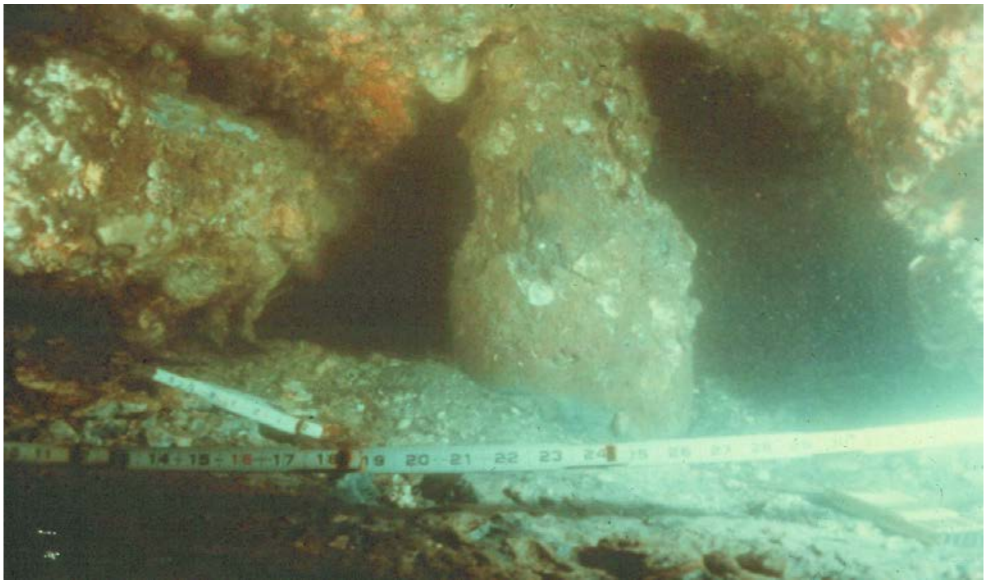

Figure 13.3.30 Cleaning a Timber Pile

Physical Limitations

This sometimes cold, dark, hostile underwater environment can result in a reduced physical working capacity. The diver is also totally dependent on external life support systems, which adds psychological stress. Things that can be done intuitively above water include a conscientiously planned effort and executed step-by-step procedures for underwater. For example, maintaining orientation and location during an underwater inspection requires continual attention. Typical distractions include living organisms, such as fish, snakes, and crustaceans and also environmental conditions, such as low temperatures, high current and heavy debris.

Decompression Sickness

Since the majority of bridge inspections are in relatively shallow water and of relatively short duration, decompression problems rarely occur. However, multiple dives have a cumulative effect and the no-decompression time limit decreases rapidly at depths greater than 50 feet. Therefore, divers routinely track their time and depth as a safety precaution. OSHA requires that a decompression chamber be on-site and ready for use for any dive made outside the no-decompression limits or deeper than 100 feet of seawater.

Marine Traffic

Another concern for divers is vessel traffic near the area to be inspected. Someone will always be topside with the responsibility of watching boat traffic (see Figure 13.3.31). In addition, display flags indicating that a diver is down. The international code flag "A", or “Alpha” flag (white and blue), signifies that a diver is down and to stay clear of the area. OSHA requires this flag. However, it is also prudent to display the sport diver flag (white stripe on red), since it is more likely that recreational boaters will recognize this flag (see Figure 13.3.32).

Figure 13.3.31 Commercial Marine Traffic

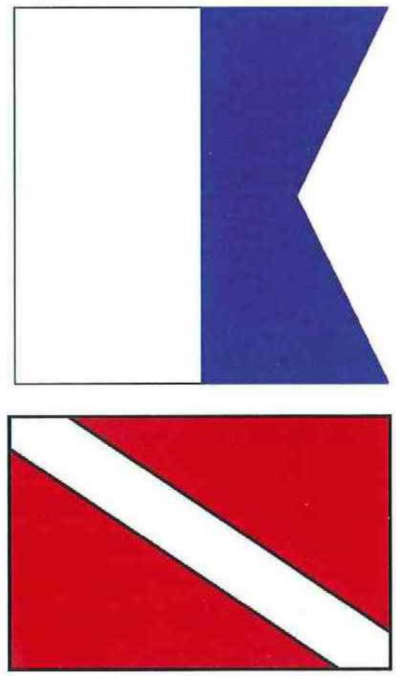

Figure 13.3.32 Alpha (top) and Sport Diver (bottom) Flags

13.3.9 Types of Underwater Inspection

Diving inspectors are responsible for identifying the location of underwater elements including a description of the underwater elements. They also verify if the inspection frequency and procedures are adequate in accordance with the inspection record. Inspectors can make recommendations to improve the underwater inspection procedures listed in the inspection record if conditions have changed.

According to the AASHTO *Manual for Bridge Evaluation*, underwater inspections can include wading or diving inspections. The National Bridge Inspection Standards (NBIS), however, define an underwater inspection as the inspection of the underwater portion of a bridge substructure and the surrounding channel that cannot be visually inspected at low water by wading or probing, which will require diving or appropriate techniques. For this topic three general methods used to perform underwater inspections are presented:

- Wading inspection
- Commercial SCUBA
- Surface-supplied diving

Wading Inspection

Wading inspection is the basic method of underwater inspection used on structures over wadeable streams. The substructure units and the waterway are evaluated using a probing rod, sounding rod or line, waders, and possibly a boat. Regular bridge inspection teams can often perform wading inspections during periods of low water (see Figure 13.3.33).

Figure 13.3.33 Inspector Performing a Wading Inspection

Commercial SCUBA

SCUBA, an acronym for Self-Contained Underwater Breathing Apparatus, is used for many underwater inspections in this country (see Figure 13.3.34). In this mode, the diver operates independently from the surface personnel, carrying their own supply of compressed breathing gas (typically air) with the diver inhaling the air from the supplied tank and the exhaust being vented directly to the surrounding water. SCUBA diving is employed during underwater bridge inspections due to its ease of portability and maneuverability in the water. It is used where the dives have a short duration at different locations rather than a long sustained dive. This dive mode is best used at sites where environmental and waterway conditions are favorable, and where the duration of the dive is relatively short. Exercise extreme care when using SCUBA equipment at bridge sites where the waterway exhibits low visibility and/or high current, and where drift and debris may be present at any height in the water column. The use of SCUBA gear is limited to water depths of 130 feet and the time on the bottom will be limited by the amount of air the diver can carry and the amount of time based upon the no-decompression limits.

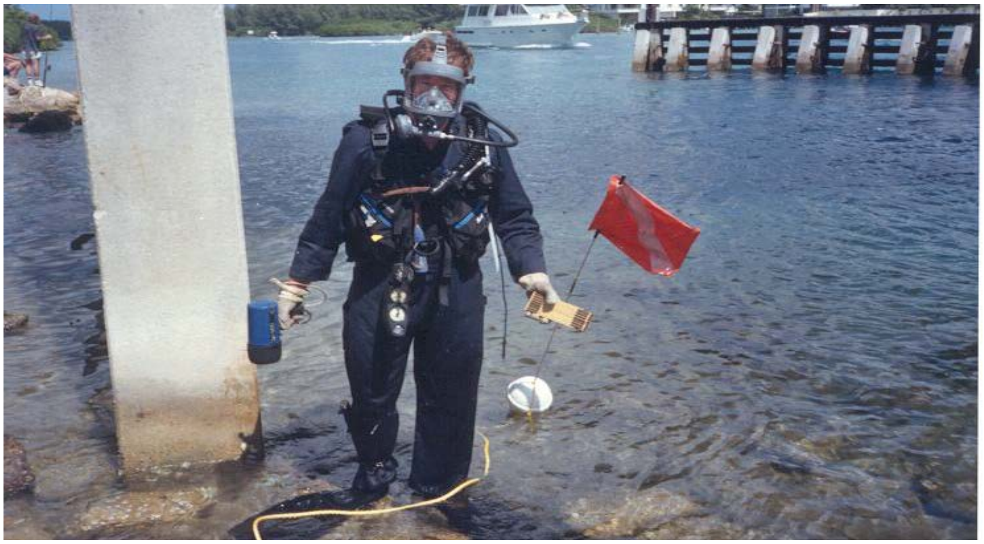

Figure 13.3.34 SCUBA Inspection Diver

Surface-Supplied Diving

As its name implies, surface-supplied diving uses a breathing gas supply that originates above the water surface and is commonly referred to as lightweight diving equipment. This breathing gas (again, typically compressed air) is transported underwater to the diver via a flexible umbilical hose. Surface-supplied equipment provides the diver with a nearly unlimited supply of breathing gas, and also provides a safety tether line and hard-wire communications system connecting the diver and above water personnel. Using surface-supplied equipment, work may be safely completed under adverse conditions that often accompany underwater bridge inspections, such as: fast current, cold and/or contaminated water, physically confined space, submerged drift and debris, and dives requiring heavy physical exertion or of relatively long duration (see Figure 13.3.35). Depths of surface-supplied dives can be conducted down to 190 feet or if the bottom times are less than 30 minutes, to a depth of 220 feet. This form of diving provides advantages such as an "unlimited" air supply and communications plus bottom times that can exceed the decompression time limits used for SCUBA. The disadvantages of this form of diving is that it requires more topside support than SCUBA and it is limited in mobility due to the connection to the surface.

Figure 13.3.35 Surface-Supplied Diving Inspection

Inspection Type Selection Criteria

In determining whether a bridge can be inspected by wading or whether it requires the use of diving equipment, water depth is not be the sole criteria. Many factors combine to influence the proper underwater inspection type:

- Water depth
- Water visibility
- Current velocity
- Streambed conditions (softness, mud, "quick" conditions, and slippery rocks)
- Debris
- Substructure configuration

13.3.10 Underwater Inspection Equipment

Diving Equipment

Essential personal diving equipment includes:

- Wet suit or dry suit (also known as a exposure suits) (see Figure 13.3.36)
  - Wet suits allow a thin layer of water between the diver's skin and the suit. The water layer is warmed by the diver's body heat and acts as insulation to keep the diver warm.
  - Dry suits utilize air instead of water to insulate the body and are very effective in cold or polluted water.
- Face mask or helmet (see Figure 13.3.37)
- Buoyancy compensator (a flotation device capable of maintaining a diver face up at the surface)
- Breathing apparatus and/or reserve breathing air supply
- Weight belt
- Swim fins
- Knife
- Wristwatch
- Depth gauge
- Submersible pressure gauge
- Flashlight or dive light

Surface-supplied air diving equipment typically includes a compressor, which supplies air into a volume tank for storage. This compressed air is then filtered and regulated to the diver’s helmet or mask through an umbilical hose (see Figures 13.3.38 and 13.3.39). The umbilical is typically made up of several members, including, at a minimum, a breathing air hose, strength member (or safety line), communication line, and pneumofathometer hose. The pneumofathometer provides diver depth measurements to the surface (see Figure 13.3.40).

For self-contained diving, the breathing gas supply is contained within a pressurized tank, which is carried by the diver.

Figure 13.3.36 Vulcanized Rubber Dry Suit

Figure 13.3.37 Full Face Lightweight Diving Mask with Communication System

Figure 13.3.38 Surface-Supplied Air Equipment, Including Air Compressor, Volume Tank With Air Filters, and Umbilical Hoses

Figure 13.3.39 Surface-Supplied Diving Equipment Including Helmet or Hard Hat

Figure 13.3.40 Pneumofathometer Gauge

Equipment malfunction leading to loss of air supply needs to be a constant concern to the dive team. Even in shallow water, submerged drift and debris adjacent to a bridge can make an emergency ascent an arduous affair, for both the diver and the support team. As such, a reserve air supply will always be worn by the diver using surface supplied air (see Figure 13.3.39). Carbon monoxide poisoning can occur if the air intake of the surface supplied air compressor is located near the exhaust of other motorized equipment (see Figure 13.3.38).

Figure 13.3.41 Surface-Supplied Diver with a Reserve Air Tank

Surface Communications

While not required in all situations, a two-way communication system linking the diver(s) and topside personnel greatly enhances the underwater inspection. There are two types of diver-to-surface communications: a conventional hardwire and a wireless system. In the hardwire system, the diver has a microphone and speaker connected to a surface transmitter-receiver through a cable. This is regularly used in surface-supplied diving. It can also be used when a SCUBA diver is using a full face mask with the mask tended to the surface with a strength or communication line. The wireless systems are available for use in SCUBA diving equipment. The advantage of a wireless system is that it allows the diver to have more mobility (see Figure 13.3.42), and can be used during self-contained diving operations.

There are several advantages provided to the underwater inspection team, through the use of direct two-way communication

- Dive team safety is increased in the event of diver entanglement or equipment malfunction
- Divers can immediately describe observations and location of deficiencies for simultaneous recording by a note taker on the surface
- Divers can verbally interact with topside inspection personnel to clarify what is being observed, without leaving the suspect area
- Note takers can follow drawings, verify their validity, note damage on the drawings at the proper location, and track the progress of the diver
- Surface communication also allows an inspection team leader/engineer at the surface to discuss observations with a diver who is not yet an inspection team leader, to direct attention to specific zones, and to ensure that a satisfactory inspection is completed, according to the type and severity of damage found (see Figure 13.3.43)

Figure 13.3.42 Wireless Communication Box System

Figure 13.3.43 Surface Communication With Inspection Team Leader

Access Equipment

While inspection of short-span bridges can often be accessed from shore, many bridges require a boat or barge for access. Boats may be in different sizes and types, but large enough so it can safely handle the diving equipment and personnel as well as a suitable size for the waterway conditions (see Figures 13.3.44 and 13.3.45). The boat needs to be equipped with an engine which will be dictated by the waterway conditions and the boat size.

Figure 13.3.44 Access Barge and Exit Ladder

Figure 13.3.45 Access From Dive Boat

Tools

A number of inspection tools are available. The dive team needs to have access to the appropriate tools and equipment (including both hand and power tools) as warranted by the type of inspection being conducted.

Hand Tools

While most hand tools can be used underwater, the most useful include rulers, calipers, scrapers, probes (ice picks, dive knifes, and screwdrivers), flashlight, hammers (especially masonry and geologist's hammers), axes, hand drills, wire brushes, incremental borers, hand saws, and pry bars (see Figure 13.3.46). These tools are usually tethered to the diver to prevent their loss underwater. Working with hand tools could be slow and may be impractical for larger jobs.

Figure 13.3.46 Diver with a Pry Bar and Diver with Hand Scraper

Power Tools

Power tools include both pneumatic and hydraulic tools. Pneumatic tools are not usually designed for underwater use, but can be adapted to perform the necessary tasks. Examples of pneumatic tools that can be used include pneumatic drills, chippers, hammers, scalers, and saws. Pneumatic tools are also limited to practical depths of 100 to 150 feet and can obscure the diver's vision by the bubbles produced by the tools.

Hydraulic tools are modified versions of tools used on dry land. Examples include grinders, chippers, drills, hammers and saws. The advantage of using hydraulic tools is that they do not create the bubbles that a pneumatic tool creates.

While pneumatic tools are sometimes used, hydraulic tools tend to be favored for heavy or extensive work often required during underwater inspections.

Cleaning Tools

Light cleaning can be accomplished with scrapers and wire brushes. Heavier cleaning requires automated equipment such as grinders and chippers. One of the most effective means of cleaning is with the use of water blasters (see Figure 13.3.47). Take particular care with such equipment to ensure that structural damage does not result from overzealous blasting.

Figure 13.3.47 Cleaning with a Water Blaster

Advanced Inspection Methods

When inspecting underwater elements, nondestructive evaluations (NDE) may be required to determine the structural condition and may be used in Level III inspection.

Steel

For steel substructures, the inspector is often concerned with measuring the remaining thickness of any corroded members. Nondestructive evaluations for underwater steel members include:

- Ultrasonic measuring devices measure the thickness of steel by passing a sound wave through the member. The transducer is placed on one side only, and the thickness is displayed on an LED readout. Totally submersible or surface display units are available. They are very effective for measuring thickness. There are two types of ultrasonic devices to be used underwater. One utilizes a waterproof transducer and cable that is carried below the water surface while the electronics and display remain on the surface. The second type can be placed in a waterproof container and taken underwater with the diver.
- Underwater magnetic particle testing equipment, typically consisting of an electromagnetic yoke and powdered metallic particles, are used to detect flaws at or near the surface of ferrous metal members and welds. The articulating yoke is positioned on the member in question, and energized. A liquid suspension containing a fluorescent dye and magnetic particles is applied to the area between the legs of the yoke. Discontinuities in the specimen, such as cracks, will cause a magnetic flux leakage field, which will attract the particles. The inspector photographs the particle pattern to document the test results. This is commonly used during inspections on offshore structures and not commonly used on bridges due to lack of underwater weld. It is also difficult to implement due to the high currents and the poor clarity of inland water.

Concrete

For concrete substructures, there are several nondestructive tests for in-depth inspections that can be performed.. Nondestructive evaluations for underwater concrete members include:

- An ultrasonic pulse velocity meter (or V-meter) is an ultrasonic device that requires two transducers and measures the distance required for the sound wave to pass through the concrete. This device is used to estimate the strength of concrete. It is also used to locate the discontinuity and low strength areas such as cracks and voids. Direct transmission methods require the transducers to be on opposite sides of the member and will provide the most accurate data. Indirect transmission methods place the transducers on the same side of the member and will require correction factors to properly interpret the data. Similar devices have also been developed for timber.
- A waterproof rebound hammer (also known as a Schmidt hammer) can be used underwater to estimate the compressive strength of in-place concrete based on its surface hardness. To use the hammer, the diver places and then presses it to the concrete surface until a mass in the hammer is released causing impact. The inspector estimates the concrete's strength with the use of the data.
- A rebar locator (or R-meter) is used to locate and measure the depth of cover and the size of reinforcing bars in concrete by inducing a magnetic field. This device will use a low frequency magnetic field to locate the steel.

Coring is a partially destructive evaluation method whose use is limited to critical areas. Cores can be taken in either concrete or timber (see Figure 13.3.48).

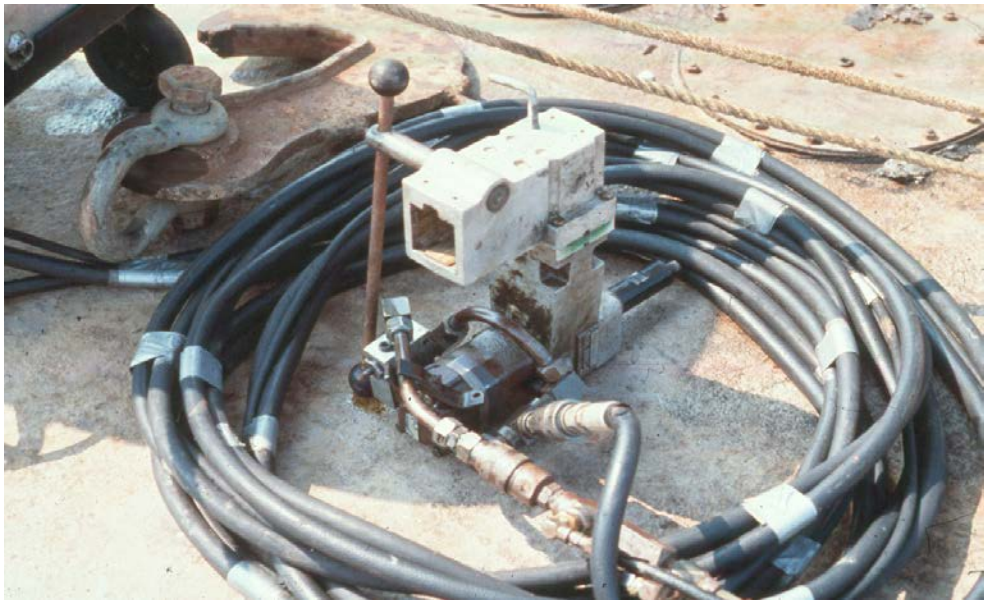

Figure 13.3.48 Coring Equipment

Concrete coring requires pneumatic or hydraulic equipment. Deep cores (3 feet or more) can be taken to provide an interior assessment of massive substructures (see Figure 13.3.49). Two-inch diameter cores are common, but coring tools are available in other sizes (see Figure 13.3.50). Cores not only provide knowledge about interior concrete consistency but also can be tested to determine compression strength. Be sure to select coring locations so the reinforcement is not damaged, unless a sampling of the reinforcement is desired. Patch the core holes upon completion.

Figure 13.3.49 Concrete Coring Taking Place

Figure 13.3.50 Concrete Core

Timber

Ultrasonic devices (V-meters) such as those used for concrete evaluations can be used to test timber members for internal voids or material breakdown caused by marine borers or decay.

Timber coring is much simpler and less costly to perform than concrete coring (see Figure 13.3.51). While power tools are sometimes used, the most effective procedure is still to hand core with an increment borer. This approach preserves the core for laboratory as well as field evaluation. The core indicates evidence of borers or other infestation, and of void areas. Always plug the hole with a treated hardwood dowel to prevent infestation.

Figure 13.3.51 Timber Core

Underwater Imaging

Photography

Color digital cameras come with a variety of lens and flash units. Popular cameras that can be used above water can be used underwater by placing them in a clear waterproof case, also known as a "housing". The boxes are constructed of clear plastic and can be used underwater (see Figure 13.3.52 and 13.3.53). There are also waterproof digital cameras that are designed specifically for underwater photography.

Figure 13.3.52 Various Waterproof Camera Housings

Figure 13.3.53 Diver Using a Camera in a Waterproof Housing

In some cases, visibility is limited and the camera needs to be placed close to the subject. Suspended particles often dilute the light reaching the subject and can reflect light back into the lens. When visibility is very low and the water is extremely turbid, clearwater boxes can be used (see Figure 13.3.54). A clearwater box is a clear plastic box that can be filled with clean water. The box can be placed up against the subject, which will displace the dirty water and allowing the camera to focus on the member being photographed.

Figure 13.3.54 Diver Using a Clearwater Box

Video

Video equipment is available either as self-contained, submersible units or as submersible cameras (or surface video cameras in a waterproof housing) having cable connection to the surface to view on the monitor or to record (see Figure 13.3.55). The latter type allows a surface operator to direct shooting while the diver concentrates on aligning the camera only. The operator can view the monitor, control the lighting and focusing, and communicate with the diver to obtain an optimum image. Since a sound track is linked to the communication equipment, a running commentary can also be obtained.

Smaller video cameras are in plastic cases and can be used with or without the umbilical to the surface where they are monitored. Video cameras may also be attached to a staff or a truck mounted arm so it can be deployed from a bridge deck and can relay images to the monitors and recording devices.

Figure 13.3.55 Underwater Video Inspection

Remotely Operated Vehicle (ROV)

An extension of the video camera is a remotely operated vehicle (ROV), where the diver is eliminated and the camera is mounted on a surface controlled propulsion system (see Figure 13.3.56). Its effectiveness diminishes substantially in stream velocities greater than 1.5 knots and is limited by cloudy water, inability to determine the exact orientation and position of the camera, and difficulties the operator may have controlling the vehicle due to the current or the umbilical being tangled. The ROV cannot perform cleaning operations prior to photos being taken.

Figure 13.3.56 Remotely Operated Vehicle (ROV)

Underwater Acoustic Imaging

Underwater acoustic imaging can provide greatly improved images of the channel bottom conditions, undermining and submerged foundations (see Figure 13.3.57). This can aid in the planning of diving operations by detecting areas of possible damage and will allow the divers to concentrate in these areas. It can also enhance diver safety by identifying potential dive hazards before anyone would enter the water. Acoustic imaging can also provide images of an underwater element that a underwater camera may not be able to take due to the turbidity of the water. Imaging also can operate at distances of 200 feet, while cameras, even in fairly clear water, has an effective range of only a few feet.

This is also useful when an emergency evaluation of a bridge may be necessary after a bridge is damaged by a collision, especially if the water conditions (e.g., high current, low visibility, debris) preclude the use of divers.

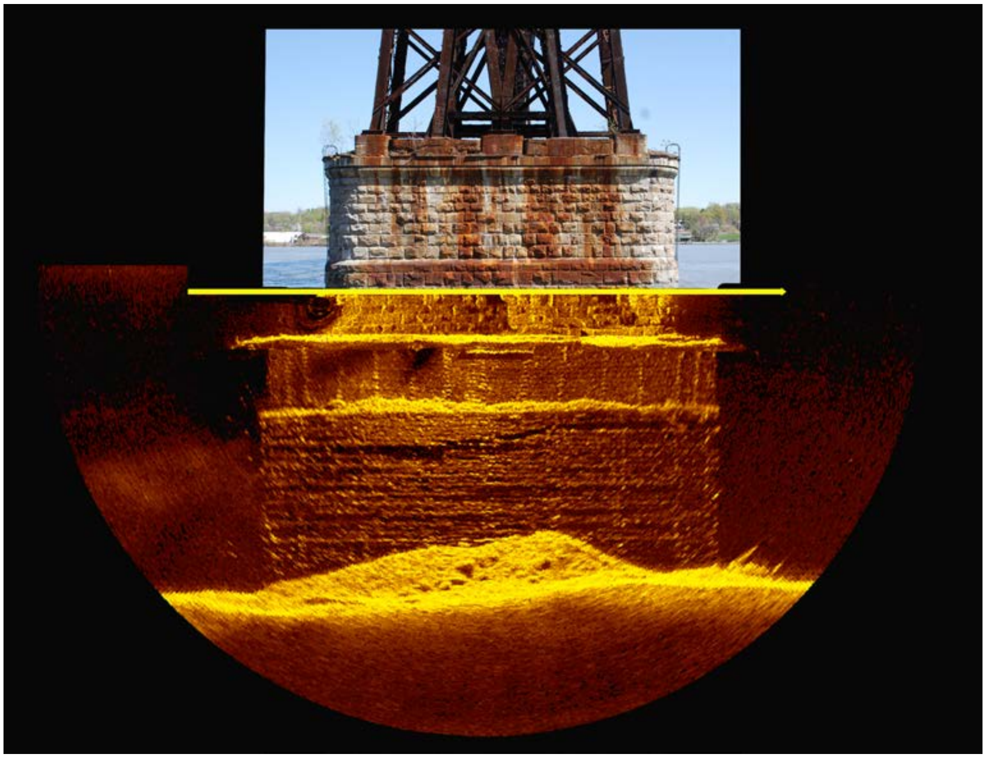

Figure 13.3.57 Acoustic Imaging of a Pier

13.3.11 Scour Inspections

Divers may be able to note scour under certain conditions. The most important assessment is how much of the bent or pier is exposed when compared to plans and typical designs.

Local scour is often detectable by divers since this type of scour is characterized by holes near bents, piers, or abutments. Divers routinely check for such scour holes. A typical approach is to take depth measurements around the substructure, both directly adjacent and at concentric intervals. Note that divers typically operate in low current situations. Sediment often refills scour holes during these periods, making detection of even local scour difficult. However, since this refilled sediment is usually soft, a diver using a probing rod can often detect the soft areas indicating scour refilling.

The diver's role is primarily to point out a potential scour problem. Almost invariably, an additional interdisciplinary engineering investigation will be needed. The diver's primary role in scour investigation is to measure scour by one of these methods:

- Sounding devices
- Geophysical inspections
- Diver inspections

Sounding Devices

Although sounding-sensing devices can be used independently of diving, they are commonly part of an underwater inspection. See Advanced Inspection Methods in Topic 13.2.6 on the procedures to record the stream cross section and profile. An on-site diver can investigate questionable readings and more fully determine the channel bottom conditions.

Fathometer

A fathometer consists of a transducer that is suspended in the water, a sending/receiving device, and a recording device that will display the depth on paper or a display. It can be either in color or black and white. A transducer floats just below the waterline and bounces sound waves off the bottom. Depths are continuously recorded on a strip chart.

Advantages of fathometer include the following:

- Inexpensive
- Effective
- "User-Friendly" output

Disadvantages include the following:

- False readings can occasionally occur due to heavy drift or heavy turbulence
- Fathometers may fail to detect refilled scour holes during calm water
- Fathometers do not provide what type of material makes up the channel bottom
- The strip chart moves at a constant rate and does not record a horizontal

scale; unless the boat can be kept at a constant speed, the scale becomes distorted; GPS has been added in recent years to provide a more exact location where readings are taken

Geophysical Inspection

Scour most commonly occurs during a flood. After a flood, the sediment settles, possibly refilling any scour hole that the flood may have caused. Geophysical tools can be used to measure scour after a scour hole has been refilled.

Ground-Penetrating Radar

Ground-penetrating radar (GPR) equipment are also used in scour surveys (see Figure 13.3.58). They can be used to obtain high resolution, continuous subsurface profiles on land or in shallow water which is less than 25 feet deep. GPR transmits short electromagnetic pulses into the subsurface and will measure the travel time to and from the subsurface for the signal to return. Once the signal encounters an interface between two different materials, a portion of the energy will be sent back to the surface and the rest will be sent into deeper layers. These are not as effective when encountering material that is highly conductive (e.g., clay), in salt water, or water with heavy amounts of sediment.

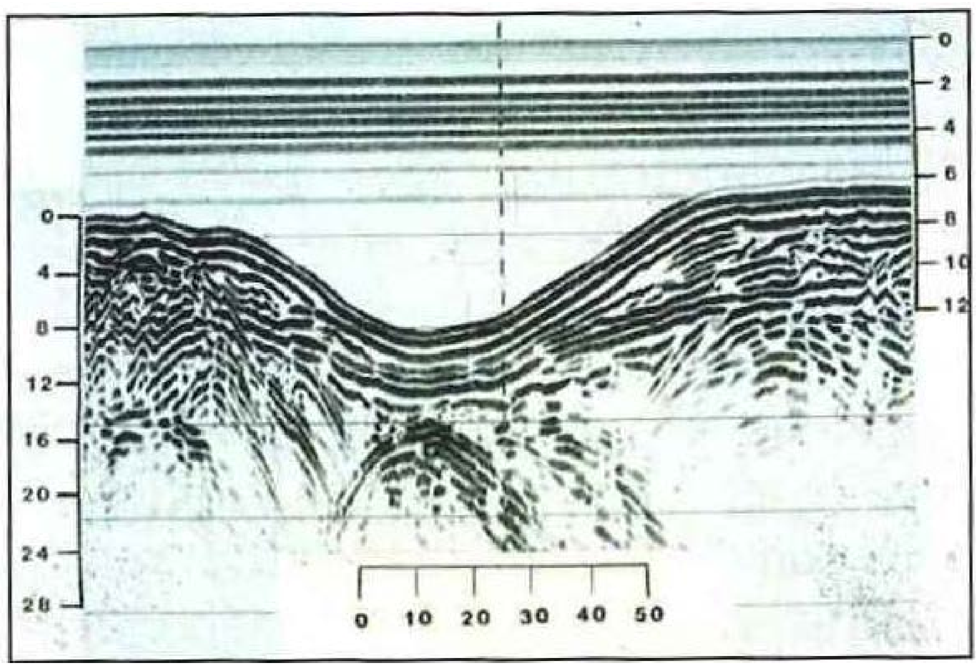

Figure 13.3.58 Ground Penetrating Radar Record

Tuned Transducers

Tuned transducers, or low-frequency sonar, is a seismic system that operates through the transmission and reception of acoustic waves. This system consists of a transmitter, a transducer towed alongside the boat, a receiver and a graphic recorder. The transmitter will produce a sound wave that is directed toward the bottom of the channel by the transducer. A portion of the sound wave will be reflected back to the surface and a portion will penetrate into the bottom of the channel. Other portions of the wave will bounce off the material once there is acoustical impedance between the layers (see Figure 13.3.59).

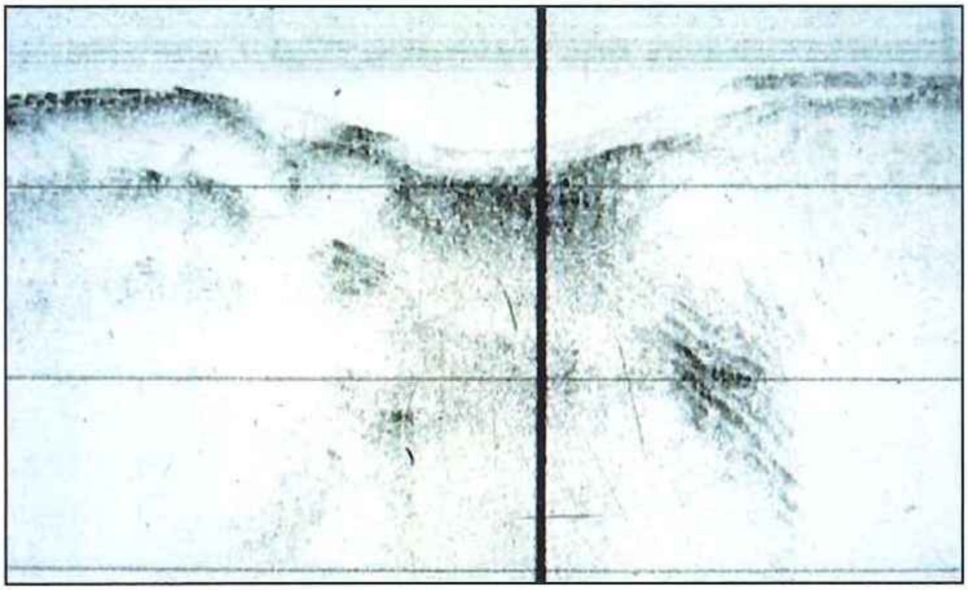

Figure 13.3.59 Tuned Transducer Record

Diver Inspections

When identifying a scour critical bridge, the diver has a limited role. Although divers may be able to identify the conditions during an underwater inspection, the greatest scour occurs at periods of high flow. Diver inspections include:

- Record bottom conditions adjacent to submerged foundations
- Detect undermining and scour holes near the upstream end of the foundation (see Figure 13.3.60)
- Detect soil build-up soil at downstream end
- Note any debris which may cause local scour
- Note type of bottom material
- Note the presence, location and size of rip rap
- Detect small diameter but deep scour holes that may expose the footing or cause undermining
- Record dimensions of undermining, if it exists
- Note any piles to be examined if they are exposed

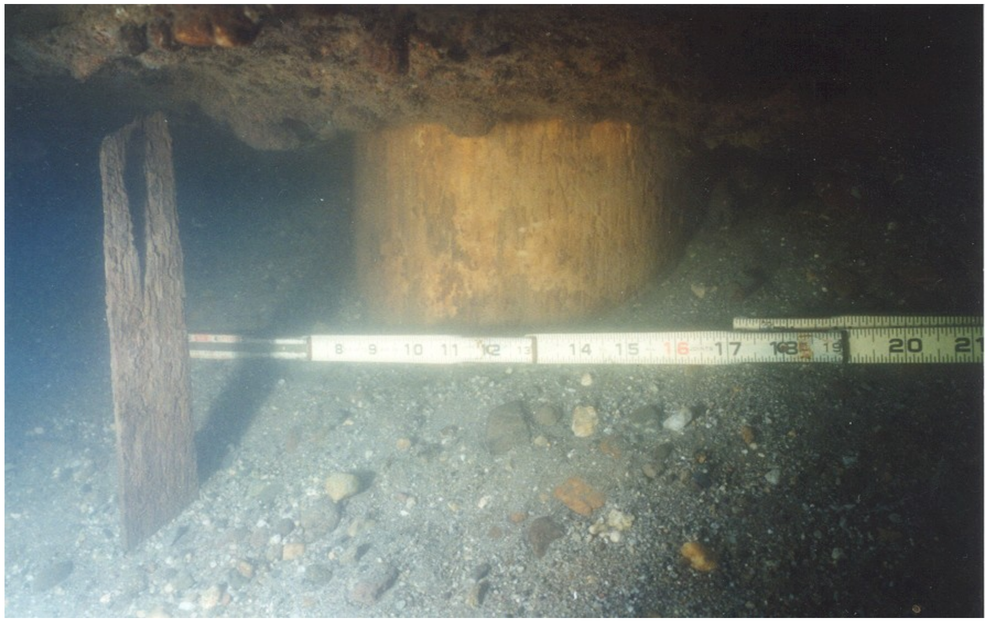

Figure 13.3.60 Pier Undermining, Exposing Timber Foundation Pile

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Topic 14.1 Culvert Characteristics

14.1.1 Introduction

A culvert is a structure designed hydraulically to take advantage of submergence to increase water carrying capacity. Culverts, as distinguished from bridges, are usually covered with embankment and are composed of structural material around the entire perimeter. Some culverts are supported on spread footings with the streambed serving as the bottom of the culvert. If culverts satisfy NBIS bridge length requirements of 20 feet or greater, they may be classified as bridges in the National Bridge Inventory (NBI).

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.

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 14.1.1). There have also been recent advances in culvert design and analysis techniques. Long span corrugated metal culverts with spans in excess of 40 feet were introduced in the late 1960's.

Figure 14.1.1 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.

Bridge-size culverts are inspected regularly to identify potential safety problems and maintenance needs. Culverts smaller than bridges may or may not be inspected, depending on the state. Preserving the investment in the structure and minimizing property damage due to improper hydraulic functioning are also key reasons for regular inspections and other maintenance actions.

Purpose of Culvert Inspection

The National Bridge Inspection Program (NBIP) 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 20 feet 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 20 feet along the centerline of the road and also includes multiple pipes where the distance between openings is less than or equal to 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 *FHWA 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 for culverts. Additionally, the NBIS program does not specifically address structures where the total opening length is less than 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 20 feet 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 are also generally applicable to culverts with openings which are less than 20 feet long. For culverts (and span-type structures) less than 20 feet in length, the state in which the structure is located will incorporate it into their inventory and inspection program. In this case, the state defines the criteria whereby culverts are to be included in the their inventory and inspection program.

Ideally, all culverts are 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 defines 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 20 feet are inspected every two years under the NBIS program. If possible, all culverts are 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 is 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 is updated whenever a replacement is accomplished.

Safety

Safety is the most important reason why culverts as well as bridges are inspected. To ensure that a culvert is functioning safely, the inspector evaluates the structural integrity, hydraulic performance, and roadside compatibility of the culvert.

- 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 due to scour or undermining.

- 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. Headwalls and wingwalls are presented in detail in Topic 14.1.6. 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 – Presented in Topic 2.2, Safety Fundamentals for Bridge Inspectors.

Maintenance Needs

Lack of maintenance is a prime cause of improper functioning of culverts and other drainage structures. Regular periodic inspections allow minor problems to be spotted and corrected before they become serious.

Outcomes

The primary outcome of this topic as well as Topics 2.1, 2.2, 3.1, 4.2, 4.3, 7.6, 13.2, 14.2, and 14.3 is to provide information that will enable bridge inspectors to perform the following tasks:

- Properly inspect an existing culvert.
- Evaluate structural adequacy.
- Evaluate hydraulic adequacy and recognize potential flood hazards.
- Correctly document and evaluate the findings of a culvert inspection using the appropriate FHWA and AASHTO criteria.
- Recognize and document traffic safety conditions.
- Recommend corrective actions/maintenance needs.

To meet the primary outcome, the topics in this reference manual provide general procedures for conducting, reporting, and documenting a culvert inspection, and guidelines for evaluating specific hydraulic and structural culvert components.

A second outcome of these sections is to provide inspectors with the information necessary to understand and evaluate the significance of defects and their effect on hydraulic and structural performance. Topics 14.2 and 14.3 present information on rigid and flexible culverts. Durability concepts are also reviewed in these topics.

14.1.2 Differentiation Between Culverts and Bridges

Traditional definitions of culverts are based on the span length rather than function or structure type. For example, the NBIS bridge length definition included in the *FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* states:

"A structure including supports erected over a depression or a obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes."

Therefore, structures that are less than 20 feet may be known as culverts.

Many structures that measure more than 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 are designed to support the permanent load of the soil over the culvert as well as transient loads including vehicular traffic. Either transient loads or permanent 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, transient live loads on culverts are generally not as significant as the permanent loads unless the cover is shallow. Box culverts with shallow cover are examples of the type of installation where transient live loads may be significant. Permanent and transient loading is presented in detail in Topic 14.1.3.

Figure 14.1.2 Box Culvert with Shallow Cover

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 scour and undermining. 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 culvert ends are located near travel 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 or differential 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.

14.1.3 Structural Characteristics of Culverts

Loads on Culverts

In addition to their hydraulic functions, culverts 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 are carried by culverts: permanent loads and transient loads.

Permanent Loads

Permanent 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, 120 pounds per cubic foot is generally assumed.

Transient Loads

The vehicular live loads and live load surcharge on a culvert include the loads and forces, which act upon the culvert due to vehicular or pedestrian traffic. The highway wheel loads (as part of the AASHTO HL-93 design load) used for design and analysis are shown in Figure 14.1.3. The effect of live loads decreases as the height of cover over the culvert increases. When the cover is less than two feet, concentrated loads may be considered as being spread uniformly over a rectangle with sides 1.15 times the depth of cover plus the initial footprint. This concept is illustrated in Figures 14.1.4 and 14.1.5. In addition to the truck load, the HL-93 is also comprised of a 640 pound lane load. This load converts into an additional 64 pounds per square foot, but may be ignored if the depth of the cover is greater than 8 feet.

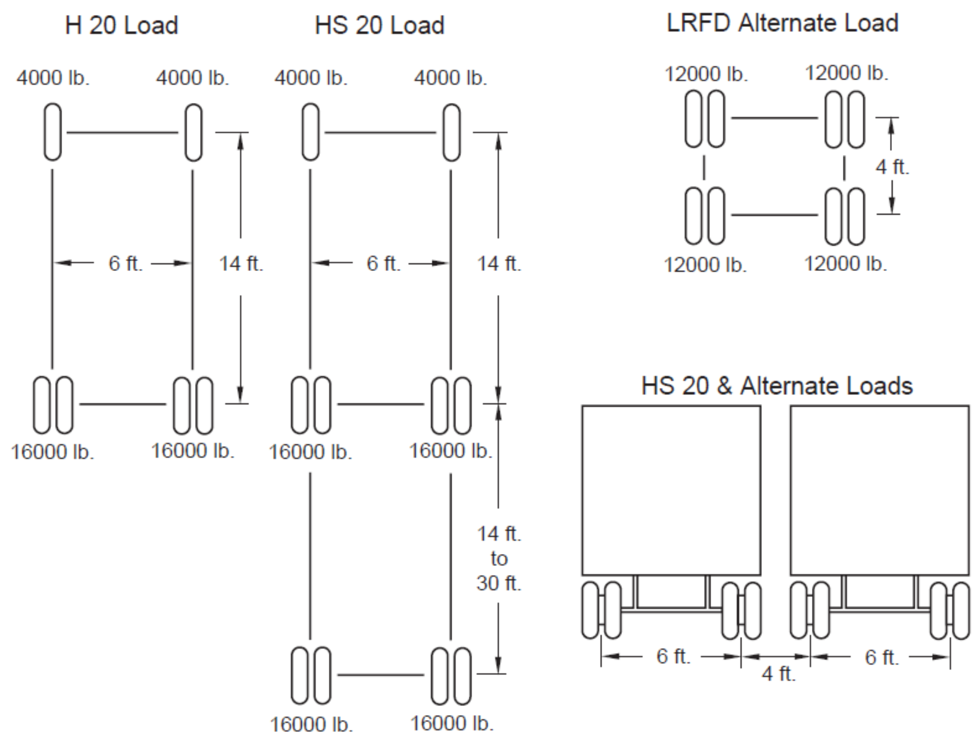


Figure 14.1.3 AASHTO Wheel Loads and Wheel Spacings

(Source: *Concrete Pipe Design Manual*, American Concrete Pipe Association, April 2007)

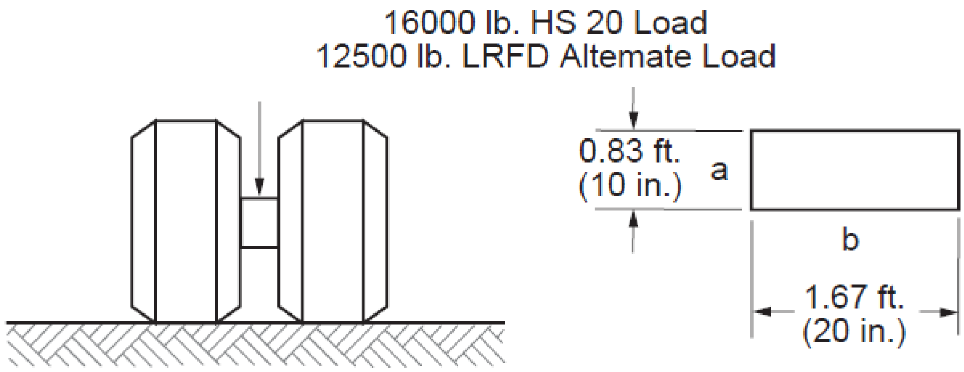
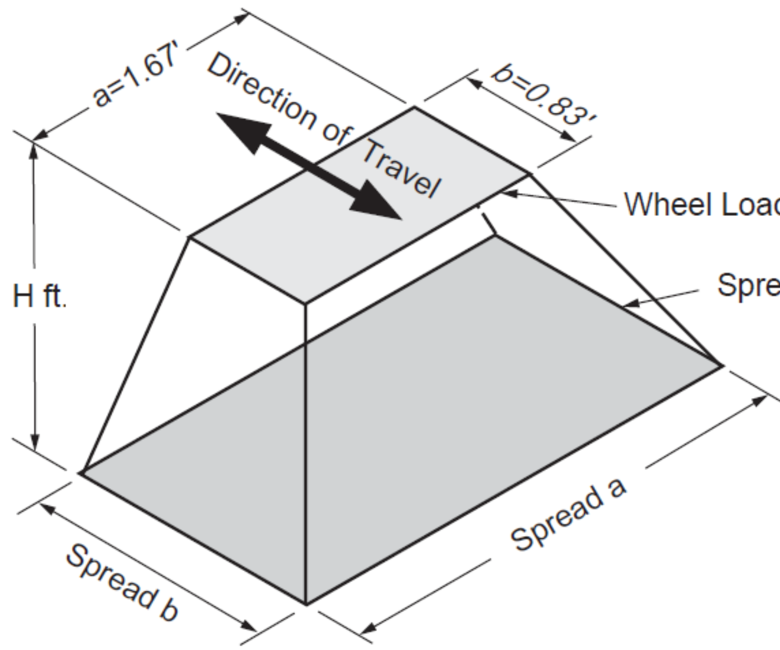


Figure 14.1.4 AASHTO Wheel Load Surface Contact Area (Foot Print)

(Source: *Concrete Pipe Design Manual*, American Concrete Pipe Association, April 2007)



Soil Type	H, ft	P, lbs	Spread a, ft	Spread b, ft
Select Granular Soil Fill	H < 2.03	16,000	a + 1.15H	b + 1.15H
Other Soils	H < 2.33	16,000	a + 1.00H	b + 1.00H

Figure 14.1.5 Spread Load Area (Single Dual Wheel)

(Source: *Concrete Pipe Design Manual*, American Concrete Pipe Association, April 2007)

Categories of Structural Materials

Based upon material type, culverts are divided into two broad structural categories: rigid and flexible.

Rigid Culverts

Culverts made from materials such as reinforced concrete or 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 presented in detail in Topic 14.2.

Flexible Culverts

Flexible culverts are commonly made from steel or aluminum. In some states composite materials are used. 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.

Flexible culverts are presented in detail in Topic 14.3.

Construction and Installation Requirements

The structural behavior of flexible and rigid culverts is often dependent on construction practices during installation (see Figure 14.1.6). 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 are 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 is shaped to a sufficient width to permit compaction of the remainder of the backfill, and enough loose material is placed on top of the bedding to fill the corrugations. When using rigid culverts, the bedding conforms to the conditions specified in the plans and is 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 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 is 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 are set to prevent waterponding or pocketing.

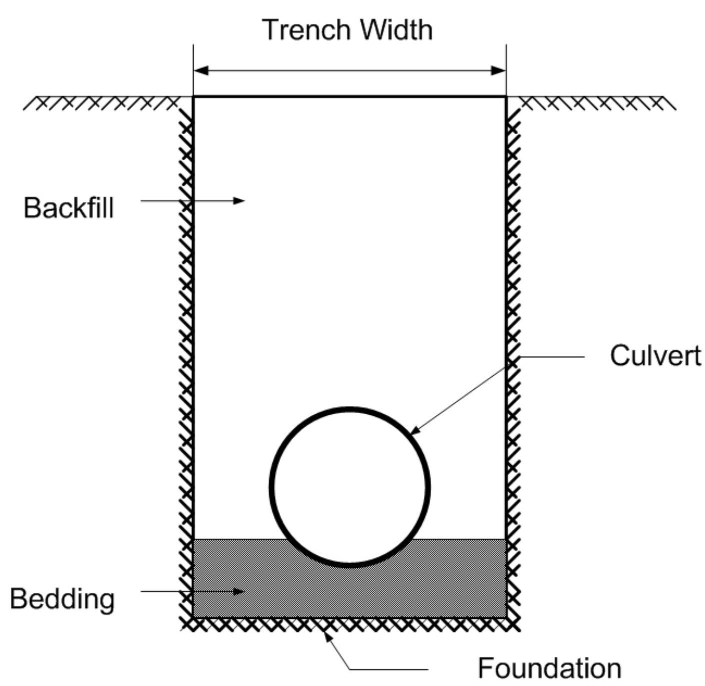


Figure 14.1.6 Culvert Construction and Installation Requirements

14.1.4 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 14.1.7). 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 14.1.7 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 14.1.8). 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 14.1.8 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 14.1.9). Foundation conditions must be adequate to support the footings. Riprap is frequently used for scour protection.

Figure 14.1.9 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 (see Figure 14.1.10). Due to the flat sides and top, rectangular shapes are not as structurally efficient as other culvert shapes. In addition, box sections have an integral floor.

Figure 14.1.10 Concrete Box Culvert

Multiple Barrels

Multiple barrels are used to obtain adequate hydraulic capacity under low embankments or for wide waterways (see Figure 14.1.11). 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 or when a culvert is constructed, excessive sedimentation is more likely to occur in any or all of the barrels based upon the conditions. 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 14.1.11 Multiple Cell Concrete Culvert

Frame Culverts

Frame culverts are constructed of cast-in-place (see Figure 14.1.12) or precast reinforced concrete. This type of culvert has no floor (concrete bottom) and fill material is placed over the structure.

Figure 14.1.12 Frame Culvert

14.1.5 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 180 inches in diameter, with larger sizes available as special designs. Standard box sections are also available with spans as large as 144 inches. Precast concrete arches on cast-in-place footings are available with spans up to 41 feet. A listing of standard sizes is provided in Topic 14.2. Refer to Topic 14.2 for a detailed discussion of precast concrete culverts.

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. Refer to Topic 14.2 for a detailed discussion of cast-in-place concrete culverts.

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 Figures 14.1.13 and 14.1.14). Several factors such as span length, vertical and horizontal clearance, peak stream flow and terrain determine which flexible culvert shape is used. Refer to Topic 14.3 for a detailed discussion of metal culverts.

Figure 14.1.13 Large Structural Plate Pipe Arch Culvert

Figure 14.1.14 Large Structural Plate Box Culvert

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 14.1.13. Refer to Topic 14.2 for a detailed discussion of stone masonry.

Figure 14.1.15 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 analogous to a short span timber bridge on timber abutments (see Figure 14.1.14). Refer to Topic 14.2 for a detailed discussion of timber culverts.

Figure 14.1.16 Timber Box Culvert

Plastic

Plastic culverts are relatively new and are not as common. They are round in shape, similar to corrugated metal culverts (see Figure14.1.17). Refer to Topic 14.3 for a detailed description of plastic culverts.

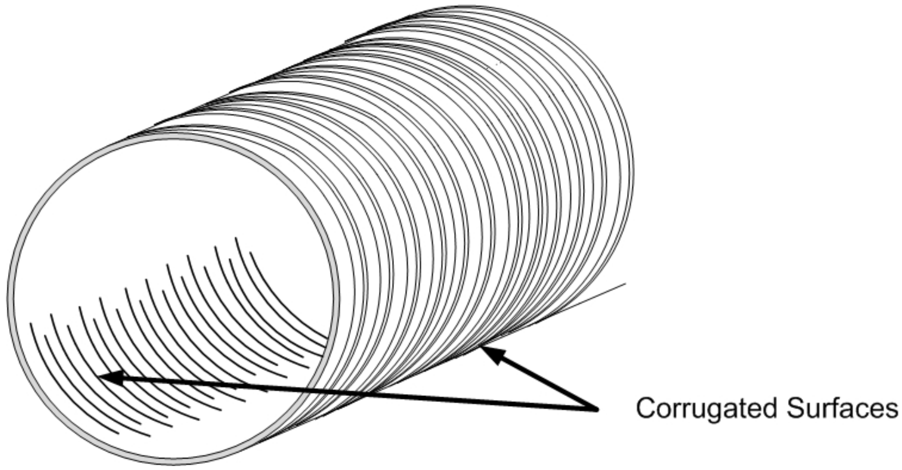


Figure 14.1.17 Schematic of a Single Walled Plastic Culvert

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, and asbestos cement. These materials are not commonly found because they are either labor intensive (terra cotta) or used for specialized situations (stainless steel and cast iron).

14.1.6 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 14.1.18).
- Mitered - The end of the culvert is cut to match the slope of the embankment. This is commonly used when the embankment has some sort of slope paving (see Figure 14.1.18).
- Skewed - Culverts, which are not perpendicular to the roadway, may have their ends cut parallel to the roadway (see Figure 14.1.20).
- Pipe end section - A section of pipe is added to the ends of the culvert barrel. These are typically used on 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 14.1.21), but can be constructed of timber or masonry. Metal headwalls are usually found on metal box culverts.

Figure 14.1.18 Culvert End Projection

Figure 14.1.19 Culvert Mitered End

Figure 14.1.20 Culvert Skewed End

Figure 14.1.21 Culvert Headwall and Wingwalls

Miscellaneous Appurtenance Structures may also be used with end treatments to improve hydraulic efficiency and reduce scour. Typical appurtenances include:

- Aprons - Used to reduce streambed scour at the inlets and outlets of culverts. Aprons are typically concrete slabs, but they may also be riprap (see Figure 14.1.22). Most aprons include an upstream cutoff wall (also known as a toe 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 14.1.23).

  Appurtenances such as aprons and energy dissipaters are subject to fast flowing water. Inspect these appurtenances to determine they are in condition to perform their intended duties. For concrete appurtenances, look for material deteriorations such as cracking, spalling, chloride contamination, abrasion and reinforcing steel corrosion. See Topic 6.2 for anticipated modes of concrete deterioration and inspection procedures for concrete.

Figure 14.1.22 Apron

Figure 14.1.23 Riprap Basin

14.1.7 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 discharges or the amount of runoff the culvert is 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, it is important for the inspector to 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, it is helpful to understand how changes in the topography of the drainage area can cause major changes in runoff. Climatic and topographic factors are briefly presented:

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
- 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
- Water regulation features such as dams and irrigation canals
- Other factors such as the type of soil and elevation

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.

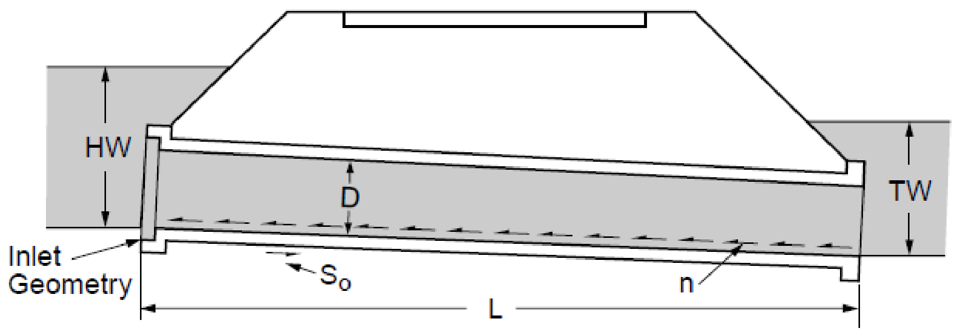
Hydraulic Analysis

The factors within a hydraulic analysis affecting a culvert's capacity may include headwater depth (see Figure 14.1.24), tailwater depth, inlet geometry, the slope of the culvert barrel, barrel area, barrel length, 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 14.1.24). 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.



D = Inside diameter for a circular pipe
HW = Headwater depth at culvert entrance
L = Length of culvert
n = Surface roughness of the pipe wall, usually expressed in terms of Manning's n
S_o = Slope of the culvert pipe
TW = Tailwater depth at culvert outlet

Figure 14.1.24 Factors Affecting Culvert Discharge (Source: Concrete Pipe Design Manual, American Concrete Pipe Association, April 2007

Most culverts, except those in flat terrain, operate under inlet control during peak flows. Since the entrance characteristics govern, minor modifications at the culvert inlet can significantly affect 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 affect 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 headwall, 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.

14.1.8 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, debris and sedimentation buildup - 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.

14.1.9 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:

- Structural - High embankments may impose very high permanent loads on all sides of a culvert and can cause shear or bending failure (see Figure 14.1.25).
- Foundation - 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 14.1.26).
- Hydraulic - 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 14.1.27).
- Debris accumulation - Branches, sediment and trash can often be trapped at the culvert entrance restricting the channel flow and causing scour (see Figure 14.1.28).

Figure 14.1.25 Bending or Shear Failure

Figure 14.1.26 Cracking of Culvert End Treatment Due to Foundation Settlement

Figure 14.1.27 Scour and Undermining at Culvert Inlet

Figure 14.1.28 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, look for high water 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. Select one end of the culvert and inspect the embankment, waterway, headwalls, wingwalls, and culvert barrel. Progress 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 13.2)
- End treatments
- Appurtenance structures
- Culvert barrel

Overall Condition

General observations of the condition of the culvert are 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. Remain observant for changes in the drainage area that might affect runoff characteristics and hydraulic analyses.

Approach Roadway and Embankment

Inspection of the approach roadway and embankment includes an evaluation of the functional adequacy (see Figure 14.1.29).

Inspect the approach roadway and embankment for the following functional requirements:

- Signing
- Alignment
- Clearances
- Adequate shoulder profile
- Safety features

Figure 14.1.29 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. Inspect the approach roadway and embankment 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

Examine approach roadways for sudden dips, cracks, and sags in the pavement. These usually indicate excessive deflection of the culvert or inadequate compaction of the backfill material.

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 (see Figure 14.1.30).

Figure 14.1.30 Repaired Roadway Over a Culvert

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

Inspect the embankment around the culvert entrance and exit for slide failures in the fill around the box (see Figure 14.1.31). Check for debris at the inlet and outlet and within the culvert. Also note if vegetation is obstructing the ends of the culvert.

Figure 14.1.31 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 such as a plumb bobs, hammers, and probing rods are used to check for misalignment, sound for defects, and check for scour and undermining. In general, inspect headwalls for movement or settlement, cracks, deterioration, and traffic hazards (see Figure 14.1.32). Check culvert ends for undermining, scour, and evidence of piping.

Figure 14.1.32 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.

Inspect wingwalls to ensure they are in proper vertical alignment (see Figures 14.1.32 and 14.1.33). Wingwalls may be tilted due to settlement, slides or scour. See Topic 12.1 for a detailed description of defects and inspection procedures of wingwalls.

Figure 14.1.33 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 14.1.34). Stresses increase because a full box shape is not present at the end.

Figure 14.1.34 Skewed End

Headwalls – Inspect headwalls and wingwalls for undermining and settlement. Cracking, tipping or separation of culvert barrel from the headwall and wingwalls is usually evidence of undermining (see Figure 14.1.35).

Figure 14.1.35 Culvert Headwall and Wingwall End Treatment

Appurtenance Structures Typical appurtenance structures are:

- Aprons
- Energy Dissipators

Aprons – Check aprons for any undermining or settlement. Also inspect the joints between the apron and headwalls to see if they are watertight (see Figure 14.1.36). Piping may occur is water is allowed contact with the culvert outer surfaces.

Figure 14.1.36 Apron

Energy Dissipaters – Energy dissipaters may include stilling basins, riprap or other devices. Inspect energy dissipaters for material defects, settlement, undermining, and overall effectiveness (see Figure 14.1.37).

Figure 14.1.37 Energy Dissipater

Culvert Barrel

Inspect the full length of the culvert from the inside. Visually examine all components of the culvert barrel including walls, floor, top slab, and joints. 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.

Inspect the culvert barrels for defects such as misalignment, joint defects, cracking, spalling, section loss, and other material defects. For a detailed description of culvert inspection, refer to Topic 14.2 for rigid culverts or Topic 14.3 for flexible culverts.

14.1.10 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. See Topics 14.2 and 14.3 for detailed explanations on how abrasion and corrosion affects the durability of rigid and flexible culverts.

14.1.11 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
- Electrical Resistivity
- Soil Characteristics

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 produce 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.

14.1.12 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. Make field repairs 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 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 reinforced 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.

Table of Contents

Chapter 14
Characteristics, Inspection and Evaluation of Culverts

Topic 14.2 Rigid Culverts

14.2.1 Introduction

Culverts are classified as rigid culverts when the load-carrying capacity of the culvert is primarily provided by the structural strength of the culvert, with little strength developed from the surrounding soil. By this definition, rigid culverts do not bend or deflect appreciably when loaded.

Unlike bridges, culverts have no distinction between substructure and superstructure. Culverts also have no "deck", since earth backfill separates the culvert structure from the riding surface (see Figure 14.2.1).

Figure 14.2.1 Rigid Culvert

14.2.2 Design Characteristics

Concrete Culverts

Concrete culverts are the most common type of rigid culverts used today. Types of concrete culverts include:

- Concrete box culverts (either cast-in-place or precast)
- Concrete pipe culverts
- Concrete arch culverts
- Concrete frame culverts

See Figures 14.2.25a through 14.2.25c at the end of this topic for standard sizes of concrete pipes and Figure 14.2.26 at the end of this topic for standard concrete pipe shapes.

Concrete Box Culverts

One of the most common rigid culverts used today is the concrete box culvert (see Figure 14.2.2). A box culvert has an integral bottom slab that supports the side walls and provides a lined channel for the water to flow. The dimensions of the box culvert are determined by hydraulic, structural and geotechnical design criteria, as well as site constraints, which include channel dimensions and the amount of available cover. 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 multi-cell box culvert can be used (see Figure 14.2.3). 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.

Figure 14.2.2 Concrete Box Culvert

Figure 14.2.3 Multi-Cell Concrete Box Culvert

There are two basic types of concrete box culverts: cast-in-place and precast. Precast concrete box culverts are generally the preferred type of concrete box culvert. For situations with complex site geometries or other special applications, cast-in-place concrete box culverts may be the preferred choice.

Cast-in-Place

Reinforced cast-in-place (CIP) concrete box culverts are typically 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. Cast-in-place box culverts are also generally preferred for special applications, such as side- or slope-tapered inlets, aquatic organism passage, or customized fit with other infrastructure including additional culverts, stormdrains and drop inlets.

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. One of the major advantages of precast concrete box culverts is the increased speed of construction. Standard box sections are available with spans as large as 12 feet (see Figure 14.2.4). Some box sections may have spans of up to 20 feet if a special design is used.

ASTM C 1433 Precast Reinforced Concrete Box Sections for Culverts, Storm Drains and Sewers is an industry recognized reference. These specifications cover single-cell precast reinforced concrete box sections intended to be used for the construction of culverts for the conveyance of storm water, industrial wastes, and sewage.

Figure 14.2.4 Precast Concrete Box Culvert

Concrete Pipe Culverts

Precast concrete pipe culverts are manufactured in three standard shapes:

- Circular
- Horizontal elliptical
- Vertical elliptical

Circular pipe culverts are very common (see Figure 14.2.5). In situations where the required size of the opening is very large, two or more concrete pipe culverts may be used (see Figure 14.2.6).

Figure 14.2.5 Concrete Pipe Culvert

Figure 14.2.6 Twin Concrete Pipe Culvert

The size of the opening is primarily determined by the following factors: a) magnitude of the peak design flow; b) allowable headwater (pooled water surface) at the inlet for the peak design flow; c) permissible barrel and outlet flow velocities; and d) aquatic organism passage design considerations. The circular shape is the most common shape manufactured for pipe culverts. It is hydraulically and structurally efficient under most conditions. Elliptical shapes are used in situations where horizontal or vertical clearance is limited. The oblong shape allows the pipe to fit where a circular pipe may not, but still allows for the necessary size opening. Elliptical shaped pipe culverts may also be used when a wider section is desirable for low flow levels. No matter the shape, a pipe culvert tends to reduce the flow area of the design discharge, and possibly lesser flows, thereby increasing the flow velocity. An increased flow velocity has greater potential to scour the streambed at the outlet of the pipe.

Concrete culvert pipe is manufactured in up to five standard strength classifications. Higher classification numbers indicate higher strength. All of these standard shapes are manufactured in a wide range of sizes. Circular and elliptical pipes are available with standard sizes as large as 12 feet in diameter, with larger sizes available for special designs. Several factors such as span length, vertical and horizontal clearance, peak stream flow and terrain determine which shape of pipe culvert is used.

Concrete Arch Culverts

An arch culvert is a curved-shape culvert that works primarily in compression and does not have a bottom, or floor (see Figure 14.2.7). This type of culvert, as well as embedded culverts (i.e., culverts having buried inverts), are commonly and effectively used at stream crossings required to provide aquatic organism passage.

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 are either cast-in-place or precast.

Figure 14.2.7 Concrete Arch Culvert

Concrete Frame Culverts

Concrete frame culverts are either cast-in-place or precast reinforced concrete, which is generally shaped similar to a box culvert. It differs from a box culvert, however, since there is no floor in a frame culvert (see Figure 14.2.8). Rigid culverts with a natural bottom (by way of embedment or having an open bottom) are commonly used to provide for aquatic organism passage.

Figure 14.2.8 Concrete Frame Culvert

Masonry Culverts

Stone Masonry Arch Culverts

Stone and brick are durable, low maintenance materials. Currently stone and brick are seldom used for constructing new culvert barrels. Stone masonry culverts, when constructed, were usually in the shape of an arch (see Figure 14.2.9).

Figure 14.2.9 Stone Masonry Arch Culvert

Timber Culverts

Timber Box Culverts

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 (see Figure 14.2.10). These culverts are normally utilized in areas of seasonal flows, such as heavy flow in the spring and little to no flow during the summer months.

Figure 14.2.10 Timber Box Culvert

Loads on Culverts

There are several basic loads applied in the design of a culvert and include:

- 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)

Box Culverts

Box culverts face similar types of loads on each slab and wall of the culvert (see Figure 14.2.11).

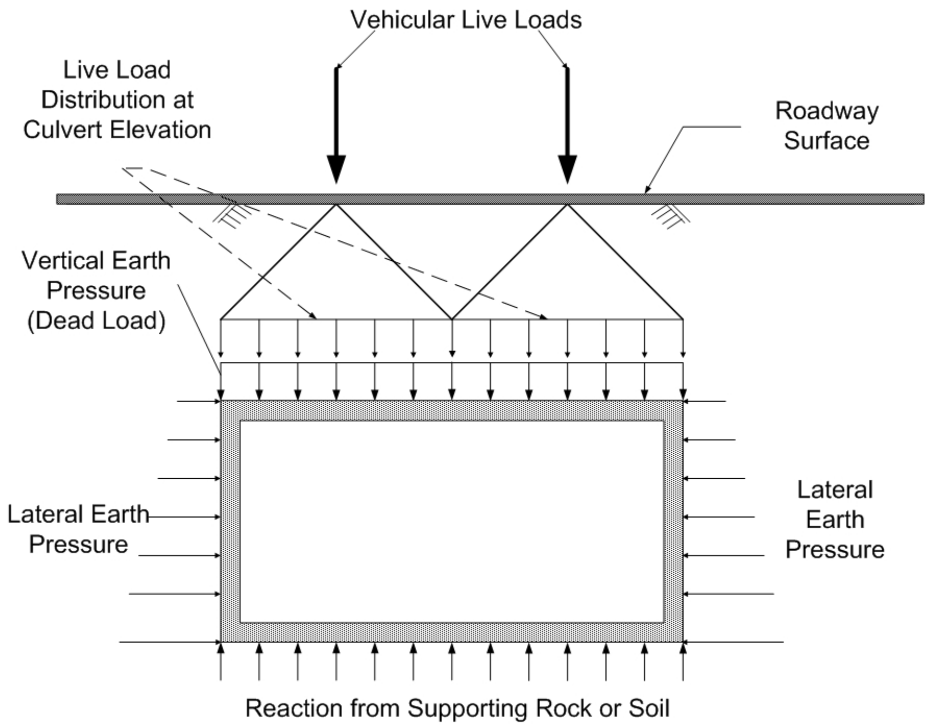


Figure 14.2.11 Loads on a Concrete and Timber Box Culvert

Pipe Culverts

Pipe culverts are subject to the same types of forces that are placed upon the box culverts which are dead loads, vertical earth pressure, horizontal earth pressure, and live loads

Arch and Frame Culverts

Arch and frame culverts have the same types of loads as box culverts.

For a detailed description of loads on pipe, arch and frame culverts, see Topic 14.1.3.

Primary and Secondary Members

Primary members for culverts may vary based upon the type of culvert. Primary members for the various types of culverts are:

- Box culverts – top slab, bottom slab and the walls (webs)
- Frame culverts – top slab, wall (webs), foundation and footing
- Arch culverts – culvert barrel, foundation and footing
- Pipe culverts – culvert barrel

There are no secondary members for the culvert barrels. Wingwalls and headwalls are discussed in Topic 14.2.4 inspection locations.

Steel Reinforcement for Concrete Culverts

Steel reinforcement for culverts is in the form of either primary or secondary reinforcement. Depending upon the potential for corrosion, chemical attack or other steel reinforcement deficiencies, states may use epoxy-coated reinforcing bars. Some states have also incorporated stainless steel reinforcement into concrete culverts.

Primary Reinforcement

The primary reinforcing steel for box culverts resists tension and shear forces. Tension reinforcement is placed transversely in the box culvert slabs and vertically in the walls. Shear reinforcement may be placed diagonally in each of the box culvert corners (see Figure 14.2.12). Single cell precast concrete box culverts may use steel welded wire for tension and shear reinforcement.

Primary reinforcement for arch (see Figure 14.2.14) and pipe culverts (see Figure 14.2.15) also resists tension and shear. Arch and pipe culvert primary reinforcement is placed transversely in the walls of the culverts.

Secondary Reinforcement

Longitudinal temperature and shrinkage reinforcement is placed in the slabs and the walls of box culverts (see Figure 14.2.12).

Ducts may be provided in the precast box sections for optional longitudinal post-tensioning of the boxes with high strength steel strands or bars (see Figure 14.2.13).

Secondary reinforcement for arch (see Figure 14.2.14) and pipe culverts (see Figure 14.2.15) follow the shape of the culvert itself from support to support.

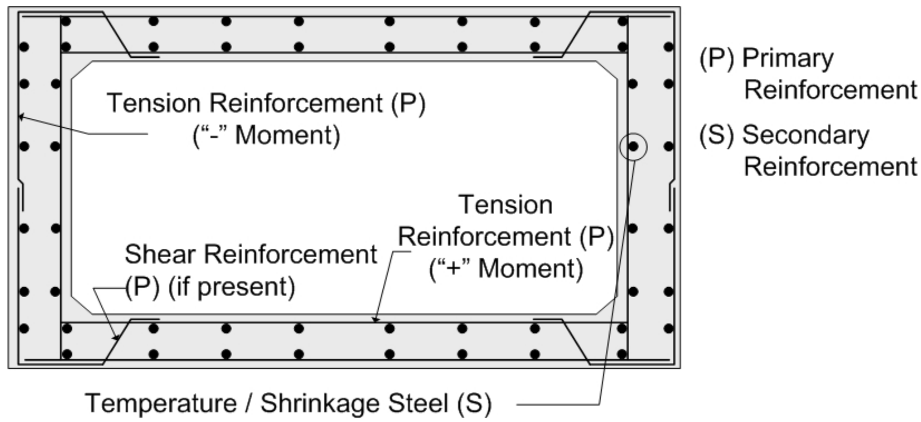


Figure 14.2.12 Steel Reinforcement in a Concrete Box Culvert

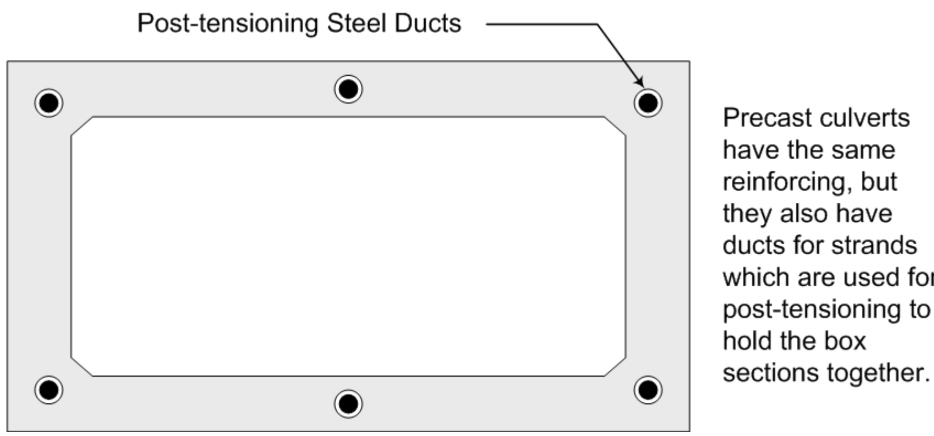


Figure 14.2.13 Precast Box Section with Post-tensioning Steel Ducts

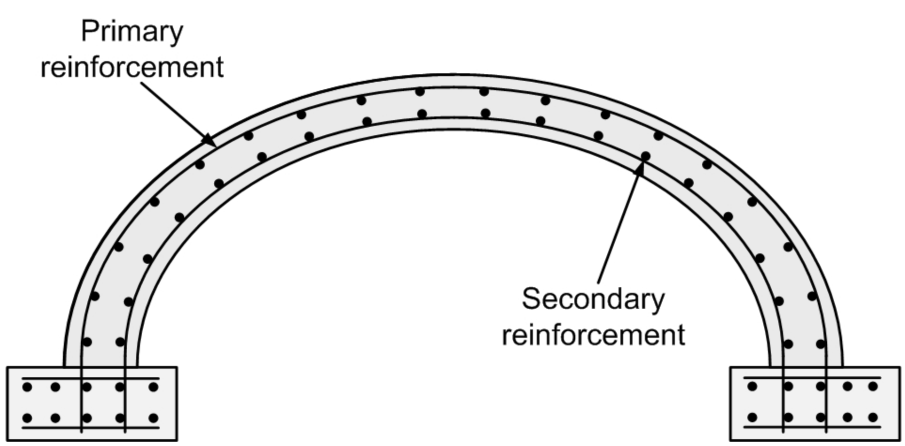


Figure 14.2.14 Steel Reinforcement in a Concrete Arch Culvert

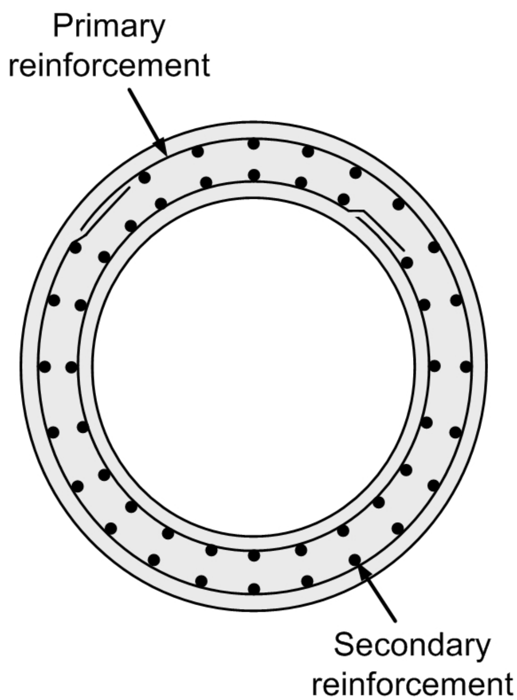


Figure 14.2.15 Steel Reinforcement in a Concrete Pipe Culvert

14.2.3 Overview of Common Deficiencies

Common deficiencies that occur in concrete rigid culverts include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali-Silica Reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion

- Loss of prestress
- Carbonation

Refer to Topic 6.2.6 for a detailed explanation of the properties of concrete, types and causes of concrete deterioration, and the examination of concrete.

Common deficiencies that occur in masonry rigid culverts include:

- Weathering
- Spalling
- Splitting
- Fire damage
- Embankment scour at culvert inlet and outlet
- Roadway settlement

Refer to Topic 6.5.4 for a detailed explanation of the properties of masonry, types and causes of masonry deterioration, and the examination of masonry.

Common deficiencies that occur in timber rigid culverts include:

- Inherent defects (checks, splits, shakes, knots)
- Fungi
- Insects
- Marine borers
- Chemical attack
- Delaminations
- Loose connections
- Surface depressions
- Fire
- Impact or collisions
- Abrasion and mechanical wear
- Overstress
- Weathering or warping
- Protective coating failure
- Embankment scour at culvert inlet and outlet
- Roadway settlement

Refer to Topic 6.1.5 for a detailed explanation of the properties of timber, types and causes of timber deterioration, and the examination of timber.

14.2.4 Inspection Methods and Locations

Previous inspection reports and as-built plans, when available, are reviewed prior to, and during, the field inspection. Review of previous reports familiarizes the inspector with the structure and makes detection of changed conditions easier. Reviewing the previous inspection reports 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 is conducted. In addition to the culvert components, the inspector looks for high-water 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.

Methods

Inspection methods for various rigid culvert materials include timber Topic 6.1.7, concrete Topic 6.2.8, and masonry Topic 6.5.6.

Visual

Concrete

The inspection of concrete culverts for cracks, spalls, and other deficiencies is primarily a visual activity.

Masonry

The inspection of masonry culverts for cracks, loose or missing mortar, vegetation, water seepage, crushing, missing stones, bulging, and misalignment is primarily a visual activity.

Timber

The inspection of timber culverts for checks, splits, shakes, fungus decay, deflection, and loose fasteners is primarily a visual activity.

Physical

Concrete

Hammer sounding of the exposed concrete is performed to determine areas of delamination. A delaminated area has a distinctive hollow “clacking” sound when tapped with a hammer. A hammer hitting sound concrete results in a solid “pinging” type sound.

Masonry

Physical inspection of a masonry culvert is similar to that of concrete.

Timber

Hammer sounding of the exposed timber is performed to determine areas of internal decay. If the area has internal decay, there is a hollow sound when the hammer is tapped.

Advanced Inspection Methods

Concrete/Masonry

Several advanced methods are available for concrete and masonry inspection. Nondestructive methods, described in Topic 15.2.2, include:

- Acoustic Wave Sonic/Ultrasonic Velocity Measurements
- Electrical Methods
- Delamination Detection Machinery
- Ground-Penetrating Radar
- Electromagnetic Methods
- Pulse Velocity
- Flat Jack Testing
- Impact-Echo Testing
- Infrared Thermography
- Laser Ultrasonic Testing
- Magnetic Field Disturbance
- Neutron Probe for Detection of Chlorides
- Nuclear Methods
- Pachometer
- Rebound and Penetration Methods
- Ultrasonic Testing
- Smart Concrete

Other methods, described in Topic 15.2.3, include:

- Carbonation
- Concrete Permeability
- Concrete Strength
- Endoscopes and Videoscopes
- Moisture Content
- Petrographic Examination
- Reinforcing Steel Strength
- Chloride Test
- ASR Evaluation

Timber

Several advanced methods are available for timber inspection. Nondestructive methods, described in Topic 15.1.2, include:

- Sonic testing
- Spectral analysis
- Ultrasonic testing
- Vibration

Other methods, described in Topic 15.1.3, include:

- Boring or drilling
- Moisture content
- Probing
- Field Ohmmeter

Locations

Areas Subjected to Movement and Misalignment

Vertical Movement

Vertical movement can occur in the form of uniform settlement or differential settlement. Uniform settlement has little effect on the culvert. However, differential settlement can produce severe distress which varies in magnitude based upon the span length. This may cause cracking of the culvert. See Topic 6.2.6 for a detailed presentation of concrete deficiencies including cracking. Common causes of vertical movement are soil bearing failure, consolidation of soil, scour, undermining, and subsidence from mining or solution cavities. Locations to inspect for vertical movement include the following:

- Railing for evidence of settlement
- Existing and new cracks in the roadway pavement or concrete
- Check for scour and undermining around the culvert footing or foundation

Lateral Movement

Lateral movement occurs when the horizontal earth pressure acting on the walls exceeds the friction forces that hold the structure in place. Common causes of lateral movement are slope failure, seepage, changes in soil characteristics (i.e. frost and ice), and time consolidation of the original soil. Locations to inspect for lateral movement include the following:

- General alignment
- Settled approach pavement
- Clogged drain or weep holes

Rotational Movement

Rotational movement, or tipping, of the culvert is generally the result of unsymmetrical settlements or lateral movements due to horizontal earth pressure. Common causes are undermining, scour, saturation of backfill, and improper design. Locations to inspect for rotational movement include the following:

- Vertical alignment of the walls
- Clogged drains or weep holes
- Cracks

Vertical and horizontal misalignment is checked by visual observation. Look for culvert sagging, cracking or separation of joints in precast culverts. Sags can best be detected during low flows by looking for areas where the water is deeper or where sediment has been deposited. Sags may also trap water which may further aggravate settlement problems by saturating the soil.

When excessive accumulations of sediment are present, it may be necessary to have the sediment removed before checking for sags. An alternate method is to take profile elevations of the top slab. Check horizontal alignment or bulging for straightness or smooth curvature for those culverts that were constructed with a curved alignment. It can be checked by sighting along the walls and by examining joints for differential movement (see Figure 14.2.16).

Alignment problems may be caused by improper installation, undermining or uneven settlement of the fill. It is important to determine which of these problems may be causing the settlement. If it is determined that undermining is the cause, notify maintenance forces since the damage will continue until the problem is corrected. Also, try to determine whether the undermining is due to piping (loss of fill from underneath the culvert), water exfiltration or infiltration of backfill material. Look for holes in the downstream side embankment. If the misalignment is due to improper installation or uneven settlement, repeat inspections may be necessary to determine if the settlement is progressing or if it has stabilized.

Figure 14.2.16 Sighting Along Culvert Sidewall to Check Horizontal Alignment

Bearing Areas

Bearing zones for rigid culverts will be located where the footing is supported by the earth. For concrete and masonry culverts, look for cracking and spalling. In timber culverts, look for crushing.

Shear Zones

Horizontal and vertical forces can cause high shear zones in culvert walls or slabs. For concrete and masonry culverts, look for diagonal cracking. In timber culverts, look for splitting.

Flexural Zones

High flexural moments are caused by horizontal and vertical forces which occur at the slabs and culvert walls. These moments cause compression and tension depending on the load type and location of the neutral axis. Look for deficiencies caused by overstress due to compression or tension caused by flexural moments. Check compression areas for splitting, crushing or buckling. Check tension areas for cracking or distortion.

Areas Exposed to Drainage

Examine areas that are exposed to drainage for decay on timber culverts. For concrete culverts, examine for spalling, delamination and exposed rebar (see Figure 14.2.17). Also inspect concrete culvert headwalls and wingwalls, since these areas are often exposed to surface drainage carrying road salts, which chemically attack and destroy the walls. In masonry culverts, look for spalling, delamination, and seepage which can result in stone and mortar deterioration with the eventual loosening and/or the loss of stones (see Figure 14.2.18).

Figure 14.2.17 Spalls and Delaminations on Top Slab of Concrete Box Culvert

Figure 14.2.18 Missing Stones in Masonry Culvert

Areas Exposed to Traffic

Check for collision damage from vehicles passing adjacent to the culvert.

Damage to concrete culverts may include spalls and exposed reinforcement and possibly steel reinforcement section loss. Damage to timber culverts includes split or broken members.

Scour and Undermining

Scour is the removal of material from a streambed as a result of the erosive action of running water. Scour can cause undermining or the removal of supporting foundation material from beneath the culvert. Refer to Topic 13.2 for a more detailed description of scour and undermining.

Inspection for scour includes probing around the culvert inlet and outlet for signs of undermining. Sometimes silt loosely fills in a scour hole and offers no protection or bearing capacity for the culvert inlet and outlet. Also check timber culverts frames (no floors) for these conditions.

Joints

Expansion joints are 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 (see Figure 14.2.19). When inspecting a joint in a rigid culvert, be sure to check for the following deficiencies:

- Exfiltration - This occurs when leaking joints allow water flowing through the culvert to leak into the supporting material. Minor leaking may not be a significant problem, but if the leaking joint contributes to or the loss of supporting material (also known as piping), a serious misalignment of the culvert or failure may occur. Leaking joints may be detected during low flows visually and by checking around the ends of the culvert for piping.
- Infiltration - This occurs when water is flowing or seeping into the culvert through open joints, which may allow supporting soil into the culvert. Infiltration occurs when the water is higher than the culvert inlet, with the water seeping into the culvert. This can cause settlement and misalignment if the water carries soil particles from the backfill. Infiltration may be difficult to detect visually in its early stages, but it may be indicated by open joints, staining at the joints on the sides and top of the culvert, deposits of soil in the invert, or depressions over the culvert.
- Cracks - 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. Cracks may also indicate improper handling during installation, improper gasket placement, and movement or settlement of the culvert sections. If no other problems other than cracks are evident, such as differential movement between culvert sections, and the cracks are not open or spalling, they could be considered a minor problem. Severe cracking at the joints will be similar in significance to separated joints.
- Separated joints - Joint inspection also identifies any joints that are opened widely or are not open to uniform width. Joint separations are significant because they accelerate the damage caused by exfiltration and infiltration, resulting in the erosion of the backfill material. They are noted when severe misalignment is observed. Longitudinal movement of the soil in the general direction of the culvert's centerline could cause the sections to pull apart. The slippage of the embankment may also cause a joint separation to occur.

Figure 14.2.19 Precast Concrete Box Culvert Joint

Cracks

Longitudinal Cracks

Concrete is strong in compression but weak in tension. Reinforcing steel is provided to accommodate the tensile stresses. Hairline longitudinal cracks in the crown or invert indicate that the steel has accepted part of the load. Cracks less than 0.01 inches in width are minor and only need to be noted in the inspection report. Document cracks greater than 0.01 inches in width but less than 0.1 inches, in the inspection report and noted as possible candidates for maintenance. Longitudinal cracking in excess of 0.1 inches in width may indicate overloading or poor bedding. If the pipe is placed on hard material and backfill is not adequately compacted around the pipe or under the haunches of the pipe, loads will be concentrated along the bottom of the pipe and may result in flexure or shear cracking (see Figure 14.2.20).

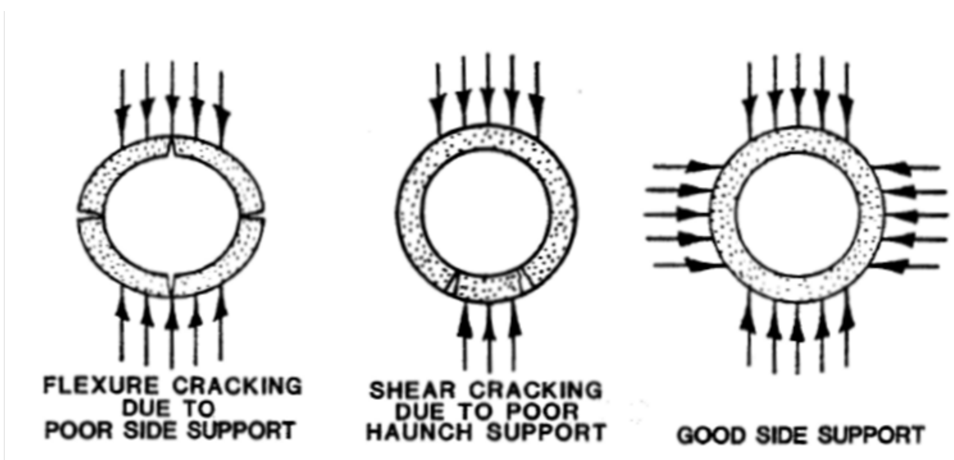


Figure 14.2.20 Longitudinal Cracks in Pipe Culvert

Also note other signs of distress such as differential movement, efflorescence, spalling, or rust stains. When cracks are wider than 0.1 inches, take measurements of the fill height and the diameter of the pipe both horizontally and vertically to permit analysis of the original design. Crack measurements and photographs are useful for monitoring conditions during subsequent inspections.

Transverse Cracks

Transverse cracks may also be caused by poor bedding (see Figure 14.2.21). Cracks can occur across the bottom of the pipe (broken belly) when the pipe is only supported at the ends of each section. This is generally the result of poor installation practices such as not providing indentions (bell holes) in hard foundation material for the ends of bell and spigot-type pipe or not providing a sufficient depth of suitable bedding material. Cracks may occur across the top of pipe (broken back) when settlement occurs and rocks or other areas of hard foundation material near the midpoint of a pipe section are not adequately covered with suitable bedding material.

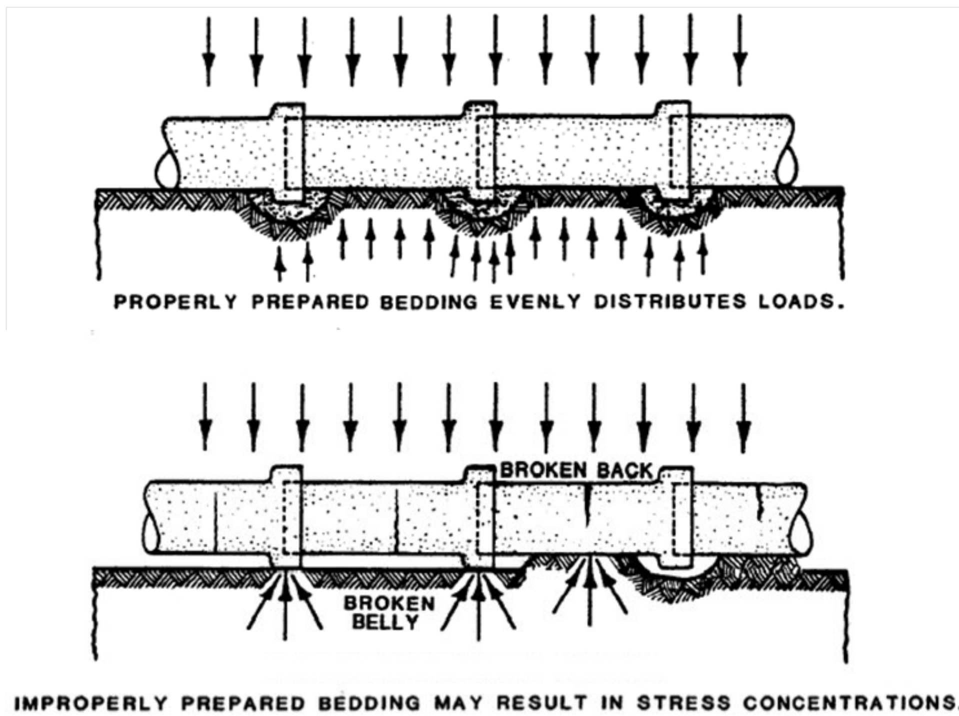


Figure 14.2.21 Transverse Cracks in Pipe Culvert

Spalls

A spall is a depression in the concrete resulting from the separation and removal of a portion of the surface concrete, revealing a fracture roughly parallel to the surface of the concrete. In precast concrete culverts, spalls often occur along the edges of either longitudinal or transverse cracks when the crack is due to overloading or poor support rather than simple tension cracking. Spalling may also be caused by the corrosion of the steel reinforcing when water is able to reach the steel through cracks or shallow cover. As the steel corrodes, the oxidized steel expands, causing the concrete covering the steel to spall. Spalling may be detected by visual examination of the concrete along the edges of cracks. Perform tapping with a hammer along cracks to check for areas that have fractured but are not visibly separated. These areas will produce a hollow sound when tapped. These areas may be referred to as delaminations.

Slabbing

Slabbing, also known as shear-slabbing or slab shear, refers to a radial failure of the concrete which occurs from straightening of the reinforcement cage due to excessive deflection. This is characterized by large slabs of concrete "peeling" away from the sides of the pipe and a straightening of the reinforcing steel (see Figure 14.2.22). Slabbing may be a severe problem that can occur under high fills.

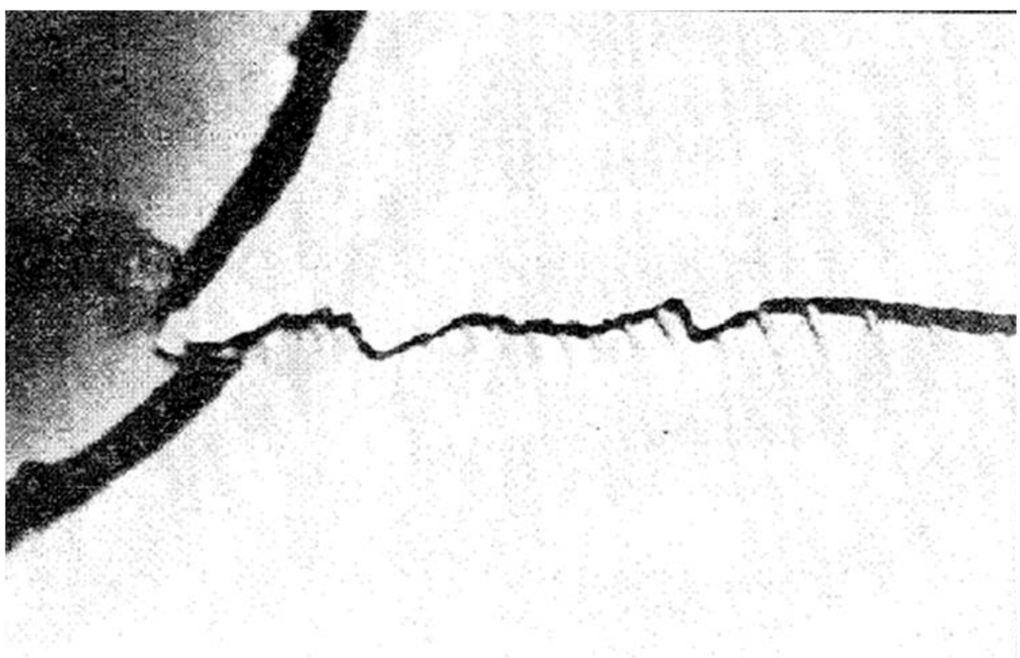


Figure 14.2.22 Shear Slabbing (Source: *FHWA Culvert Inspection Manual*)

Durability

Durability is a measure of a culvert's ability to withstand chemical attack and abrasion. Rigid culvers are subject to chemical attack in strongly acidic environments such as drainage from mines and may also be damaged by abrasion. Note any mild deterioration or abrasion that is less than 1/4 inch deep in the inspection report. Document severe surface deterioration greater than 1/4 inch deep as a potential candidate for maintenance. When the invert is completely deteriorated, it may be considered a critical finding. Note in the report when linings are used to protect against chemical attack or abrasion. Also document the condition of the lining, if present.

End Section Drop-off

This type of distress is usually due to outlet erosion as discussed earlier in the sections on end treatments and waterways. It is caused by the erosion of the material supporting the pipe sections on the outlet end of the culvert barrel.

Wingwalls and Headwalls

Wingwalls and headwalls are provided to support the embankment around the openings of the culvert (see Figure 14.2.23). Inspect wingwalls for differential settlement and proper vertical alignment. See Topic 14.1 for general culvert characteristics including wingwalls and Topic 12.1 for a detailed description of deficiencies and inspection methods of wingwalls.

Figure 14.2.23 Cast-in-Place Concrete Headwall and Wingwall

14.2.5 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of rigid culverts. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the *AASHTO Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-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. Component condition rating codes range from 9 to 0, where 9 is the best rating possible. See Topic 4.2 (Item 62) for additional details about NBI component condition rating guidelines. Item 62 component condition rating guidelines are included in Figure 14.2.24. It is also important to note that Items 58-Deck, 59-Superstructure, and 60-Substructure are coded “N” for culvert structures.

For rigid culverts, the NBI component condition rating guidelines yield a one-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, headwalls and wingwalls. Integral wingwalls are included in the evaluation up to the first construction or expansion joint. The one-digit code that best describes the culvert’s overall condition is chosen, and the component condition rating codes range from 9 to 0, where 9 is the highest possible component condition rating.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating.

Element Level Condition State Assessment

In an element level condition state assessment of a rigid culvert, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	**Description**
Substructure	
241	Reinforced Concrete Culvert
242	Timber Culvert
244	Masonry Culvert
243	Other Culvert

BME No.	**Description**
Wearing Surfaces and Protection Systems	
521	Concrete Protective Coating

The unit quantity for culverts is feet and represents the culvert length along the barrel multiplied by the number of barrels (for multiple barrel culverts). The inspector visually evaluates each 1-foot slice of the culvert barrel(s) and assigns the appropriate condition state description. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coatings is square feet, with the total area distributed among the four condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of rigid culverts:

Defect Flag No.	**Description**
358	Concrete Cracking
359	Concrete Efflorescence
360	Settlement
361	Scour
368	Barrel Distortion

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

The culvert 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 shall be included in the evaluation.

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.

Figure 14.2.24 NBI Component Condition Rating Guidelines for Culverts

Dimensions and Approximate Weights of Concrete Pipe

*ASTM C 76 – Reinforced Concrete Culvert, Storm Drain and Sewer Pipe, Tongue and Groove Joints						
	WALL A		WALL B		WALL C	
Internal Diameter inches	**Minimum Wall Thickness, inches**	**Approximate Weight, pounds per foot**	**Minimum Wall Thickness, inches**	**Approximate Weight, pounds per foot**	**Minimum Wall Thickness, inches**	**Approximate Weight, pounds per foot**
96	8	2710	9	3090	9 ¾	3355
102	8 ½	3078	9 ½	3480	10 ¼	3760
108	9	3446	10	3865	10 ¾	4160

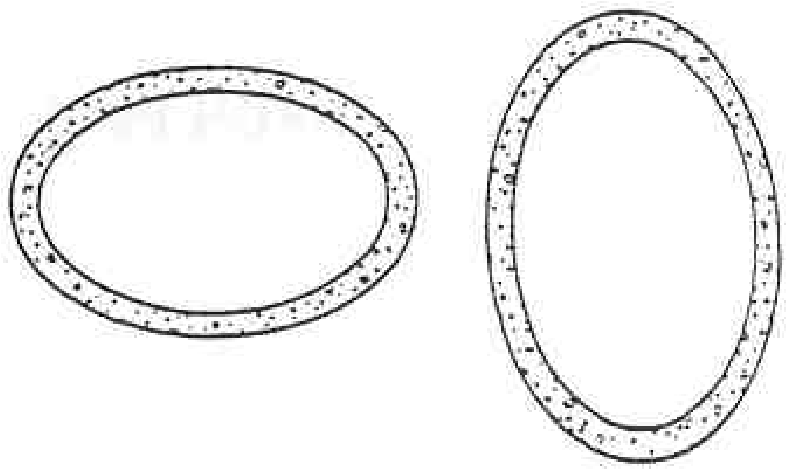
Large Sizes of Pipe Tongue and Groove Joint			
Internal Diameter Inches	**Internal Diameter Feet**	**Wall Thickness Inches**	**Approximate Weight, pounds per foot**
114	9 ½	9 ½	3840
120	10	10	4263
126	10 ½	10 ½	4690
132	11	11	5148
138	11 ½	11 ½	5627
144	12	12	6126
150	12 ½	12 ½	6647
156	13	13	7190
162	13 ½	13 ½	7754
168	14	14	8339
174	14 ½	14 ½	8942
180	15	15	9572

* For description of ASTM C 76 see page 14.2.30

Figure 14.2.25a Standard Sizes for Concrete Pipe (Source: American Concrete Pipe Association)

Typical Cross Section of Arch Pipe

Horizontal and Vertical Ellipse Pipe



Dimensions and Approximate Weights of Elliptical Concrete Pipe

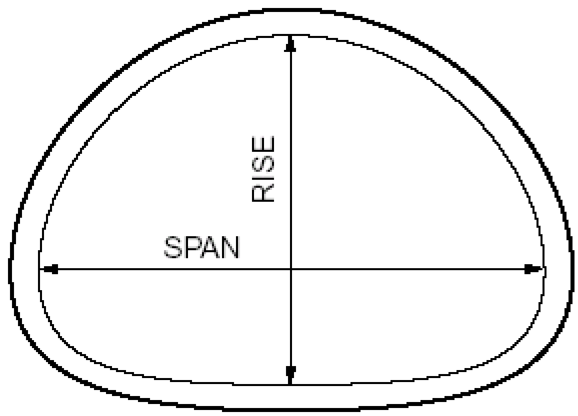
*ASTM C 507 – Reinforced Concrete Elliptical Culvert, Storm Drain and Sewer Pipe

Equivalent Round Size, inches	Minor Axis, inches	Major Axis, inches	Minimum Wall Thickness, inches	Water-Way Area, square feet	Approximate Weight, pounds per foot
96	77	121	9 ½	52.4	3420
102	82	128	9 ¾	59.2	3725
108	87	136	10	66.4	4050
114	92	143	10 ½	74.0	4470
120	97	151	11	82.0	4930
132	106	166	12	99.2	5900
144	116	180	13	118.6	7000

* For description of ASTM C 507 see page 14.2.30

Figure 14.2.25b Standard Sizes for Concrete Pipe (Source: American Concrete Pipe Association)

Typical Cross Section of Arch Pipe



Dimensions and Approximate Weights of Concrete Arch Pipe

*ASTM C 506 – Reinforced Concrete Arch Culvert, Storm Drain and Sewer Pipe					
Equivalent Round Size, inches	**Minimum Rise, inches**	**Minimum Span, inches**	**Minimum Wall Thickness, inches**	**Water-Way Area, square feet**	**Approximate Weight, pounds per foot**
96	77 1/4	122	9	51.7	3110
108	87 1/8	138	10	66.0	3850
120	96 7/8	154	11	81.8	5040
132	106 1/2	168 3/4	10	99.1	5220

* For description of ASTM C 506 see page 14.2.30

Figure 14.2.25c Standard Sizes for Concrete Pipe (Source: American Concrete Pipe Association)

American Society for Testing and Materials (ASTM) Descriptions for Select Rigid Pipe Culverts

ASTM C 76	Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe: Covers reinforced concrete pipe intended to be used for the conveyance of sewage, industrial wastes, and storm waters, and for the construction of culverts. Class I – 60 inches through 144 inches in diameter; Class II, III, IV and V – 12 inches through 144 inches in diameter. Larger sizes and higher classes are available as special designs.
ASTM C 506	Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe: Covers pipe to be used for the conveyance of sewage, industrial waste, and storm water and for the construction of culverts in sizes from 15 inch through 132 inch equivalent circular diameter. Larger sizes are available as special designs.
ASTM C 507	Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe: Covers reinforced elliptically shaped concrete pipe to be used for the conveyance of sewage, industrial waste and storm water, and for the construction of culverts. Five standard classes of horizontal elliptical, 18 inches through 144 inches in equivalent circular diameter and five standard classes of vertical elliptical, 36 inches through 144 inches in equivalent circular diameter are included. Larger sizes are available as special designs.

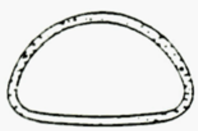
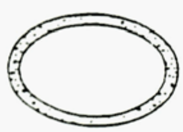
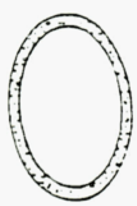
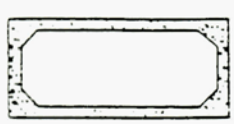
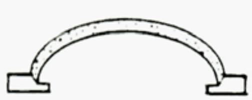
SHAPE	RANGE OF SIZES	COMMON USES
CIRCULAR 	12 to 180 inches reinforced 4 to 36 inches non-reinforced	Culverts, storm drains, and sewers.
PIPE ARCH 	15 to 132 inches equivalent diameter	Culverts, storm drains, and sewers. Used where head is limited.
HORIZONTAL ELLIPSE 	Span x Rise 18 to 144 inches equivalent 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 14.2.26 Standard Concrete Pipe Shapes
(Source: *FHWA Culvert Inspection Manual*, Supplement to the *BIRM*, July 1986)

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Table of Contents

Chapter 14

Characteristics, Inspection and Evaluation of Culverts

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Topic 14.3 Flexible Culverts

14.3.1 Introduction

Like all culverts, flexible culverts are designed for full flow. Unlike bridges, culverts have no distinction between substructure and superstructure and because earth backfill separates the culvert structure from the riding surface, culverts have no "deck." Most flexible culverts have a circular or elliptical configuration (see Figure 14.3.1). Some flexible box and arch culverts are in use today (see Figure 14.3.2). From their design nature, flexible culverts have little structural bending strength without proper backfill. The material from which they are made, such as corrugated steel or aluminum can be flexed or bent and can be distorted significantly without cracking. Consequently, flexible culverts depend on the backfill support to resist bending. In flexible culvert designs, proper interaction between the soil and structure is critical.

Figure 14.3.1 Pipe Arch Flexible Culvert

Figure 14.3.2 Flexible Box Culvert

14.3.2 Design Characteristics

Structural Behavior

A flexible culvert is a composite structure made up of the culvert barrel and the surrounding soil. The barrel and the soil are both vital elements to the structural performance of the culvert.

Flexible pipe has relatively little bending stiffness or bending strength on its own. Flexible culvert materials include steel, aluminum, and plastic. As loads are applied to the culvert, it attempts to deflect. In the case of a round pipe, the vertical diameter decreases and the horizontal diameter increases (see Figure 14.3.3). When good embankment material is well-compacted around the culvert, the increase in horizontal diameter of the culvert is resisted by the lateral soil pressure. With round pipe the result is a relatively uniform radial pressure around the pipe which creates a compressive thrust in the pipe walls. As illustrated in Figure 14.3.4, the compressive thrust is approximately equal to vertical pressure times one-half the span length.

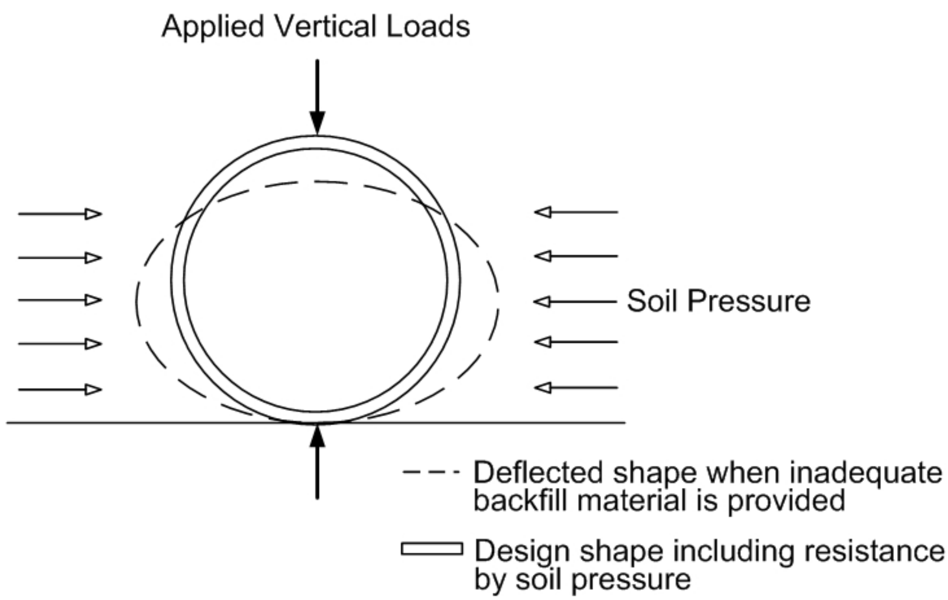


Figure 14.3.3 Flexible Culvert: Load vs. Shape

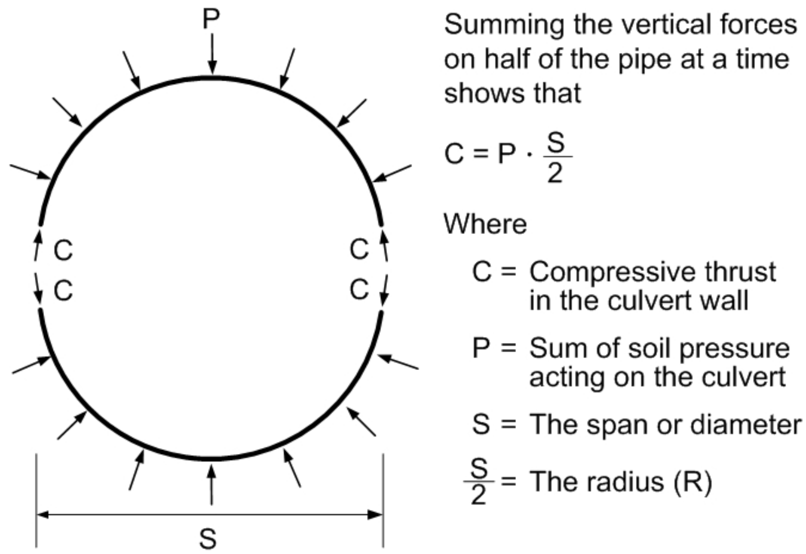


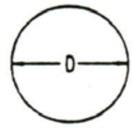
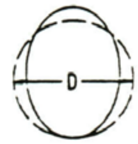
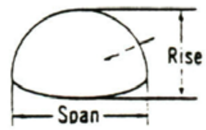
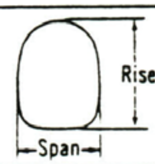
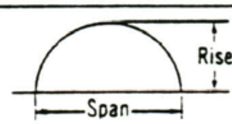
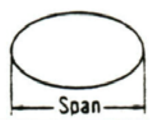
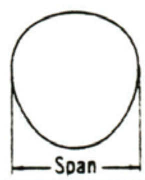
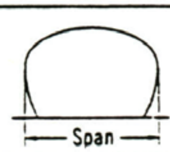
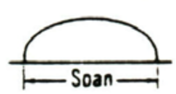
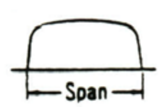
Figure 14.3.4 Formula for Ring Compression

An arc of a flexible round pipe, or other shape will be stable as long as adequate soil pressures are achieved, and as long as the soil pressure is resisted by the compressive force C on each end of the arc. Good quality backfill material and proper installation are critical in obtaining a stable soil envelope around a flexible culvert.

In long span culverts the radius (R) is usually large. To prevent excessive deflection due to permanent dead and/or transient live loads, longitudinal or circumferential stiffeners are sometimes added. The circumferential stiffeners are usually metal ribs bolted to the outside of the culvert. Longitudinal stiffeners may be metal or reinforced concrete. Concrete thrust beams provide some circumferential stiffening as well as longitudinal stiffening. The thrust beams are added to the structure prior to backfill. They also provide a solid vertical surface for soil pressures to act on and a surface which is easier to backfill against. The use of concrete stress relieving slabs is another method used to achieve longer spans or reduce minimum cover. A stress-relieving slab is cast over the top of the backfill above the structure to distribute transient live loads to the adjacent soil.

14.3.3 Types and Shapes of Flexible Culverts

Flexible culverts are constructed from corrugated steel or aluminum pipe or field assembled structural plate products. Structural plate steel products are available as structural plate pipes, box culverts, or long span structures. See Figure 14.3.5 for standard shapes for corrugated flexible culverts.

Shape		Range of Sizes	Common Uses
Round		6 in – 26 ft	Culverts, subdrains, sewers, service tunnels, etc. All plates same radius. For medium and high fills (or trenches).
Vertically-elongated (ellipse) 5% is common		4–21 ft nominal; before elongating	Culverts, sewers, service tunnels, recovery tunnels. Plates of varying radii; shop fabrication. For appearance and where backfill compaction is only moderate.
Pipe-arch		Span x Rise 18 in. x 11 in. to 20 ft 7 in. x 13 ft 2 in.	Where headroom is limited. Has hydraulic advantages at low flows. Corner plate radius, 18 inches or 31 inches for structural plate.
Underpass*		Span x Rise 5 ft 8 in. x 5 ft 9 in. to 20 ft 4 in. x 17 ft 9 in.	For pedestrians, livestock or vehicles (structural plate).
Arch		Span x Rise 6 ft x 1 ft 9½ in. to 25 ft x 12 ft 6 in.	For low clearance large waterway opening, and aesthetics (structural plate).
Horizontal Ellipse		Span 20–40 ft	Culverts, grade separations, storm sewers, tunnels.
Pear		Span 25–30 ft	Grade separations, culverts, storm sewers, tunnels.
High Profile Arch		Span 20–45 ft	Culverts, grade separations, storm sewers, tunnels, Ammo ammunition magazines, earth covered storage.
Low Profile Arch		Span 20–50 ft	Low-Wide waterway enclosures, culverts, storm sewers.
Box Culverts		Span 10–21 ft	Low-wide waterway enclosures, culverts, storm sewers.
Specials		Various	For lining old structures or other special purposes. Special fabrication.

*For equal area or clearance, the round shape is generally more economical and simpler to assemble.

Figure 14.3.5 (Exhibit 11 Culvert Inspection Manual Report No. FHWA-IP-86-2) Standard Corrugated Steel Culvert Shapes (Source: Handbook of Steel Drainage and Highway Construction Products, American Iron and Steel Institute)

Corrugated Pipe

Factory-made pipe is produced in two basic shapes: round and pipe arch. Both shapes are produced in several wall thicknesses, several corrugation sizes, and with annular (circumferential) or helical (spiral) corrugations. Pipes with helical corrugations have continuously welded seams or lock seams. Both round and arch steel pipe shapes are available in a wide range of standard sizes.

Structural Plate

Structural plate steel pipes are field assembled from standard corrugated galvanized steel plates. Standard plates have corrugations with a 6-inch pitch and a depth of 2 inches. Plates are manufactured in a variety of thicknesses and are pre-curved for the size and shape of structure to be erected.

Structural steel plate pipes are available in four basic shapes:

- Round
- Pipe arch
- Arch
- Underpass

Structural plate aluminum pipes are field assembled with a 9 inch pitch and a depth of 2.5 inches.

Structural plate aluminum pipes are produced in five basic shapes:

- Round
- Pipe arch
- Arch
- Pedestrian/animal underpass
- Vehicle underpass

Long Span Culverts

Long span steel culverts are assembled using conventional 6 by 2 inch corrugated galvanized steel plates and longitudinal and circumferential stiffening members. There are five standard shapes for long span steel structures:

- Horizontal elliptical
- Pipe arch
- Low profile arch
- High profile arch
- Pear shape

Each long span installation represents, to a certain extent, a custom design. The inspector reviews the design or as-built plans when checking dimensions of existing long span structures.

Long span aluminum structures are assembled using conventional 9 by 2 1/2 inch corrugated aluminum plates and aluminum rib stiffeners. Long span aluminum structures are essentially the same size and available in the same five basic shapes as steel long spans.

See the end of this Topic for the different standard sizes for each flexible culvert shape (pg 164-193 Culvert Inspection Manual Report No. FHWA-IP-86-2)

Box Culverts

Corrugated steel box sections use standard corrugated galvanized steel plates with special reinforcing elements applied to the areas of maximum moments. Steel box culverts are available with spans that range from 10 feet to 21 feet.

The aluminum box culvert utilizes standard aluminum structural plates with aluminum rib reinforcing added in the areas of maximum bending stresses. Ribs are bolted to the exterior of the aluminum shell during installation. Aluminum box culverts are suitable for shallow depths of fill.

Plastic Culverts

Plastic culverts are most commonly made using high density polyethylene (HDPE). These round sections utilize one or more "walls" and are available up to 60 inches in diameter. Single-walled culverts are often corrugated on the inner and outer surfaces (see Figure 14.3.6), while dual-walled culverts have a smooth inner surface and either a smooth or corrugated outer surface (see Figure 14.3.7). Heavy-duty plastic culverts are also available in sizes up to 36 inches.

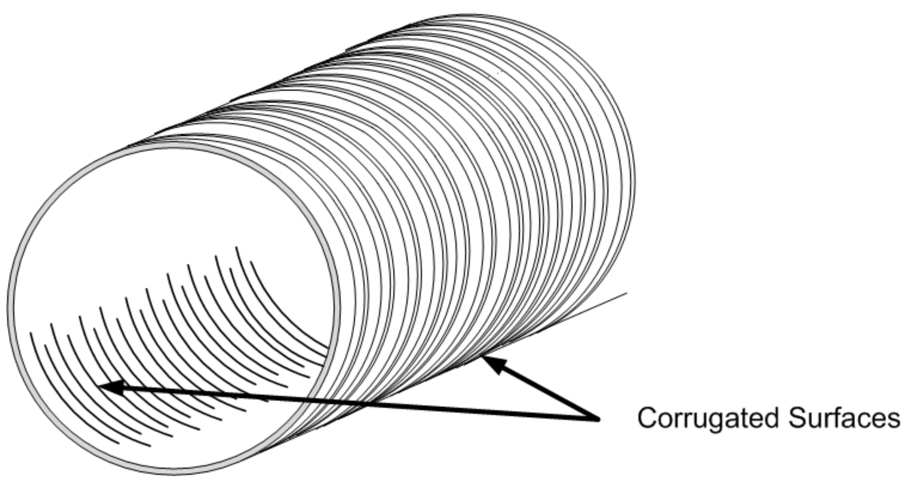


Figure 14.3.6 Schematic of a Single Walled Culvert

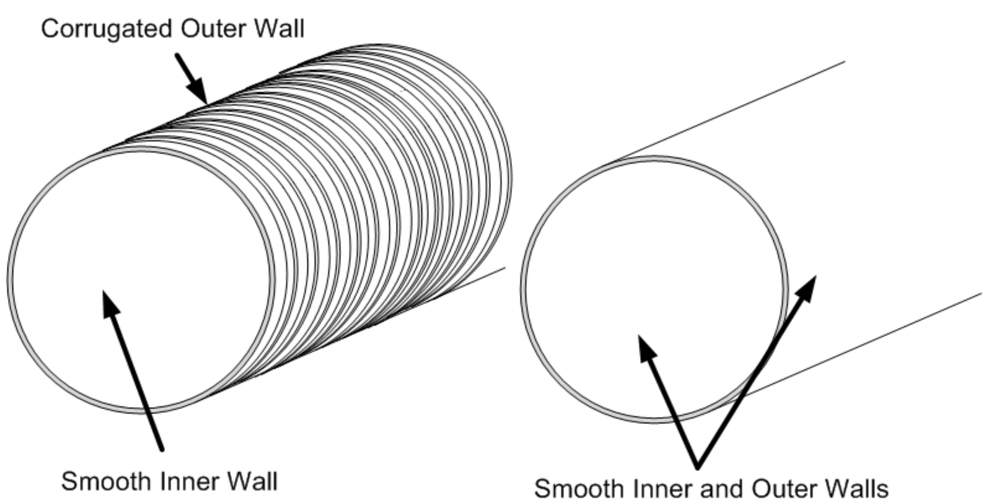


Figure 14.3.7 Schematics of Dual Walled Culverts

Plastic culverts offer several advantages over traditional corrugated metal pipe (CMP) sections:

- Strength-to-weight ratio - the favorable strength-to-weight ratio allows plastic culverts to provide maximum strength and shock resistance from a lighter section, making HDPE sections competitive against CMP sections installed with higher clear cover distances.
- Lightweight - the weight savings compared to CMP or concrete pipe allows HDPE culverts to be installed using minimum manpower and light-duty equipment. Lightweight pipe also provides a safer work environment when compared to heavy weight pipes.
- Hydraulically efficient - compared to CMP, the smoothness of HDPE culverts (for applicable dual-wall culverts) provides increased hydraulic efficiency. This permits a smaller HDPE section to be used for an equally performing larger CMP section.
- Corrosion resistance - unlike CMP culverts, plastic culverts will not "rust". They also have shown good performance against corrosive chemicals, brackish water, and soil elements. Abrasion resistance is also greater for plastic culverts than for CMP culverts. However, plastic culverts are susceptible to low crack growth and oxygen degradation.
- Flexibility - due to the inherent nature of HDPE and other plastic resins, plastic culverts offer increased flexibility over CMP. Although less common in the roadway industry, this allows for easier placement of a curved pipeline.

Applications utilizing plastic culverts include:

- New culvert structures with adequate clear cover above the culvert.
- Rehabilitation of older culverts using a "slip lining" installation method.
- Temporary culvert installations or highway drainage systems.

14.3.4 Overview of Common Deficiencies

Common deficiencies that can occur to flexible culvert materials include the following:

- Pitting
- Surface Rust
- Section Loss
- Overload Damage
- Heat Damage
- Buckling
- Embankment erosion at culvert entrance and exit
- Roadway settlement
- Irregular dimensions
- Loose or missing seams and fasteners

Refer to Topic 6.3 for a more detailed presentation of the properties of steel, types and causes of steel deterioration, and the examination of steel.

14.3.5 Inspection Methods and Locations

Refer to Topic 14.1 for a more detailed presentation of methods and locations of culvert distress.

A logical sequence for inspecting culverts helps ensure that a thorough and complete inspection will be conducted. In addition to the culvert components, 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. Select one end of the culvert and inspect the embankment, waterway, headwalls, wingwalls, and culvert barrel. Progress toward the other end of the culvert. The following sequence is applicable to all culvert inspections:

- Review available information
- Observe overall condition
- Inspect approach roadway and embankment
- Inspect waterway (see in Topic 13.2)
- Inspect end treatments
- Inspect culvert barrel

Methods

Visual

Most defects in flexible culverts are first detected by visual inspection. In order for this to occur, a hands-on inspection, or inspection where the inspector is close enough to touch the area being inspected, is required. The types of defects to look for when inspecting the culvert barrel will depend upon the type of culvert being inspected. In general, inspect corrugated metal culvert barrels for cross-sectional shape and barrel defects such as joint defects (exhiltration or infiltration through joints or joint misalignment), seam defects (exhiltration or infiltration through seams or seam misalignment), plate buckling, lateral shifting, missing or loose bolts, corrosion, excessive abrasion, material deficiencies, and localized construction damage. A critical area for the inspection of long span metal culverts is at the 2 o'clock and 10 o'clock locations. An inward bulge at these locations may indicate potential failure of the structure.

It is becoming more common that flexible culverts are being repaired or rehabilitated with structural plate sections or with structural invert paving by using reinforced concrete. Inspect the concrete for deficiencies such as surface cracks, spalls, wear, and other deficiencies is primarily a visual activity. Structural plates can be visually inspected for deficiencies to those discussed previously for steel.

Physical

A geologist's pick hammer can be used to scrape off heavy deposits of rust and scale and to check the longitudinal seams by tapping the nuts. The hammer can then be used to locate areas of corrosion by striking the culvert walls. The walls will deform or the hammer will break through the culvert wall if significant section loss exists.

For aluminum structural plate, the bolts are checked with a torque wrench.

Sometimes surveying the culvert is necessary to determine if there is any shape distortion, and if there is distortion how much exists.

It is important to check the repairs for deficiencies as well. For concrete repairs, be sure to check for delaminations by using a hammer to “sound” the concrete. 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.

For the structural plates, inspect for section loss. This is achieved by using a wire brush, grinder, or a hammer to remove loose or flaked steel and then measure the remaining section and compare to a similar section with no loss.

It may be necessary to get a permit to work in culverts due to the confined spaces which have the potential for hazardous conditions for the inspector.

Advanced Inspection Methods

In metal culverts, visual inspections can only point out surface defects. Therefore, advanced inspection methods may be used to achieve a more rigorous and thorough inspection of the flexible culvert, including:

Several advanced methods are available for steel inspection. Nondestructive methods, described in Topic 15.3.2, include:

- Computer programs
- Corrosion sensors
- Dye penetrant
- Magnetic particle

Other inspection methods or tests for material properties, described in Topic 15.3.3, include:

- Brinell hardness test
- Charpy impact test
- Chemical analysis
- Tensile strength test

Locations

End Treatments

End treatments are inspected like any other structural component. Their effectiveness can directly affect the performance of the culvert.

The most common types of end treatments for flexible culverts are:

- Projections
- Mitered
- Pipe end section

Projections

Indicate the location and extent of any scour or undermining around the culvert ends. The depth of any scouring is measured with a probing rod. In low flow conditions scour holes have a tendency to fill up with debris or sediment. If no probing rod is used, the scour could be mistakenly reported as less than has taken place.

Inspect end treatments for evidence of water leaking around the end treatment and into the embankment. Water flowing along the outside of a culvert can remove supporting material. This is referred to as piping and it can lead to the culvert end being unsupported. If not repaired in time, piping can cause cantilevered end portions of the culvert to bend down and restrict the stream flow.

Mitered Ends

Inspection items for mitered ends are the same as for projecting ends. Take additional care to measure any deformation of the end. Mitering the end of corrugated pipe culvert reduces its structural capacity.

Pipe End Sections

Pipe end sections are typically used on relatively smaller culverts. For inspection purposes, treat the pipe end section similar to a projection.

Excerpts from a reproduction of the out-of-print Culvert Inspection Manual Report No.-IP-86-2 are located on page 14.3.12 of this topic.

14.3.6 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of flexible culverts. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using NBI component condition rating guidelines, a one-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. Component condition rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 (Item 62) for additional details about NBI component condition rating guidelines. The component condition rating code is intended to be an overall evaluation of the culvert. Integral wingwalls to the first construction or expansion joint shall be included in the evaluation. It is also important to note that Items 58-Deck, 59-

Superstructure, and 60-Substructure shall be coded "N" for all culverts.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating.

Element Level Condition State Assessment

In an element level condition state assessment of a flexible culvert, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Substructure	
240	Steel Culvert
243	Other Culvert

BME No.	Description
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating

The unit quantity for culverts is feet and represents the culvert length along the barrel multiplied by the number of barrels (for multiple barrel culverts). The inspector visually evaluates each 1-foot slice of the culvert barrel(s) and assigns the appropriate condition state description. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coatings is square feet, with the total area distributed among the four condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition state 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of flexible culverts:

Defect Flag No.	Description
356	Steel Cracking/Fatigue
357	Pack Rust
360	Settlement
361	Scour
363	Steel Section Loss
368	Culvert Barrel Distortion

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

The following excerpts are from a reproduction of the Culvert Inspection Manual Report No.-IP-86-2 – Chapter 5, Section 4 which can be found at the following website: http://www.fhwa.dot.gov/

Section 4 - CORRUGATED METAL CULVERTS

5-4.0 General

Corrugated aluminum and corrugated steel culverts are classified as flexible structures because they respond to and depend upon the soil backfill to provide structural stability and support to the culvert. The flexible corrugated metal acts essentially as a liner. The liner acts mainly in compression and can carry large ring compression thrust, but very little bending or moment force. (Rib reinforced box culverts are exceptions.) Inspection of the culvert determines whether the soil envelope provides adequate structural stability for the culvert and verifies that the "liner" is capable of carrying the compressive forces and protecting the soil backfill from water flowing through the culvert. Verification of the stability of the soil envelope is accomplished by checking culvert shape. Verification of the integrity of the "liner" is accomplished by checking for pipe and plate culvert barrel defects.

This section contains discussions on inspecting corrugated metal structures for shape and barrel defects. Because shape inspection requirements do vary somewhat for different shapes, separate sections with detailed guidelines are provided for corrugated metal pipe culvert shapes and long-span culvert shapes. Section 5 of this chapter addresses corrugated metal pipe culverts, and section 6 covers long-span corrugated metal culverts.

5-4.1 Shape Inspections

The single most important feature to observe and measure when inspecting corrugated metal culverts is the cross-sectional shape of the culvert barrel. The corrugated metal culvert barrel depends on the backfill or embankment to maintain its proper shape and stability. When the backfill does not provide the required support, the culvert will deflect, settle, or distort. Shape changes in the culvert therefore provide a direct indication of the adequacy and stability of the supporting soil envelope. By periodic observation and measurement of the culvert's shape, it is possible to verify the adequacy of the backfill. The design or theoretical cross-section of the culvert should be the standard against which field measurements and visual observations are compared. If the design cross section is unknown, a comparison can be made between the unloaded culvert ends and the loaded sections beneath the roadway or deep fills. This can often provide an indication of structure deflection or settlement. Symmetrical shape and uniform curvature around the perimeter are generally the critical factors. If the curvature around the structure becomes too flat, and/or the soil continues to yield under load, the culvert wall may not be able to carry the ring thrust without either buckling inward or deflecting excessively to the point of reverse curvature. Either of these events leads to partial or total failure.

As explained earlier in this Topic, an arc of a circular pipe or other shape structure will be stable and perform as long as the soil pressure on the outside of the pipe is resisted by the compression force in the pipe at each end of the arc.

Corrugated metal pipes can change shape safely within reasonable limits as long as there is adequate exterior soil pressure to balance the ring compression. Therefore, size and shape measurements taken at any one time do not provide conclusive data on backfill instability even when there is significant deviation from the design shape. Current backfill stability cannot be reliably determined unless changes in shape are measured over time. It is therefore necessary to identify current or recent shape changes to reliably check backfill stability. If there is instability of the backfill, the pipe will continue to change shape.

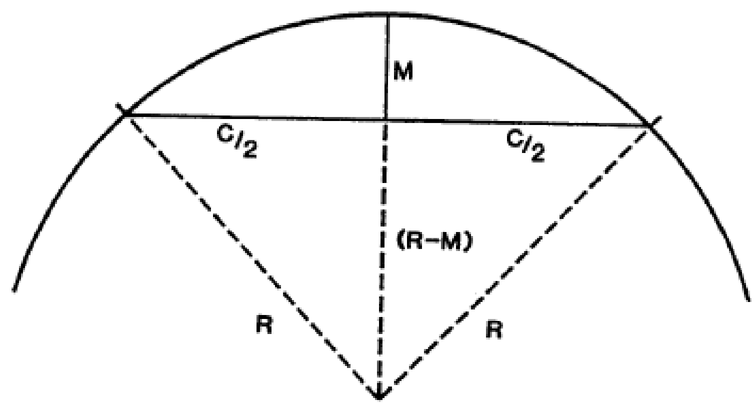
In general, the inspection process for checking shape will include visual observations for symmetrical shape and uniform curvature as well as measurements of important dimensions. The specific measurements to be obtained depend upon factors such as the size, shape, and condition of the structure. If shape changes are observed, more measurements may be necessary. For small structures in good condition, one or two simple measurements may be sufficient, for example, measuring the horizontal diameter on round pipe. For larger structures such as long span culverts, key measurements may be difficult to obtain. Horizontal diameters may be both high and large. The inspection process for long span culverts generally requires that elevations be established for key points on the structure. Although some direct measurements may also be required for long-span structures, elevations are needed to check for settlement and for calculating vertical distances such as the middle ordinate of the top arc. For structures with shallow cover, observations of the culvert with a few live loads passing over are recommended. Discernible movement in the structure may indicate possible instability and a need for more in-depth investigation.

The number of measurement locations depends upon the size and condition of the structure. Long-span culverts should normally be measured at the end and at 25 foot intervals. Measurements may be required at more frequent intervals if significant shape changes are observed. The smaller pipe culverts can usually be measured at longer intervals than long-span culverts.

Locations in sectional pipe can be referenced by using pipe joints as stations to establish the stationing of specific cross-sections. Stations should start with number 1 at the outlet and increase going upstream to the inlet. The location of points on a circular cross section can be referenced like hours on a clock. The clock should be oriented looking upstream. On structural plate corrugated metal culverts, points can be referenced to bolted circumferential and longitudinal seams.

It is extremely important to tie down exact locations of measurement points. Unless the same point is checked on each inspection, changes cannot be accurately monitored. The inspection report must, therefore, include precise descriptions of reference point locations. It is safest to use the joints, seams, and plates as the reference grid for measurement points. Exact point locations can then be easily described in the report as well as physically marked on the structures. This guards against loss of paint or scribe marks and makes points easy to find or reestablish. All dimensions in structures should be measured to the inside crest of corrugation. When possible, measurement points on structural plate should be located at the center of a longitudinal seam. However, some measurement points are not on a seam.

When distortion or curve flattening is apparent, the extent of the flattened area, in terms of arc length, length of culvert affected, and the location of the flattened area should be described in the inspection report. The length of the chord across the flattened area and the middle ordinate of the chord should be measured and recorded. The chord and middle ordinate measurements can be used to calculate the curvature of the flattened area using the formula shown in Exhibit 66.



C = MEASURED CHORD

M = MEASURED MIDDLE ORDINATE

SOLVE FOR R_A = ACTUAL RADIUS

$$R_A = \frac{4M^2 + C^2}{8M}$$

IF R_A IS > R_D (DESIGN RADIUS) THEN ACTUAL CURVE IS FLATTER THAN DESIGN

Figure 14.3.8 (Exhibit 66) Checking Curvature by Curve and Middle Ordinate

5-4.2 Inspecting Barrel Defects

The structural integrity of corrugated metal culverts and long-span structures is dependent upon their ability to perform in ring compression and their interaction with the surrounding soil envelope. Defects in the culvert barrel itself, which can influence the culvert's structural and hydraulic performance, are discussed in the following paragraphs. Rating guidelines are provided in the sections dealing with specific shapes.

a. Misalignment - The inspector should check the vertical and horizontal alignment of the culvert. The vertical alignment should be checked visually for sags and deflection at joints. Poor vertical alignment may indicate problems with the subgrade beneath the pipe bedding. Sags trap debris and sediment and may impede flow. Since most highway culverts do not have watertight joints, sags which pocket water could saturate the soil beneath and around the culvert, reducing the soil's stability. The horizontal alignment should be checked by sighting along the sides for straightness. Vertical alignment can be checked by sighting along bolt lines. Minor horizontal and vertical misalignment is generally not a significant problem in corrugated metal structures unless it causes shape or joint problems. Occasionally culverts are intentionally installed with a change in gradient.

b. Joint Defects - Field joints are generally only found with factory manufactured pipe. There are ordinarily no joints in structural plate culverts, only seams. (In a few cases, preassembled lengths of structural plate pipe have been coupled or banded together like factory pipe.)

   Field joints in factory pipe serve to maintain the water conveyance of the culvert from section to section, to keep the pipe sections in alignment, keep the backfill soil from infiltrating, and to help prevent sections from pulling apart. Joint separation may indicate a lack of slope stability as described in section 5-4.2 e., circumferential seams. Key factors to look for in the inspection of joints are indications of backfill infiltration and water exfiltration. Excessive seepage through an open joint can cause soil infiltration or erosion of the surrounding backfill material reducing lateral support. Open joints may be probed with a small rod or flat rule to check for voids. Indications of joint defects include open joints, deflection, seepage at the joints, and surface sinkholes over the culvert as illustrated in Exhibits 67 and 68. Any evidence of joint defects should be recorded. Culverts in good condition should have no open joints, those in fair condition may have a few open joints but no evidence of soil infiltration, and those in marginal to poor condition will show evidence of soil infiltration.

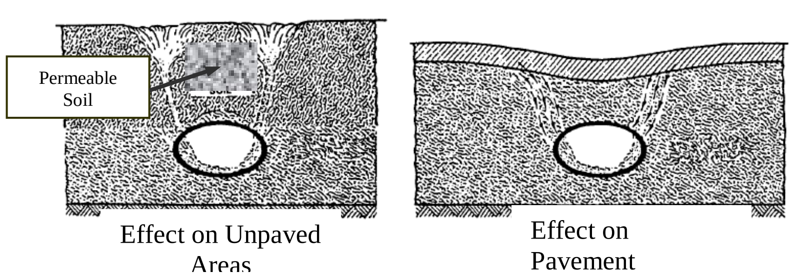


Figure 14.3.9 (Exhibit 67) Surface Indications of Infiltration

Figure 14.3.10 (Exhibit 68) Surface Hole Above Open Joint

c. Seam Defects in Fabricated Pipe - Pipe seams in helical pipe do not carry a significant amount of the ring compression thrust in the pipe. That is the reason that a lock seam is an acceptable seam. Helical seams should be inspected for cracking and separation. An open seam could result in a loss of backfill into the pipe, or exfiltration of water. Either condition could reduce the stability of the surrounding soil.

   In riveted or spot welded pipes, the seams are longitudinal and carry the full ring compression in the pipe. These seams, then, must be sound and capable of handling high compression forces. They should be inspected for the same types of defects as those described in the text for structural plate culverts, Section 12.4.3, Structural Pipe. When inspecting the longitudinal seams of bituminous-coated corrugated metal culverts, cracking in the bituminous coating may indicate seam separation.

d. Longitudinal Seam Defects in Structural Plate Culverts - Longitudinal seams should be visually inspected for open seams, cracking at bolt holes, plate distortion around the bolts, bolt tipping, cocked seams, cusped seams, and for significant metal loss in the fasteners due to corrosion.

   Culverts in good condition should have only minor joint defects. Those in fair condition may have minor cracking at a few bolt holes or minor opening at seams that could lead to infiltration or exfiltration. Marginal to poor culvert barrel conditions are indicated by significant cracking at bolt holes, or deflection of the structure due to infiltration of backfill through an open seam. Cracks 3 inches long on each side of the bolts indicate very poor to critical conditions.

(1) Loose Fasteners - Seams should be checked for loose or missing fasteners as shown in Exhibit 69. For steel structures the longitudinal seams are bolted together with high-strength bolts in two rows; one row in the crests and one row in the valleys of the corrugations. These are bearing type connections and are not dependent on a minimum clamping force of bolt tension to develop interface friction between the plates. Fasteners in steel structural plate may be checked for tightness by tapping lightly with a hammer and checking for movement.

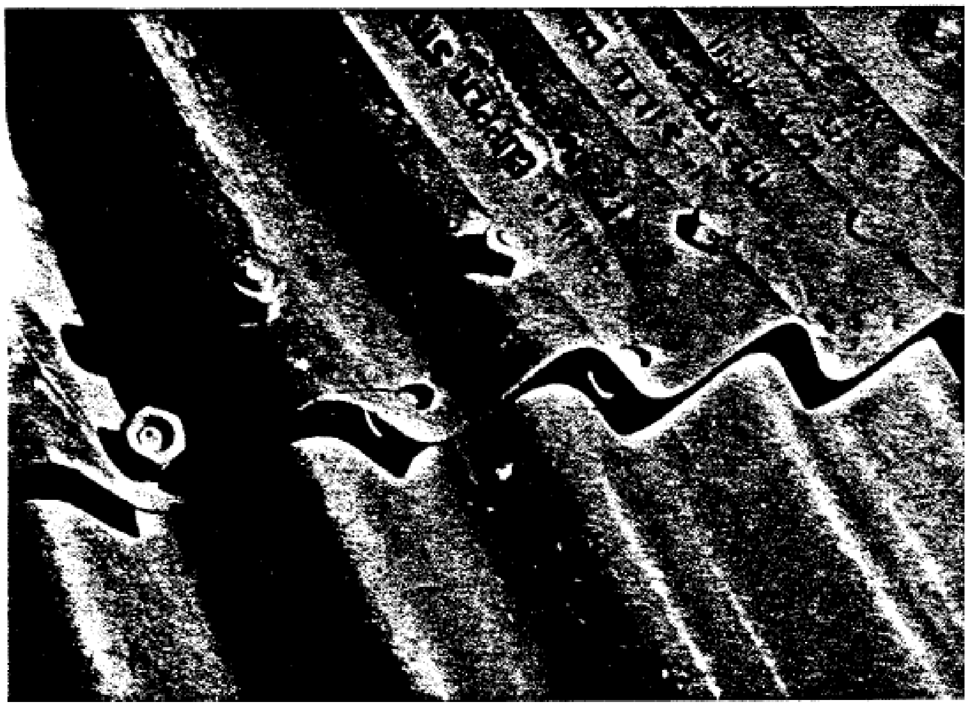


Figure 14.3.11 (Exhibit 69) Close-Up of Loose and Missing Bolts at a Cusped Seam; Loose Fasteners are Usually Detected by Tapping the Nuts with a Hammer

For aluminum structural plate, the longitudinal seams are bolted together with normal strength bolts in two rows with bolts in the crests and valleys of both rows. These seams function as bearing connections, utilizing bearing of the bolts on the edges of holes and friction between the plates. The seams in aluminum structural plate should be checked with a torque wrench (125 ft-lbs minimum to 150 ft-lbs maximum). If a torque wrench is not available fasteners can be checked for tightness with a hammer as described for steel plates.

(2) Cocked and Cusped Seams - The longitudinal seams of structural plate are the principal difference from factory pipe. The shape and curvature of the structure is affected by the lapped, bolted longitudinal seam. Improper erection or fabrication can result in cocked seams or cusped effects in the structure at the seam, as illustrated in Exhibit 70. Slight cases of these conditions are fairly common and frequently not significant. However, severe cases can result in failure of the seam or structure. When a cusped seam is significant the structure's shape appearance and key dimensions will differ significantly from the design shape and dimensions.

The cusp effect should cause the structure to receive very low ratings on the shape inspection if it is a serious problem. A cocked seam can result in loss of backfill and may reduce the ultimate ring compression strength of the seam.

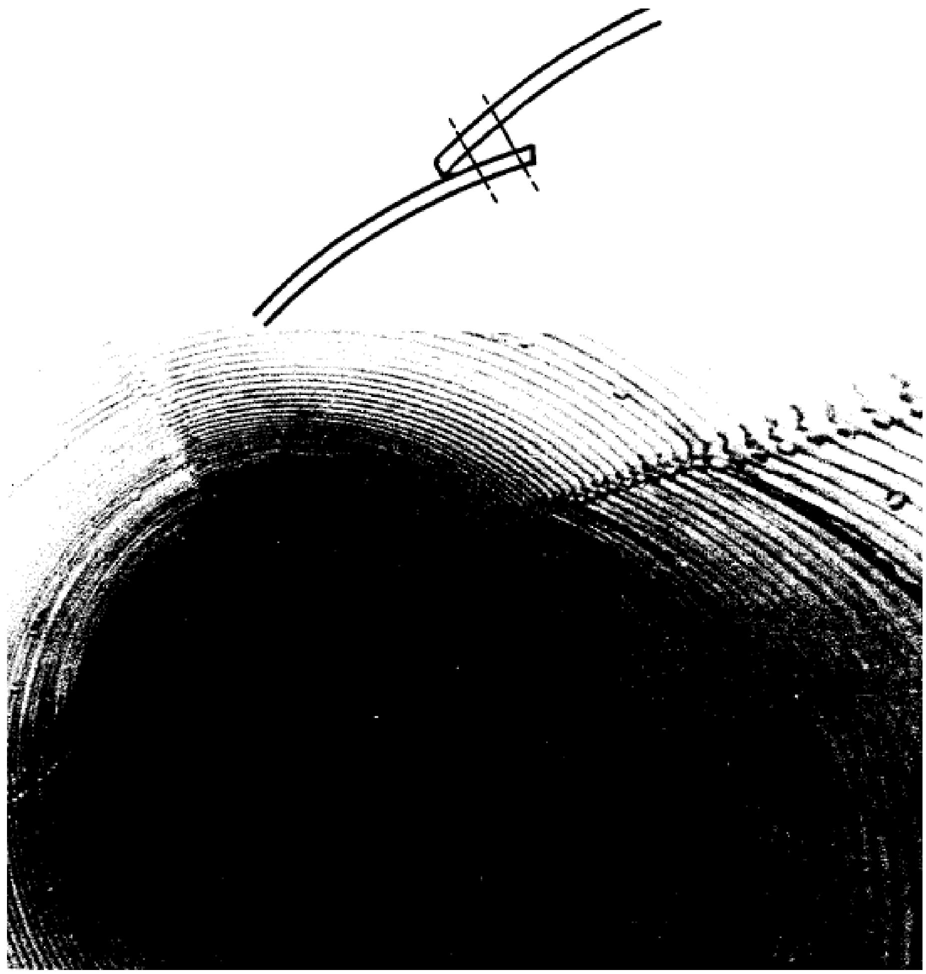


Figure 14.3.12 (Exhibit 70) Cocked Seam with Cusp Effect

(3) Seam Cracking - Cracking along the bolt holes of longitudinal seams can be serious if allowed to progress. As cracking progresses, the plate may be completely severed and the ring compression capability of the seam lost. This could result in deformation or possible failure of the structure. Longitudinal cracks are most serious when accompanied by significant deflection, distortion, and other conditions indicative of backfill or soil problems. Longitudinal cracks are caused by excessive bending strain, usually the result of deflection, Exhibit 71. Cracking may occasionally be caused by improper erection practices such as using bolting force to "lay down" a badly cocked seam.

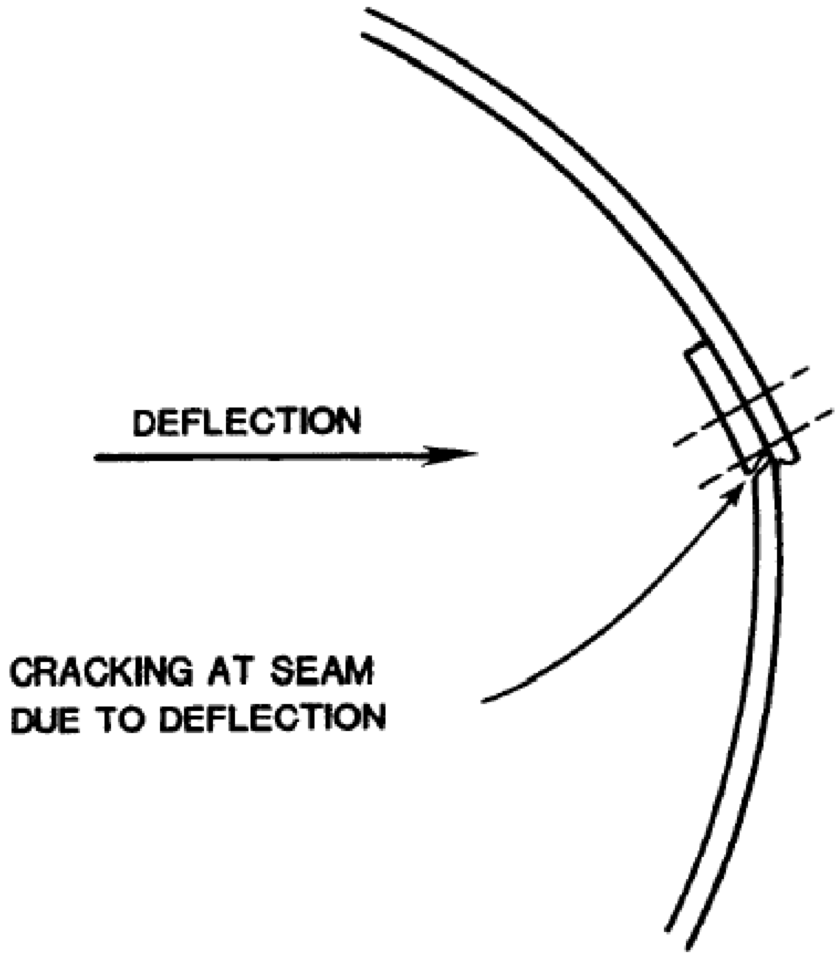


Figure 14.3.13 (Exhibit 71) Cracking Due to Deflection

(4) Bolt Tipping - The bolted seams in structural plate culverts only develop their ultimate strength under compression. Bolt tipping occurs when the plates slip. As the plates begin to slip, the bolts tip, and the bolt holes are plastically elongated by the bolt shank. High compressive stress is required to cause bolt tipping. Structures have rarely been designed with loads high enough to produce a ring compression that will cause bolt tip. However, seams should be examined for bolt tip particularly in structures under higher fills. Excessive compression on a seam could result in plate deformations around the tipped bolts and failure is reached when the bolts are eventually pulled through the plates.

e. Circumferential Seams - The circumferential seams, like joints in factory pipe, do not carry ring compression. They do make the conduit one continuous structure. Distress in these seams is rare and will ordinarily be a result of a severe differential deflection or distortion problem or some other manifestation of soil failure. For example, a steep sloping structure through an embankment may be pulled apart longitudinally if the embankment moves down as shown in Exhibit 72. Plates should be installed with the upstream plate overlapping the downstream plate to provide a "shingle" effect in the direction of flow.

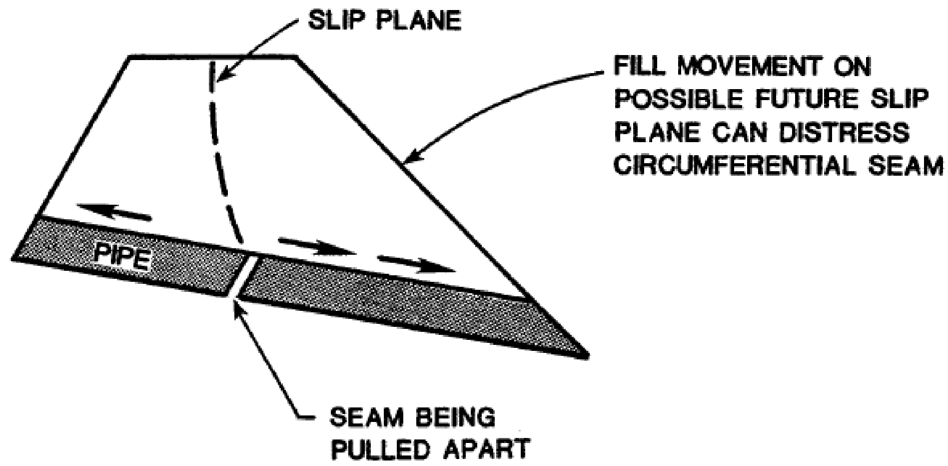


Figure 14.3.14 (Exhibit 72) Circumferential Seam Failure Due to Embankment Slippage

The circumferential seam at one or more locations would be distressed by the movement of the fill. Such distress is important to note during inspections since it would indicate a basic problem of stability in the fill. Circumferential seam distress can also be a result of foundation failure, but in such cases should be clearly evident by the vertical alignment.

f. Dents and Localized Damage - All corrugated metal culverts should be inspected for localized damage. Pipe wall damage such as dents, bulges, creases, cracks, and tears can be serious if the defects are extensive and can impair either the integrity of the barrel in ring compression or permit infiltration of backfill. Small, localized examples are not ordinarily critical. When the deformation type damages are critical, they will usually result in a poorly shaped cross section. The inspector should document the type, extent, and location of all significant wall damage defects. When examining dents in corrugated steel culverts, the opposite side of the plate should be checked, if possible, for cracking or disbonding of the protective coating.

g. Durability (Wall Deterioration) - Durability refers to the ability of a material to resist corrosion and abrasion. Corrosion is the deterioration of metal due to electrochemical or chemical reactions. Abrasion is the wearing away of culvert materials by the erosive action of bedload carried in the stream.

Abrasion is generally most serious in steep or mountainous areas where high flow rates carry sand and rocks that wear away the culvert invert. Abrasion can also accelerate corrosion by wearing away protective coatings.

Metal culverts are subject to corrosion in certain aggressive environments. For example, steel rapidly corrodes in salt water and in environments with highly acidic (low pH) conditions in the soil and water. Aluminum is fairly resistant to salt water but will corrode rapidly in highly alkaline (high pH) environments, particularly if metals such as iron or copper and their salts are present. The electrical resistivity of soil and water also provide an indication of the likelihood of corrosion. Many agencies have established guidelines in terms of pH and resistivity that are based on local performance. The FHWA has also published guidelines for aluminum and steel culverts including various protective coatings.

Corrosion and abrasion of corrugated metal culverts can be a serious problem with adverse effects on structural performance. Damage due to corrosion and abrasion is the most common cause for culvert replacement. The inspection should include visual observations of metal corrosion and abrasion. As steel corrodes it expands considerably. Relatively shallow corrosion can produce thick deposits of scale. A geologist's pick-hammer can be used to scrape off heavy deposits of rust and scale permitting better observation of the metal. A hammer can also be used to locate unsound areas of exterior corrosion by striking the culvert wall with the pick end of the hammer. When severe corrosion is present, the pick will deform the wall or break through it. Protective coatings should be examined for abrasion damage, tearing, cracking, and removal. The inspector should document the extent and location of surface deterioration problems.

When heavy corrosion is found by observation or sounding, special inspection methods such as pH testing, electrical resistivity measurement, and obtaining cores from the pipe wall are recommended. A routine program for testing pH and electrical resistivity should be considered since it is relatively easy to perform and provides valuable information.

Durability problems are the most common cause for the replacement of pipe culverts. The condition of the metal in corrugated metal culverts and any coatings, if used, should be considered when assigning a rating to the culvert barrel. Suggested rating guidelines for metal culverts with metallic coatings are shown in Exhibit 73. Modification of these guidelines may be required when inspecting culverts with non-metallic coatings. Aluminum culvert barrels may be rated as being in good condition if there is superficial corrosion. Steel culverts rated as in good condition may have superficial rust with no pitting. Perforation of the invert as shown in Exhibit 74 would indicate poor condition. Complete deterioration of the invert in all or part of the culvert barrel would indicate a critical condition as shown in Exhibit 75. Culverts with deteriorated inverts may function as an arch structurally, but are highly susceptible to failure due to erosion of the bedding.

Rating Value	General Description	Corrugated Steel	Corrugated Aluminum
9	New	Near original condition	Near original condition
8	Good	Superficial rust, no pitting	Superficial corrosion slight pitting
7	Generally Good	Moderate rust, slight pitting	Moderate corrosion no attack of core alloy
6	Fair	Fairly heavy rust, moderate pitting, slight thinning	Significant corrosion minor attack of core alloy
5	Generally Fair	Extensive heavy rust, deep pitting, moderate thinning	Significant corrosion moderate attack of core alloy
4	Marginal	Pronounced thinning (some deflection or penetration when struck with pick hammer)	Extensive corrosion significant attack of core alloy
3	Poor	Extensive heavy rust, deep pitting scattered perforations	Extensive corrosion attack of core alloy scattered perforations
2	Critical	Extensive perforations due to rust	Extensive perforations due to corrosion
1	Critical	Invert completely deteriorated	Invert completely deteriorated
0	Critical	Partial or complete collapse	Partial or complete collapse

Figure 14.3.15 (Exhibit 73) Suggested Rating Criteria for Condition of Corrugated Metal

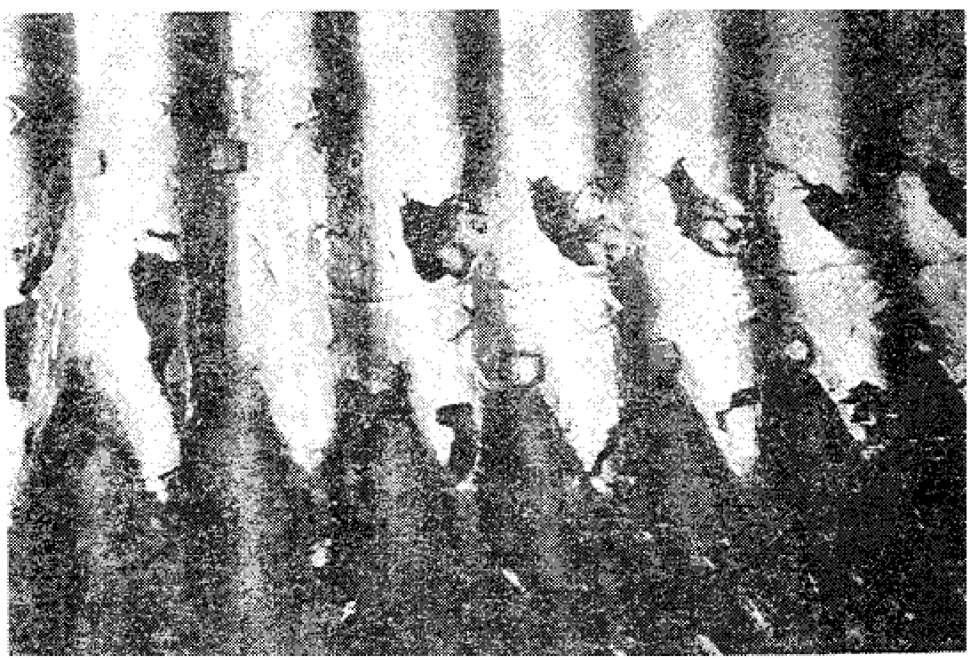


Figure 14.3.16 (Exhibit 74) Perforation of the Invert Due to Corrosion

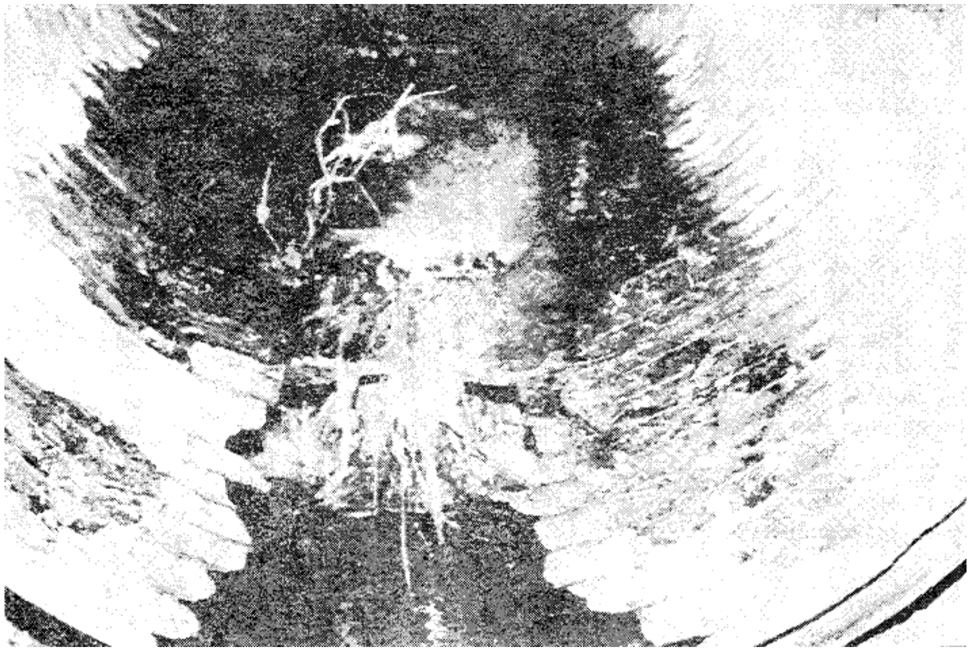


Figure 14.3.17 (Exhibit 75) Invert Deterioration

h. Concrete Footing Defects - Structural plate arches, long-span arches, and box culverts use concrete footings. Metal footings are occasionally used for the arch and box culvert shapes. The metal "superstructure" is dependent upon the footing to transmit the vertical load into the foundation. The structural plate arch is usually bolted in a base channel which is secured in the footing.

The most probable structural defect in the footing is differential settlement. One section of a footing settling more than the rest of the footing can cause wrinkling or other distortion in the arch. Flexible corrugated metal culverts can tolerate some differential settlement but will be damaged by excessive differential settlement. Uniform settlement will not ordinarily affect a metal arch but can affect the clearances in a grade separation structure if the footings settle and the road does not. The significance of differential footing settlement increases as the amount of the difference in settlement increases, the length it is spread over decreases, and the height of the arch decreases. This concept is illustrated in Exhibit 76.

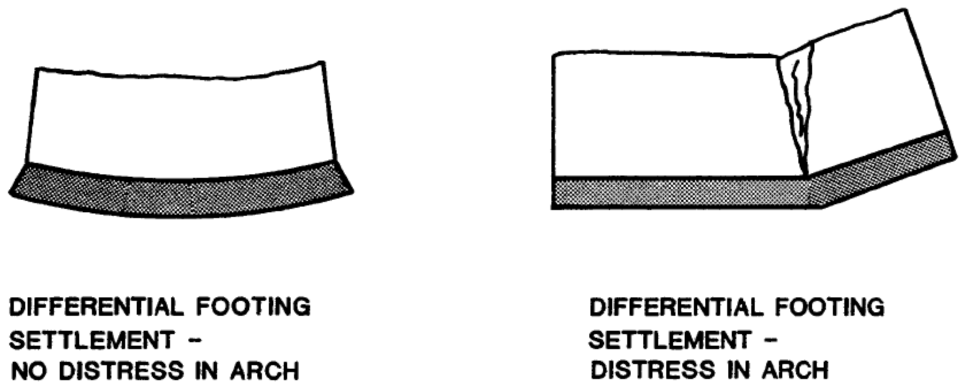


Figure 14.3.18 (Exhibit 76) Differential Footing Settlement

The inspection of footings in structural plate and long-span arches should include a check for differential settlement along the length of a footing. This might show up in severe cracking, spalling, or crushing across the footing at the critical spot. If severe enough, it might be evidenced by compression or stretching of the corrugations in the culvert barrel. Deterioration may occur in concrete and masonry footings which is not related to settlement but is caused by the concrete or mortar. In arches with no invert slab, the inspector should check for erosion and undermining of the footings and look for any indication of rotation of the footing as illustrated in Exhibits 77 and 78.

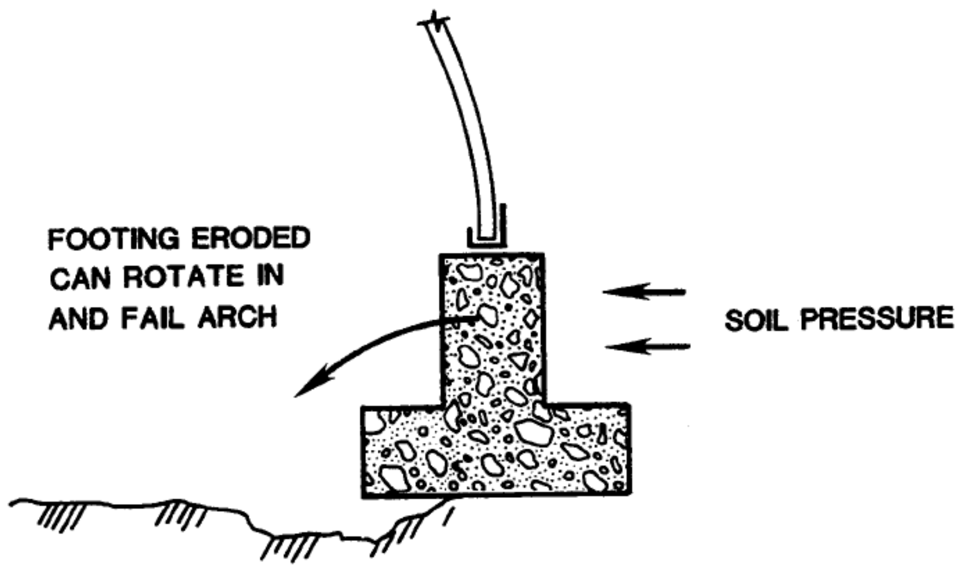


Figure 14.3.19 (Exhibit 77) Footing Rotation due to Undermining

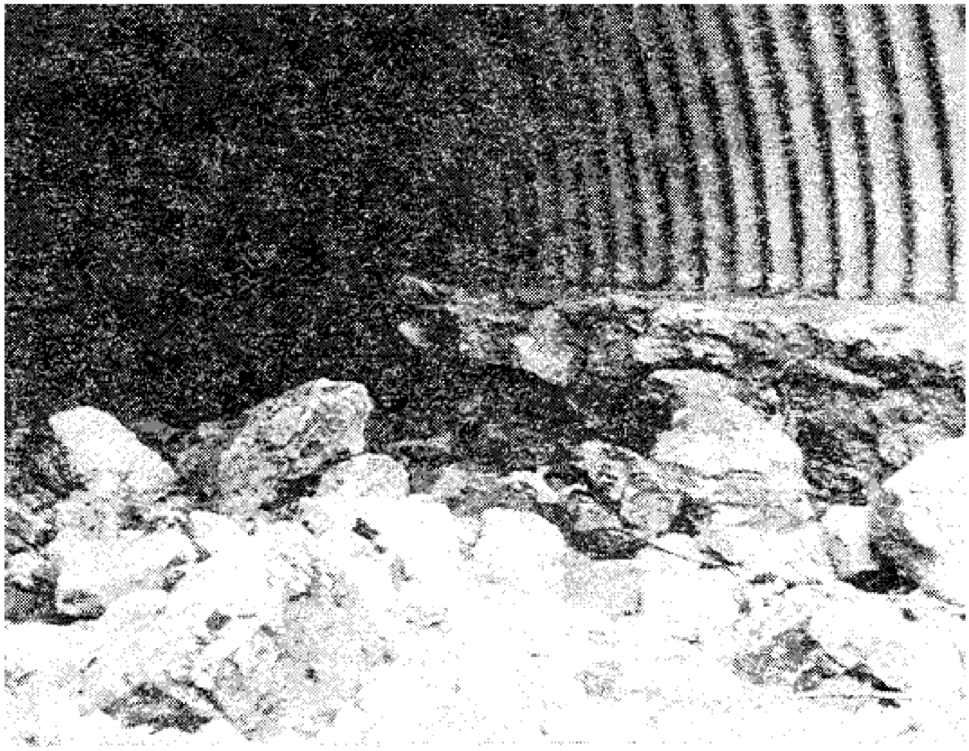


Figure 14.3.20(Exhibit 78) Erosion of Invert Undermining footing of Arch

Culverts rated in good condition may have minor footing damage. Poor to critical condition would be indicated by severe footing undermining, damage, or rotation, or by differential settlement causing distortion and circumferential kinking in the corrugated metal as shown in Exhibit 79.

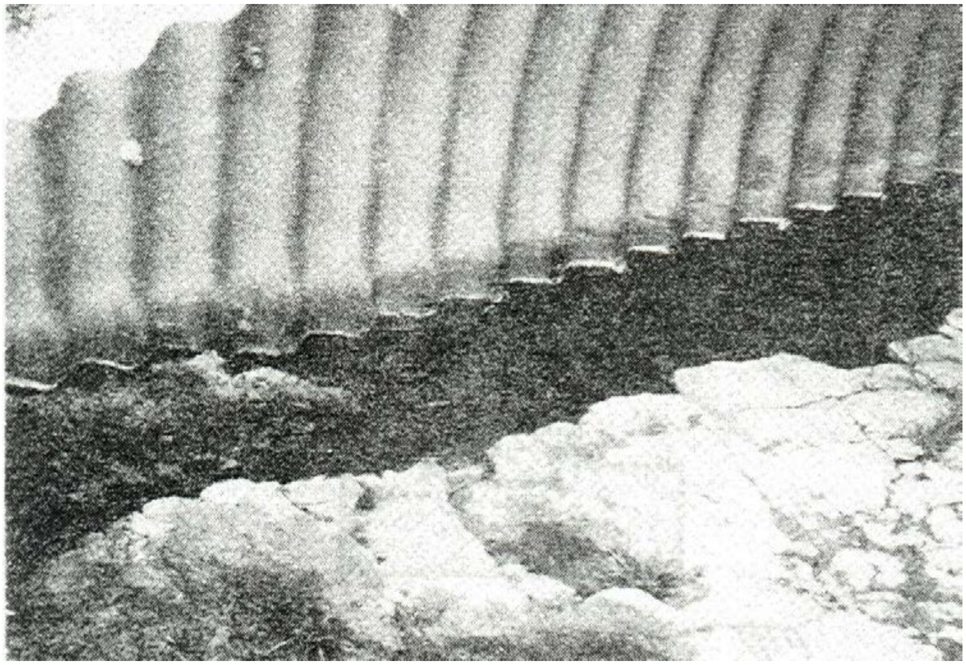


Figure 14.3.21 (Exhibit 79) Erosion Damage to Concrete Invert

i. Defects in Concrete Inverts - Concrete inverts in arches are usually floating slabs used to carry water or traffic. Invert slabs provide protection against erosion and undercutting, and are also used to improve hydraulic efficiency. Concrete inverts are sometimes used in circular, as well as other culvert shapes, to protect the metal from severe abrasive or severe corrosive action. Concrete invert slabs in arches should be checked for undermining and damage such as spalls, open cracks, and missing portions. The significance of damage will depend upon its effect on the footings and corrugated metal.

The following excerpts are from a reproduction of the out-of-print Culvert Inspection Manual (Supplement to Manual 70), July 1986 – Chapter 5, Section 5.

Section 5 - SHAPE INSPECTION OF CORRUGATED METAL CULVERT BARRELS

5-5.0 General

This section deals with shape inspections of common culvert shapes including round and vertical elongated, pipe arches, arches, and box culvert shapes. Specific guidelines for recommended measurements to be taken for each location are provided for each typical culvert shape. Additional measurements are also recommended when field measurements differ from the design dimensions or when significant shape changes are observed. Rating guidelines are also provided for each shape. The guidelines include condition descriptions with shape and barrel defects defined for each rating.

5-5.1 Using the Rating Guidelines

When using the rating guidelines, the inspector should keep the following factors in mind:

a. The inspector should select the lowest rating which best describes either the shape condition or the barrel condition. Structure shape is the most critical factor in flexible culverts, and this should be kept in mind when selecting the rating.

b. The shape criteria described for each numerical rating should be considered as a group rather than as separate criteria for each measurement check listed. Good curvature and the rate of change are critical. Significant changes in shape since the last inspection should be carefully evaluated even if the structure is still in fairly good condition.

c. The guidelines merely offer a starting point for the inspector. The inspector must still use judgment in assigning the appropriate numerical rating. The numerical rating should be related to the actions required. The inspector may wish to refer to Section 4.2 of this manual.

5-5.2 Round and Vertical Elongated Pipe

Round and vertically elongated pipes are expected to deflect vertically during construction resulting in a slightly increased horizontal span. Round pipes are sometimes vertically elongated five percent to compensate for settlement during construction. It is frequently difficult to determine in the field if a pipe was round or elongated when installed. Large round pipes may appear to be elongated if they were subjected to minor flattening of the sides during backfill.

Vehicular underpasses sometimes use 10 percent vertically elongated very large pipe which is susceptible to side flattening during installation. In shallow cover situations, adequate curvature in the sides is the important factor. The soil pressures on the sides may be greater than the weight of the shallow fill over the pipe. The result is a tendency to push the sides inward rather than outward as in deeper buried or round pipes. Side flattening, such as that shown in Exhibit 80, can be caused by unstable backfill. A deteriorated invert may have contributed to the problem by reducing the pipe's ability to transmit compressive forces.

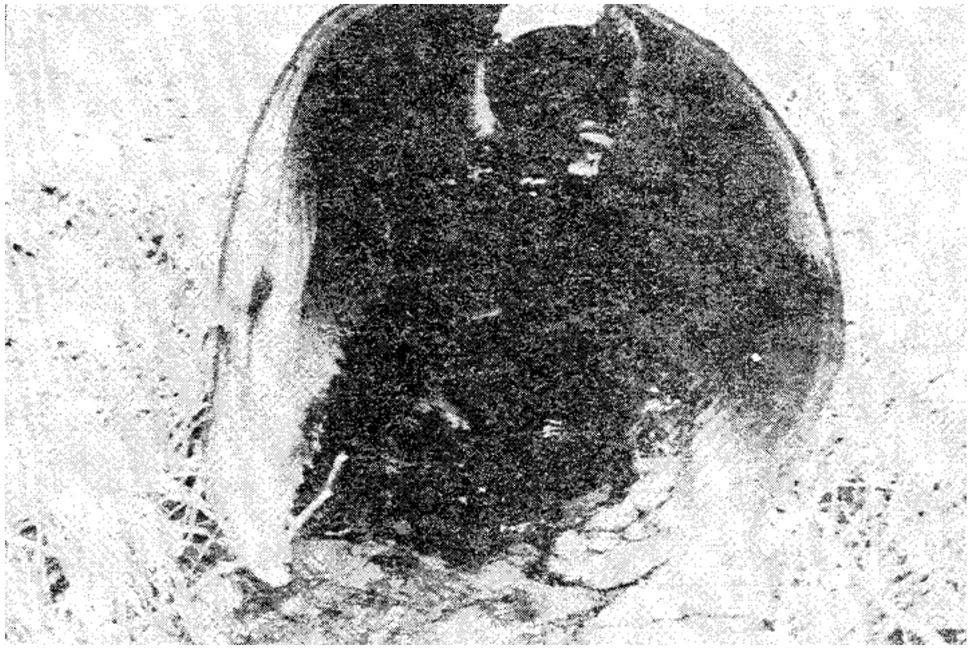
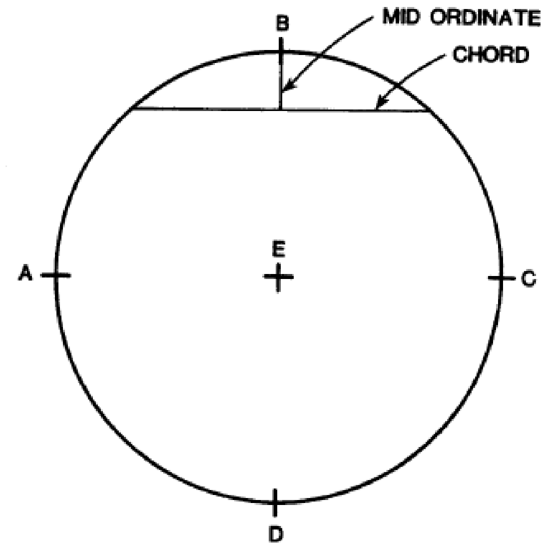


Figure 14.3.22 (Exhibit 80) Excessive Side Deflection

Flattening of the top arc is an indication of possible distress. Flattening of the invert is not as serious. Pipes not installed on shaped bedding will often exhibit minor flattening of the invert arc. However, severe flattening of the bottom arc would indicate possible distress.

The inspector should note the visual appearance of the culvert's shape and measure the horizontal span as shown in Exhibit 81. Almost all round or vertical elongated pipe can be directly measured and will not require elevations. Exceptions are large vertical elongated grade separation structures. On such structures, elevations should be obtained similar to those recommended for the long-span pear shape.



1. MINIMUM MEASUREMENTS REQUIRED:
   - HORIZONTAL DIAMETER = AC
2. IF FLATTENING OBSERVED MEASURE:
   - CHORD AND MID ORDINATE OF FLATTENED AREA
3. IF HORIZONTAL DIAMETER EXCEEDS DESIGN BY MORE THAN 10% MEASURE:
   - VERTICAL DIAMETER = BD

Figure 14.3.23 (Exhibit 81) Shape Inspection Circular and Vertical Elongated Pipe

If the visual appearance or measured horizontal diameter differs significantly from the design specifications, additional measurement, such as vertical diameter, should be taken. Flattened areas should be checked by measuring a chord and the mid ordinate of the chord. The chord length and ordinate measurement should be noted in the report with a description of the location and extent of the flattened area.

Round and vertically elongated pipe with good to fair shape will have a generally good shape appearance. Good shape appearance means that the culvert's shape appears to match the design shape, with smooth, symmetrical curvature and no visible deformations. The horizontal span should be within 10 percent of the design span. Pipe with marginal shape will be indicated by characteristics such as a fair or marginal general shape appearance, distortion in the upper half of the pipe, severe flattening in the lower half of the pipe, or horizontal spans 10 to 15 percent greater than design.

Pipe with poor to critical shape will have a poor shape appearance that does not match the design shape, does not have smooth or symmetrical curvature, and may have obvious deformations. Severe distortion in the upper half of the pipe, a horizontal diameter more than 15 percent to 20 percent greater than the design diameter, or flattening of the crown to an arc with a radius of 20 to 30 feet or more would indicate poor to critical condition. It should be noted that pipes with deflection of less than 15 to 20 percent may be rated as critical based on poor shape appearance. Guidelines for rating round corrugated metal culvert are presented in Exhibit 82.

RATING GUIDELINES FOR ROUND OR VERTICAL ELONGATED CORRUGATED METAL PIPE BARRELS

RATING	CONDITION
9	• New condition
8	• Shape: good, smooth curvature in barrel - Horizontal: within 10 percent of design • Seams and Joints: tight, no openings • Metal: - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting
7	• Shape: generally good, top half of pipe smooth but minor flattening of bottom - Horizontal Diameter: within 10 percent of design • Seams or Joints: minor cracking at a few bolt holes, minor joint or seam openings, potential for backfill infiltration • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting
6	• Shape: fair, top half has smooth curvature but bottom half has flattened significantly - Horizontal Diameter: within 10 percent of design • Seams or Joints: minor cracking at bolts is prevalent in one seam in lower half of pipe. Evidence of backfill infiltration through seams or joints • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting
5	• Shape: generally fair, significant distortion at isolated locations in top half and extreme flattening of invert - Horizontal Diameter: 10 percent to 15 percent greater than design • Seams or Joints: moderate cracking at bolt holes along one seam near bottom of pipe, deflection of pipe caused by backfill infiltration through seams or joints • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting
4	• Shape: marginal significant distortion throughout length of pipe, lower third may be kinked - Horizontal Diameter: 10 percent to 15 percent greater than design • Seams or Joints: Moderate cracking at bolt holes on one seam near top of pipe, deflection caused by loss of backfill through open joints • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting
3	• Shape: poor with extreme deflection at isolated locations, flattening of crown, crown radius 20 to 30 feet - Horizontal Diameter: in excess if 15 percent greater than design • Seams: 3 in. long cracks at bolt holes on one seam • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations
2	• Shape: critical, extreme distortion and deflection throughout pipe, flattening of crown, crown radius over 30 feet - Horizontal Diameter: More than 20 percent greater than design • Seams: plate cracked from bolt to bolt on one seam • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust
1	• Shape: partially collapsed with crown in reverse curve • Seams: failed • Road: closed to traffic
0	• Pipe: totally failed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.24 (Exhibit 82) Condition Rating Guidelines

5-5.3 Pipe Arch

The pipe arch is a completely closed structure but is essentially an arch. The load is transmitted to the foundation principally at the corners. The corners are much like footings of an arch. There is relatively little force or pressure on the large radius bottom plate. The principal type of distress in a pipe arch is a result of inadequate soil support at the corners where the pressure is relatively high. The corner may push down or out into the sail while the bottom stays in place. The effect will appear as if the bottom pushed up. This problem is illustrated in Exhibits 83 and 84.

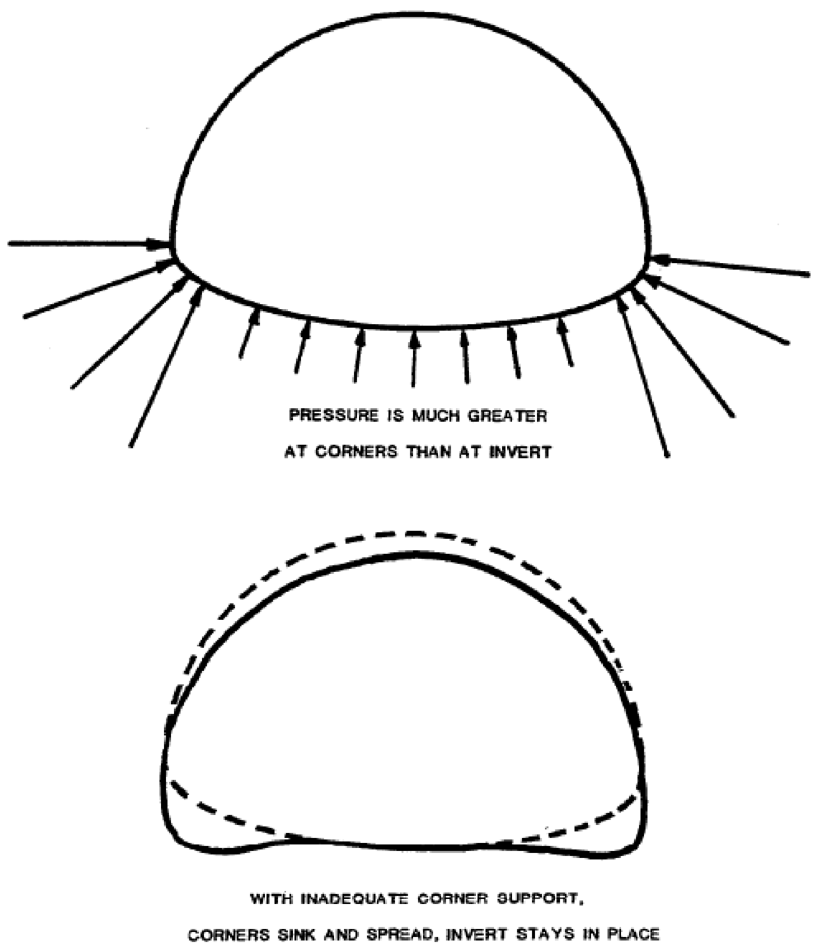


Figure 14.3.25 (Exhibit 83) Bottom Distortion in Pipe Arches

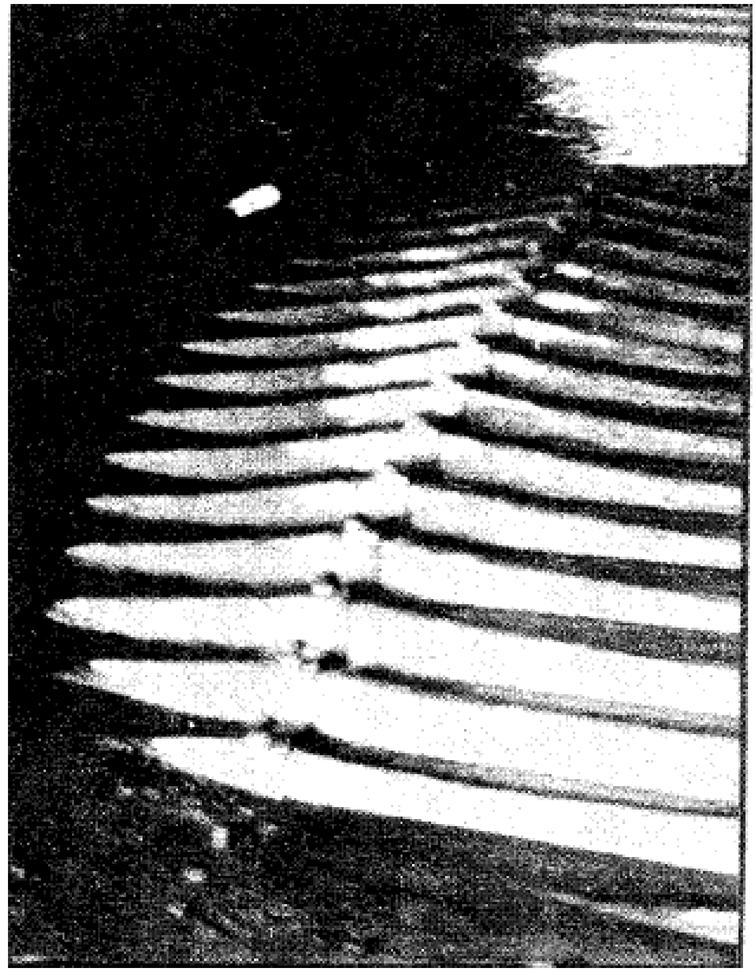


Figure 14.3.26 (Exhibit 84) Bottom and Corners of this Pipe Arch have Settled

The bottom arc should be inspected for signs of flattening and the bottom corners for signs of spreading. The extent and location of bottom flattening and corner spreading should be noted in the inspection report.

Complete reversal of the bottom arc can occur without failure if corner movement into the foundation has stabilized. The top arc of the structure is supporting the load above and its curvature is an important factor. However, if the "footing" corner should fail, the top arc would also fail. The spreading of the corners is therefore very important as it affects the curvature of the top arc.

The inspector should record the visual appearance of the shape and measure both the span and the rise. If the span exceeds the design span by more than 3 percent, the span of the top arc, the mid ordinate of the top arc, and the mid ordinate of the bottom arc should also be measured. Recommended measurements are shown in Exhibit 85.

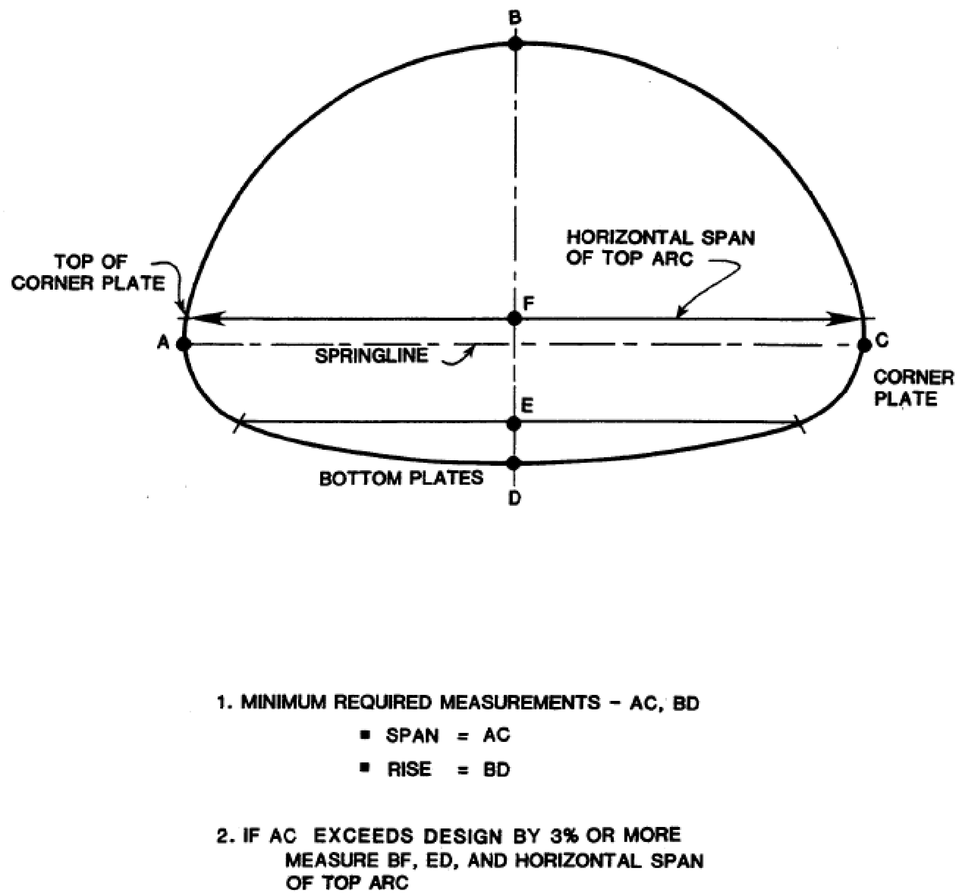


Figure 14.3.27 (Exhibit 85) Shape Inspection Structural Plate Pipe Arch

Pipe arches in fair to good condition will have a symmetrical appearance, smooth curvature in the top of the pipe, and a span less than five percent greater than theoretical. The bottom may be flattened but should still have curvature. Pipe arches in marginal condition will have fair to marginal shape appearance, with distortion in the top half of the pipe, slight reverse curvature in the bottom of the pipe, and a horizontal span five to seven percent greater than theoretical. Pipe in poor to critical condition will have characteristics such as a poor shape appearance, severe deflection or distortion in the top half of the pipe, severe reverse curvature in the bottom of the pipe, flattening of one side, flattening of the crown to an arc with a radius of 20 to 30 feet, or a horizontal span more than seven percent greater than theoretical. Guidelines for rating pipe arches are shown in Exhibit 86.

RATING GUIDELINES FOR CORRUGATED METAL PIPE-ARCH BARRELS

RATING	CONDITION	RATING	CONDITION
9	• New condition	4	• Shape: marginal, significant distortion all along top of arch, bottom has reverse curve - Horizontal Span: more than 7 percent greater than design • Joints and Seams: moderate cracking all along one seam; backfill infiltration causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting
8	• Shape: good with smooth curvature - Horizontal Span: less than 3 percent greater than design • Joints or Seams: good condition • Metal: minor construction defects, protective coatings intact - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting	3	• Shape: poor, extreme deflection in top arch in one section; bottom has reverse curvature throughout - Horizontal Span: more than 7 percent greater than design • Seams: seam cracked 3 in. on each side of bolt holes • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations
7	• Shape: generally good, smooth curvature in top half, bottom flattened but still curved - Horizontal Span: within 3 to 5 percent greater than design • Joints or Seams: minor cracking at a few bolt holes; minor joint or seam openings, infiltration of backfill possible • Metal: protective coating ineffective - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting	2	• Shape: critical, extreme deflection along top of pipe - Horizontal Span: more than 7 percent greater than design • Seams: seam cracked from bolt to bolt down one seam • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust
6	• Shape: fair, smooth curvature in top half, bottom flat - Horizontal Span: 5 percent greater than design • Joints or Seams: minor cracking all along one seam; minor joint openings with evidence of infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting	1	• Shape: structure partially collapsed • Seams: seam failed • Road: closed to traffic
5	• Shape: generally fair, significant distortion in top in one location; bottom has slight reverse curvature in one location - Horizontal Span: within 5 to 7 percent greater than design • Joints and Seams: moderate cracking at bolt holes along a seam in one section, backfill being lost through seam or joint causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting	0	• Shape: structure collapsed • Road: closed to traffic

NOTES:
1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.28 (Exhibit 86) Condition Rating Guidelines

5-5.4 Arches.

Arches are fixed on concrete footings, usually below or at the springline. The springline is a line connecting the outermost points on the sides of a culvert. This difference between pipes and arches means that an arch tends to deflect differently during backfill. Backfill forces tend to flatten the arch sides and peak its top because the springline cannot move inward like the wall of a round pipe as shown in Exhibit 87. As a result, important shape factors to look for in an arch are flattened sides, peaked crown, and flattened top arc.

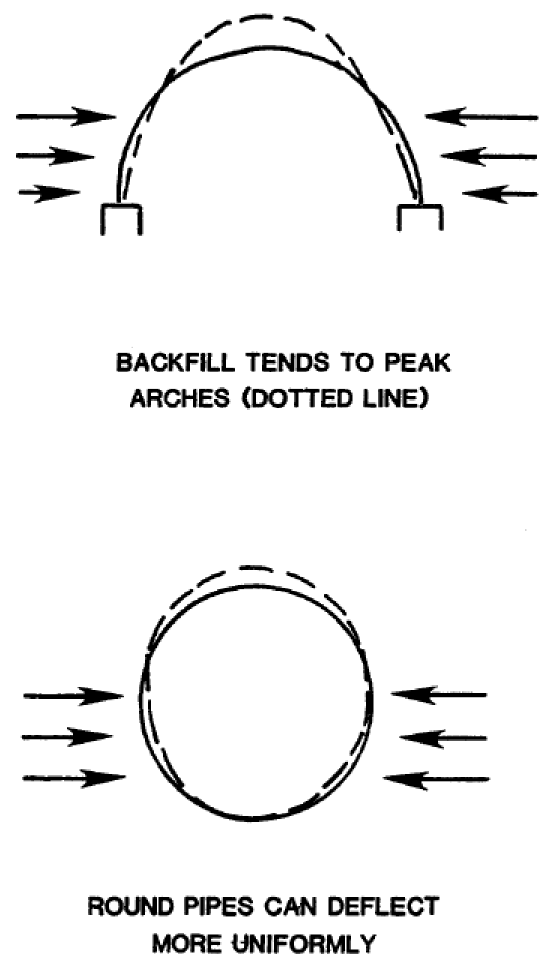


Figure 14.3.29 (Exhibit 87) Arch Deflection During Installation

Another important shape factor in arches is symmetrical shape. If the arch was erected with the base channels not square to the centerline, it causes a racking of the cross section. A racked cross-section is one that is not symmetrical about the centerline of the culvert. One side tends to flatten while the other side tends to curve more while the crown moves laterally and possibly upward. If these distortions are not corrected before backfilling the arch, they usually get worse during backfill. Exhibit 88 illustrates racked or peaked arches.

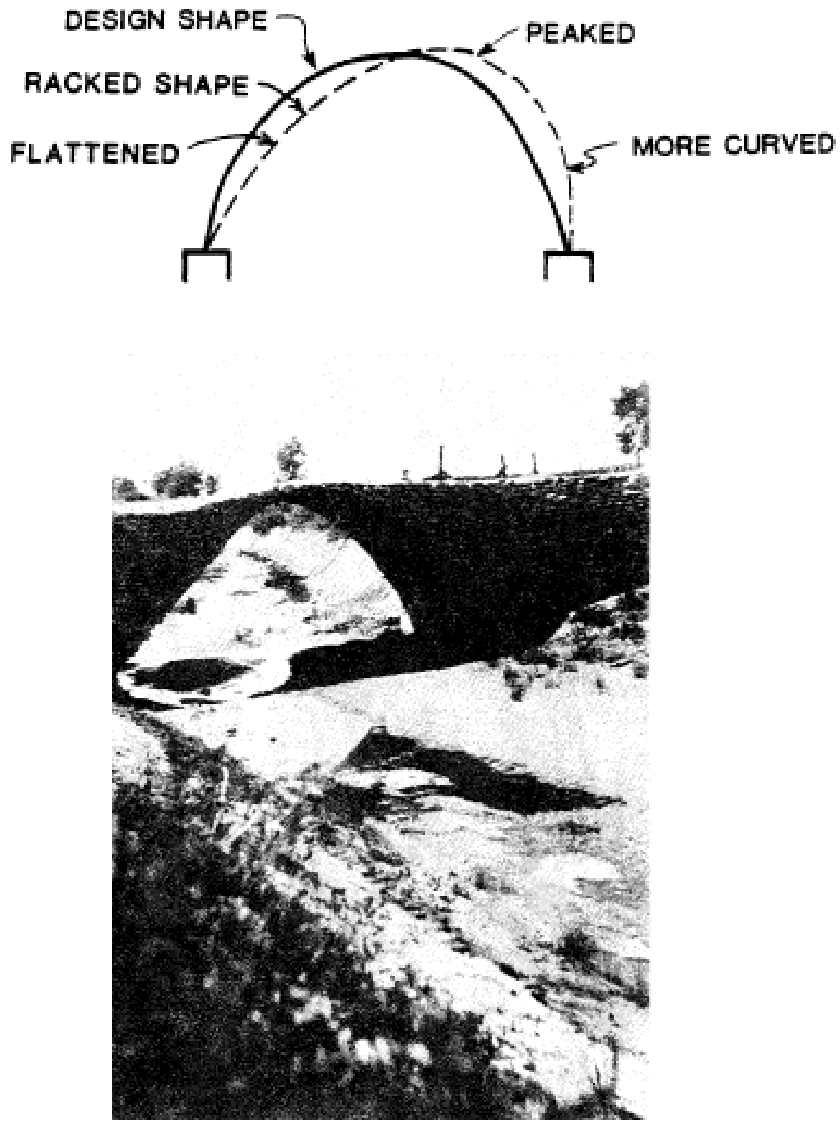
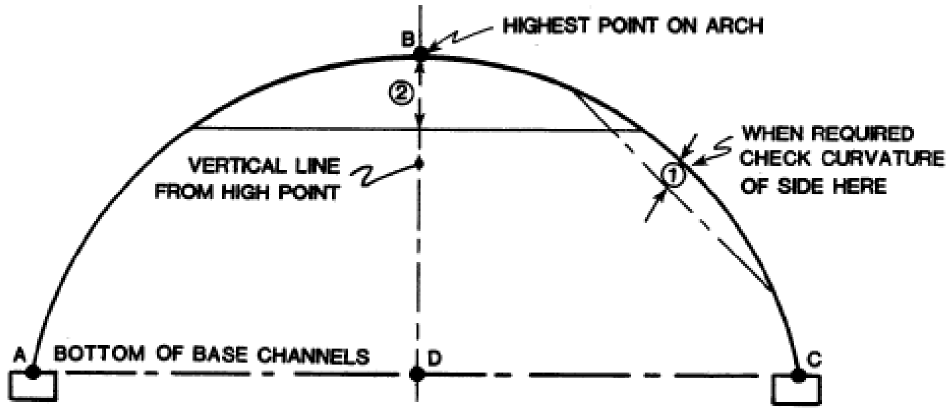


Figure 14.3.30 (Exhibit 88) Racked and Peaked Arch

Visual observation of the shape should involve looking for flattening of the sides, peaking or flattening of the crown, or racking to one side. The measurements to be recorded are illustrated in Exhibit 89. Minimum measurements include the vertical distance from the crown to the bottom of the base channels and the horizontal distances from each of the base channels to a vertical line from the highest point on the crown. These horizontal distances should be equal. When they differ by more than 10 inches or 5 percent of the span, whichever is less, racking has occurred and the curvature on the flatter side of the arch should be checked by recording chord and midordinate measurements. Racking can occur when the rise checks with the design rise. When the rise is more than 5 percent less than the design rise, the curvature of the top arc should be checked.



1. MINIMUM REQUIRED MEASUREMENTS
   - SPAN = AD + DC
   - RISE = BD

2. MINIMUM REQUIRED ELEVATIONS – B

3. IF BD GREATER THAN DESIGN BY 5% OR MORE CHECK SIDE CURVATURE

4. IF AD AND DC NOT EQUAL CHECK SIDE CURVATURE ①

5. IF BD LESS THAN DESIGN BY 5% OR MORE CHECK TOP CURVATURE ②

Figure 14.3.31 (Exhibit 89) Shape Inspection Structural Plate Arch

Arches in fair to good condition will have the following characteristics: a good shape appearance with smooth and symmetrical curvature, and a rise within three to four percent of theoretical. Marginal condition would be indicated when the arch is significantly non-symmetrical, when arch height is five to seven percent less or greater than theoretical, or when side or top plate flattening has occurred such that the plate radius is 50 to 100 percent greater than theoretical. Arches in poor to critical condition will have a poor shape appearance including significant distortion and deflection, extremely non-symmetrical shape, severe flattening (radius more than 100 percent greater than theoretical) of sides or top plates, or a rise more than eight percent greater or less than the theoretical rise. Guidelines for rating structural plate arches are shown in Exhibit 90.

RATING GUIDELINES FOR STRUCTURAL PLATE ARCH BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good, smooth symmetrical curvature • Rise: within ± 3 percent of design • Seams: properly made and tight • Metal: minor defects and damage due to contraction - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good with smooth curvature, symmetrical; slight flattening of top or sides in one section • Rise: within 3 to 4 percent of design • Seams: minor cracking at a few bolt holes; minor seam opening, possibility of soil infiltration • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: moderate erosion causing differential settlement and minor cracking in footing
6	• Shape: fair, smooth curvature but non-symetrical; slight flattening of top and sides throughout • Rise: within 4 to 5 percent of design • Seams: minor cracking of bolt holes along one or more seams; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting • Footings: moderate cracking and differential settlement of footing due to extensive erosion
5	• Shape: generally fair, significant distortion and deflection in one section; sides beginning to flattend; non-symetrical • Rise: within 5 to 7 percent of design • Seams: moderate cracking of one seam near footing; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking in footing
4	• Shape: marginal, significant distortion and deflection throughout; sides flattened with radius 100 percent greater than design • Rise: within 7 to 8 percent of design • Seams: major cracking of seam near crown; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting • Footings: rotated due to erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor, extreme distortion and deflection in one section; sides virtually flattened; extremely non-symetrical • Rise: within 8 to 10 percent of design • Seams: cracked 3" to either side of bolts • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut; major cracking and spalling
2	• Shape: critical, extreme deflection, throughout; sides flattened; extremely non-symetrical • Rise: greater than 10 percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; local reverse curve of crown and sides • Seams: failed, backfill pushing in • Road: closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.32 (Exhibit 90) Condition Rating Guidelines

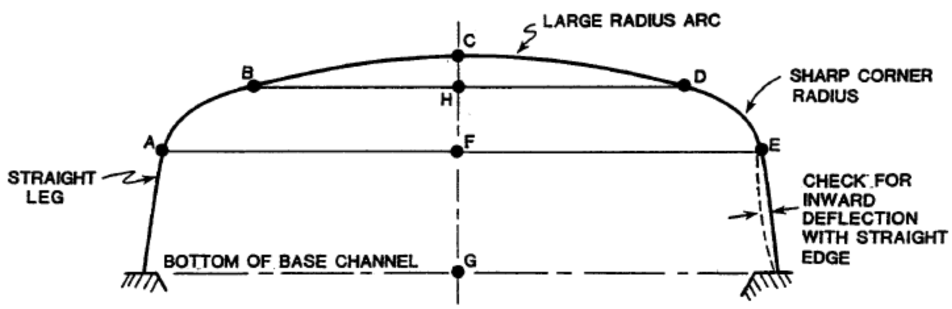
5-5.5 Corrugated Metal Box Culverts.

The box culvert is not like the other flexible buried metal structures. It behaves as a combination of ring compression action and conventional structure action. The sides are straight, not curved and the plates are heavily reinforced and have moment or bending strength that is quite significant in relation to the loads carried.

The key shape factor in a box culvert is the top arc. The design geometry is clearly very "flat" to begin with and therefore cannot be allowed to deflect much. The span at the top is also important and cannot be allowed to increase much.

The side plates often deflect slightly inward or outward. Generally an inward deflection would be the more critical as an outward movement would be restrained by soil.

Shape factors to be checked visually include flattening of top arc, outward movement of sides, or inward deflection of the sides. The inspector should note the visual appearance of the shape and should measure and record the rise and the horizontal span at the top of the straight legs as shown in Exhibit 91. If the rise is more or less than 1 ½ percent of the design rise, the curvature of the large top radius should be checked.



1. MINIMUM REQUIRED MEASUREMENTS
   - RISE = CG
   - SPAN = AE

2. IF NOT POSSIBLE TO MEASURE CG, MEASURE BD AND CH

3. IF CG DIFFERS BY MORE THAN 1½% OF DESIGN OR AE DIFFERS BY MORE THAN ±3% OF DESIGN MEASURE
   - CHORD OF TOP ARC = BD
   - MIDDLE ORDINATE OF TOP ARC = CH

Figure 14.3.33 (Exhibit 91) Shape Inspection Structural Plate Box Culverts

The radius points are not necessarily located at the longitudinal seams. Many box culverts use double radius plates and the points where the radius changes must be estimated by the inspector or can be determined from the manufacturer's literature. These points can still be referenced to the bolt pattern to describe exactly where they are. Since these are all low structures, the spots should also be marked and painted for convenient repeat inspection.

Box culverts in fair to good condition will appear to be symmetrical with smooth curves, slight or no deflection of the straight legs, a horizontal span length within five percent of the design span and the middle ordinate of the tops are within ten percent of the design. Culverts in marginal condition may appear to be non-symmetrical, have noticeable deflection in the straight legs, have spans that differ from design by five percent, or have a middle ordinate of the top arc that differ from design by 20 to 30 percent. Poor to critical conditions exist when the culvert shape appears poor, the culvert has severe deflections of the straight legs, a horizontal span that differs from design by more than five percent, or a middle ordinate of the top arc that differs from the theoretical by more than 40 to 50 percent. Guidelines for rating structural plate box culverts are shown in Exhibit 92.

RATING GUIDELINES FOR CORRUGATED METAL BOX CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design • Sides: straight leg very slightly deflected inward or outward and curvature smooth • Seams: properly made and tight • Metal: minor defects and damage due to construction - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design • Sides: straight leg slightly deflected inward or moderately deflected outward, curvature smooth • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design • Sides: straight leg moderately deflected inward or extremely deflected outward, curvature smooth • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Sides: straight leg bowed inward significantly or extremely bowed outward for distance of less than 1/4 span length • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking of footing
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 20 to 30 percent of design - Horizontal Span: more than + or - 5 percent of design - Sides: straight leg bowed inward significantly or extremely bowed outward for distance between 1/4 and 1/2 span length, curvature irregular • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting • Footings: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 30 to 40 percent less than design - Horizontal Span: more than + or - 6 percent of design Sides: straight leg extremely bowed inward for distance less than 1/2 span length or leg bowed outward severely causing bulges in metal • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 40 percent less than design - Horizontal Span: more then + or - 8 percent of design - Sides: straight leg extremely bowed inward for a distance of 1/2 to 1 span length, or leg bowed outward severely causing bulges or kinking in metal • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch.
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved • Seams; failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.

2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.34 (Exhibit 92) Condition Rating Guidelines

The following excerpts are from a reproduction of the out-of-print Culvert Inspection Manual (Supplement to Manual 70), July 1986 – Chapter 5, Section 6.

Section 6. CORRUGATED METAL LONG-SPAN CULVERTS

5-6.0 General.

This section describes methods for conducting shape inspections of long-span structures. The long-span structures addressed include four typical shapes: low profile arch, horizontal ellipse, high profile arch, and pear. These shapes are illustrated in Exhibit 93. The evaluation of shape characteristics of long-spans will vary somewhat depending upon the typical shape being inspected. However, the top or crown sections of all long-span structures have very similar geometry. The crown sections on all long-span structures can be inspected using the same criteria. This section therefore includes separate discussions on the crown section and on each of the typical long-span shapes. Guidelines are also provided for rating the condition of each shape in terms of shape characteristics and barrel defects. The methods for using the rating guidelines are the same as those described in section 5-5.1.

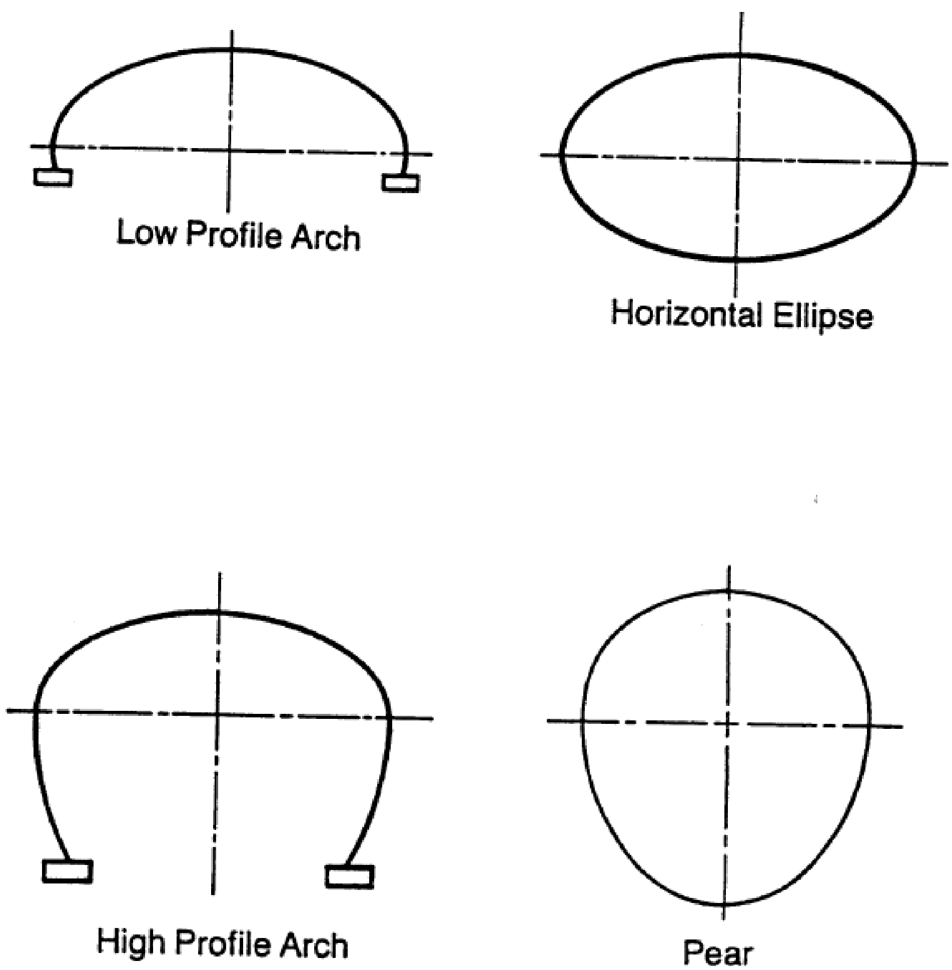
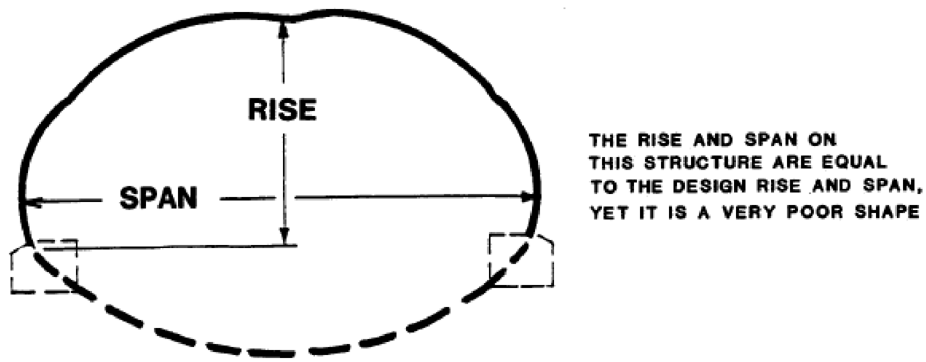


Figure 14.3.35 (Exhibit 93) Typical Long-Span Shapes

Shape inspections of long-span structures will generally consist of 1) visual observations of shape characteristics such as smooth or distorted curvature and symmetrical or non-symmetrical shape, 2) measurements of key dimensions, and 3) elevations of key points. Additional measurements may be necessary if measurements or observed shape differ significantly from design.

The visual observations are extremely important to evaluate the shape of the total cross section. Simple measurements such as rise and span do not describe curvature, yet adequate curvature is essential, as shown in Exhibit 94. However, measurements and elevations are also needed to document the current shape so that the rate change, if any, can be monitored.



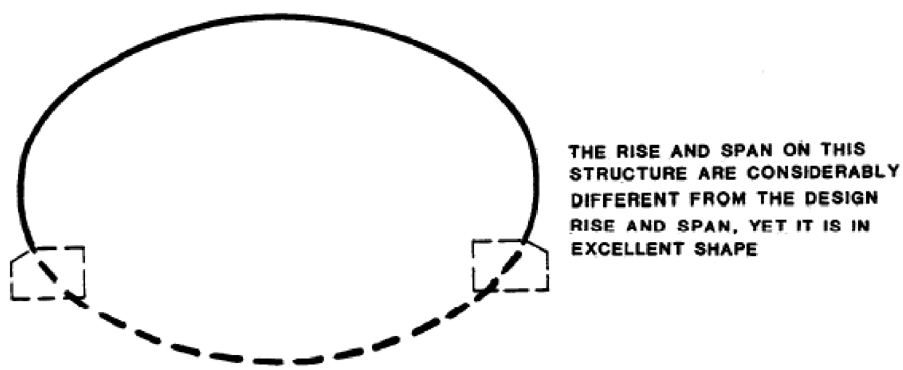


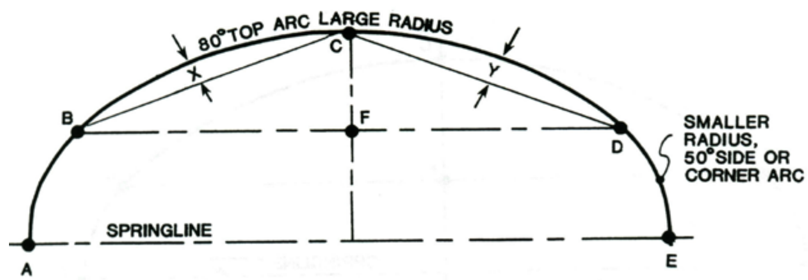
Figure 14.3.36 (Exhibit 94) Erosion Damage to Concrete Invert

Many long-spans will be too large to allow simple direct measuring. Vertical heights may be as large as 20 to 30 feet and horizontal spans may be large and as high as 12 to 15 feet above inverts. Culverts may have flowing water obscuring the invert and any reference points there. It is, therefore, in general desirable to have instrument survey points, which can be quickly checked for elevation. When direct measuring is practical a 25 foot telescoping extension rod can be used for measuring. Such rods can also serve as level rods for taking elevations.

5-6.1 Long-Span Crown Section - Shape Inspection.

As previously mentioned, the section above the springline is essentially the same for most long-span shapes. With the exception of pear shapes, the standard top geometry uses a large radius top arc of approximately 80 degrees with a radius of 15 to 25 feet. The adjacent corner or side plates are from one-half to one-fifth the top arc radius. The most important part of a long-span shape is the standard top arch geometry. Adequate curvature of the large radius top arc is critical. Inspection of the crown section should consist of a visual inspection of the general shape for smooth curvature (no distortion, flattening, peaks, or cusps) and symmetrical shape (no racking).

An inspection should also include key measurements such as the middle ordinate of the top arc. Recommended measurements and elevations are shown in exhibit 95.



1. MINIMUM REQUIRED ELEVATIONS – B, C, D

   MINIMUM REQUIRED MEASUREMENTS –
   - TOP SPAN = AE

   CALCULATE CF = ELEV C – $\frac{\text{ELEV B} + \text{ELEV D}}{2}$

2. IF CF IS GREATER THAN OR LESS THAN DESIGN BY 10% MEASURE:
   - TOP ARC CHORD = BD

3. IF BD DIFFERS BY MORE THAN 3% FROM DESIGN MEASURE FOR EACH HALF OF TOP ARC
   - HALF TOP ARC MID ORDINATES = X & Y

Note: These measurements and elevations should be obtained on all long span inspections (see exhibits 96, 98, 100 and 103).

Figure 14.3.37 (Exhibit 95) Shape Inspection Crown Section of Long Span Structures

The initial inspection should establish elevations for the radius points and the top of the crown. From these elevations the middle ordinate for the top arc can be calculated. If the actual middle ordinate is 10 percent more or less than the theoretical design mid-ordinate the horizontal span for the top arc should also be measured. For standard 80 degree arcs the theoretical middle ordinate is equal to 0.234 times the theoretical radius of the top arc. This span is not easy to measure on many long-span structures and need not be measured if the top arc mid-ordinate is within 10 percent of theoretical. Even if it is convenient and practical to direct measure the vertical heights of the points on the top arc from the bottom of the structure, it is wise to also establish their elevations from a reliable benchmark. Bottom reference points can be wiped out by erosion, covered with debris, or covered by water. When direct vertical measuring is practical, the shape may be checked on subsequent inspections with direct measurement. However, it is still important to establish elevations in case bottom reference points are lost or inaccessible.

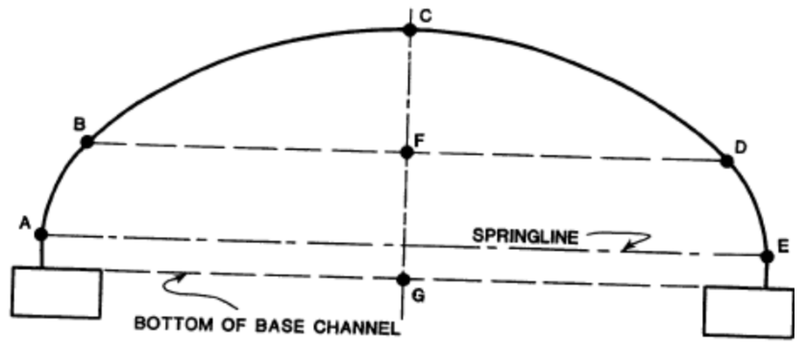
Crown sections in good condition will have a shape appearance that is good, with smooth and symmetrical curvature. The actual middle ordinate should be within 10 percent of the theoretical, and the horizontal span (if measured) should be within five percent of theoretical. Crown sections in fair condition will have a fair to good shape appearance, smooth curvature but possibly slightly non-symmetrical. Middle ordinates of the top arc may be within 11 to 15 percent of theoretical and the horizontal span may differ by more than 5 percent of theoretical.

Crown sections in marginal condition will have measurements similar to those described for fair shape. However, the shape appearance will be only fair to marginal with noticeable distortion, deflection, or non-symmetrical curvature. When the curvature is noticeably distorted or non-symmetrical, the sides should be checked for flattening by measuring the middle ordinates of the halves of the top arc. Crown sections with marginal shape may have middle ordinates for top half arcs that are 30 to 50 percent less than theoretical.

Crown sections in poor to critical condition will have a poor to critical shape appearance with severe distortion or deflection. The middle ordinate of the top arc may be as much as 20 percent less than theoretical, while middle ordinates of the top arc halves may be 50 to 70 percent less than theoretical.

5-6.2 Low Profile Long-Span Arch - Shape Inspection.

The low profile arch is essentially the same as the crown section except that the sides are carried about 10 degrees below the springline to the footing. These structures are low and can be measured more easily than other long-span shapes. Recommended measurements and elevations are shown in exhibit 96. Rating guidelines are listed in exhibit 97.



AE = SPAN, CG = RISE OR HEIGHT

1. MINIMUM REQUIRED MEASUREMENTS -
   - SPAN = AE
   - TOP ARC CHORD = BD
   - RISE = CG
2. MINIMUM REQUIRED ELEVATIONS B, C, D
3. CALCULATE CF FROM ELEVATIONS

$$CF = \text{ELEV. } C - \frac{\text{ELEV. } B + \text{ELEV. } D}{2}$$

Note: Use with exhibit 95, crown inspection.

Figure 14.3.38 (Exhibit 96) Shape Inspection Low Profile Long Span Arch

RATING GUIDELINES FOR LOW PROFILE ARCH LONG-SPAN CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design • Seams: properly made and tight • Metal: minor defects and damage due to construction - Aluminum: superficial corrosin, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core allow - Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking of footing
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy, - Steel: extensive heavy rust, deep pitting • Footings: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved • Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.39 (Exhibit 97) Condition Rating Guidelines

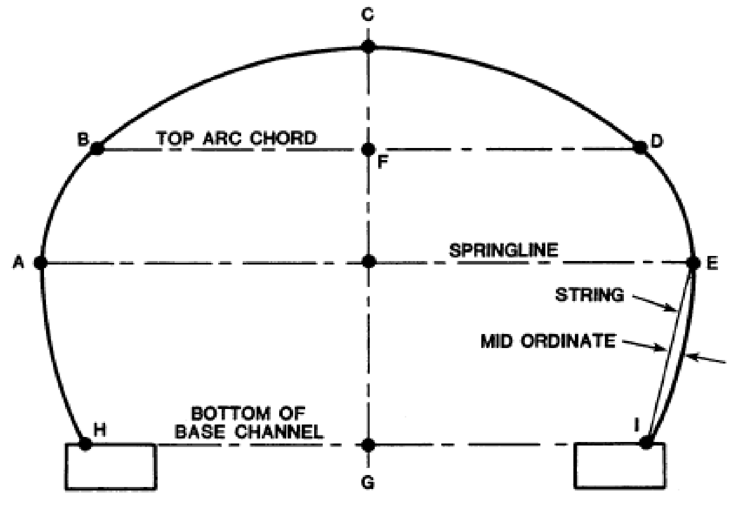
Because arches are fixed on concrete footings, backfill pressures will try to flatten the sides and peak the top. Another important shape factor is symmetry. If the base channels are not square to the centerline of the structure racking may occur during erection. In racked structures, the crown moves laterally and the curvature in one side becomes flatter while the curvature in the other side increases. Backfill pressures may cause this condition to worsen.

5-6.3 High Profile Long-Span Arch – Shape Inspection.

High profile arches have a standard crown section geometry but have high large radius side walls below the springline. Curvature in these side plates is important. In shallow fills or minimum covers, the lateral soil pressures may approach or exceed the loads over the culvert. Excessive lateral forces could cause the sidewall to flatten or buckle inward.

Inspectors should visually inspect high profile arches for flattening of the side plates. Additionally, high profile arches have the same tendencies as regular arches for peaking and racking, so inspectors must also look for peaked top arcs and non-symmetrical or racked arches.

Recommended measurements and elevations are shown in Exhibit 98. The shape of the crown section is the most important shape factor. It can be measured and evaluated using the same criteria as that described for the standard crown section. If flattening is observed in the high sidewall the curvature of the sides should be checked by measuring the middle ordinate of the side walls. If the sidewall middle ordinate is no more than 50 to 70 percent less than the theoretical middle ordinate and no other shape problems are found the arch's shape may be considered fair. When the middle ordinate approaches 75 to 80 percent less than theoretical, the shape should be considered marginal. If the middle ordinate is more than 80 to 90 percent less than theoretical the shape should be considered poor to critical. Rating guidelines are provided in Exhibit 99.



AE = SPAN, CG = RISE

1. MINIMUM REQUIRED MEASUREMENTS
   - SPAN = AE

2. MINIMUM REQUIRED ELEVATIONS – B, C, D, H, I

3. CALCULATE CF FROM ELEVATIONS

Note: Use with exhibit 95, crown inspection.

Figure 14.3.40 (Exhibit 98) Shape Inspection High Profile Long-Span Arch

RATING GUIDELINES FOR HIGH PROFILE ARCH LONG-SPAN CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design - Side Plates: smooth curvature • Seams: properly made and tight • Metal: minor defects and damage due to construction - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design - Side Plates: side flattened, mid-ordinate less than 50 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 35 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 25 percent of design • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking of footing
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 20 percent of design • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of core alloy - Steel: extensive heavy rust, deep pitting • Footings: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design - Side Plates: side flattened, mid-ordinate less than 12 percent of design • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design - Side Plates: side flattened, mid-ordinate less than10percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved - Side Plates: side flat or reversed curved • Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

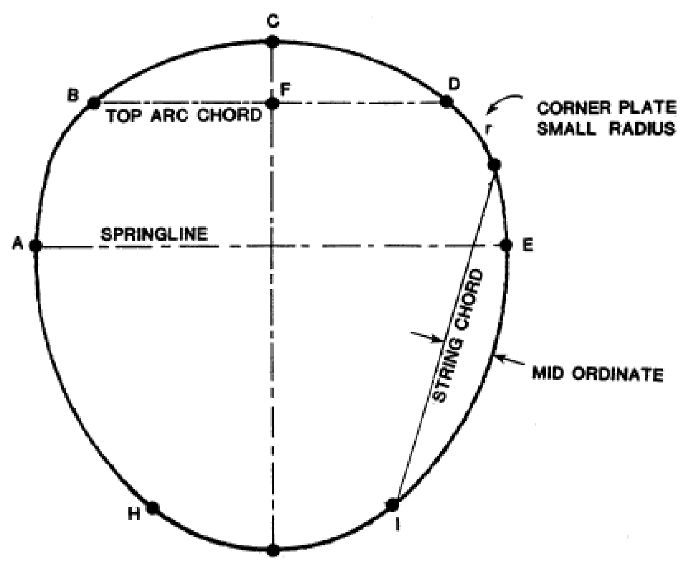
NOTES: 1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.41 (Exhibit 99) Condition Rating Guidelines

5-6.4 Pear Shape Long-Span – Shape Inspection.

The crown section of the pear shape differs from the standard top arch in that smaller radius corner arcs stop short of the horizontal springline. The large radius sides extend above the plane of the horizontal span. In checking curvature of the sides, the entire arc should be checked. Side flattening, particularly in shallow fills, is the most critical shape factor.

The pear shape behaves similarly to the high profile arch. It is essentially a high profile with a metal bottom instead of concrete footings. Pears may be inspected using the criteria for a high profile arch. The recommended measurements and elevations are shown in Exhibit 100. Rating guidelines are provided in Exhibit 101.



AE = SPAN, CG = RISE

1. MINIMUM REQUIRED MEASUREMENT – AE
   - SPAN = AE

2. MINIMUM REQUIRED ELEVATIONS B, C, D

3. WHEN FLATTENING OBSERVED IN SIDE, CHECK MID ORDINATE (RECORD CHORD LENGTH USED)

Note: Use with exhibit 95, crown inspection.

Figure 14.3.42 (Exhibit 100) Shape Inspection Long Span Pear-Shape

RATING GUIDELINES FOR PEAR SHAPED LONG-SPAN CULVERT BARREL

RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design - Side Plates: smooth curvature • Seams: properly made and tight • Metal: minor defects and damage due to construction; superficial corrosion with no pitting - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design - Side Plates: side flattened, mid-ordinate less than 50 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 35 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 25 percent of design • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: corroded locally - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Side Plates: side flattened, mid-ordinate less than 20 percent of design • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of alloy - Steel: extensive heavy rust, deep pitting
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design - Side Plates: side flattened, mid-ordinate less than 12 percent of design • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design - Side Plates: side flattened, mid-ordinate less than10percent of design • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved - Side Plates: side flat or reversed curved - Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.43 (Exhibit 101) Condition Rating Guidelines

5-6.5 Horizontal Ellipse – Shape Inspections.

For horizontal ellipses the most important shape factor is adequate curvature in the crown section. The crown section uses the standard long-span crown geometry. The sides and bottom behave similar to the corners and bottom of pipe arches. The invert has relatively minor pressure when compared with the sides, which may have several times the bearing pressure of the invert. As a result the corners and sides have the tendency to push down into the soil while the bottom does not move. The effect is as if the bottom pushed up. Inspectors should look for indications of bottom flattening and differential settlement between the side and bottom sections, as illustrated in Exhibit 102.

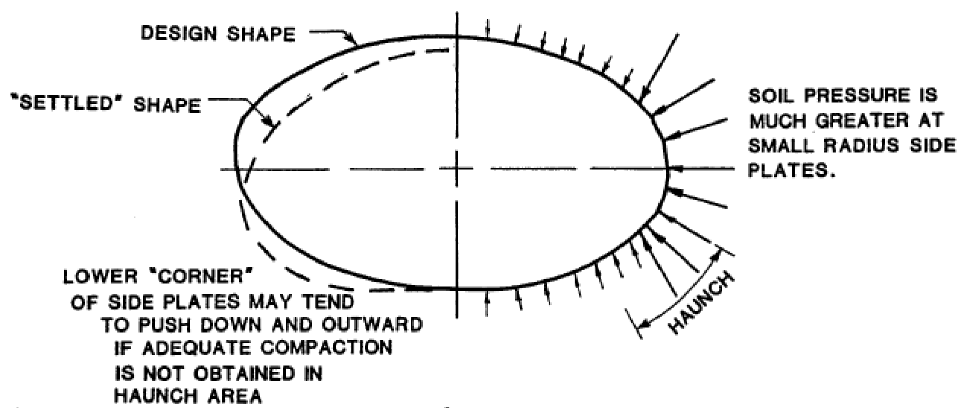


Figure 14.3.44 (Exhibit 102) Potential for Differential Settlement in Horizontal Ellipse

The recommended measurements and evaluations for a shape inspection of horizontal ellipse are shown in Exhibit 103. The measurements are essentially the same as those recommended for a standard crown section. Shape evaluation of an ellipse is also essentially the same as the evaluation of a standard crown section except that the curvature of the bottom should also be evaluated. Marginal shape would be indicated when the bottom is flat in the center and corners are beginning to deflect downward or outward. Critical shape conditions would be indicated by reverse curvature in the bottom arc. Guidelines for rating horizontal ellipse shape culverts are provided in Exhibit 104.

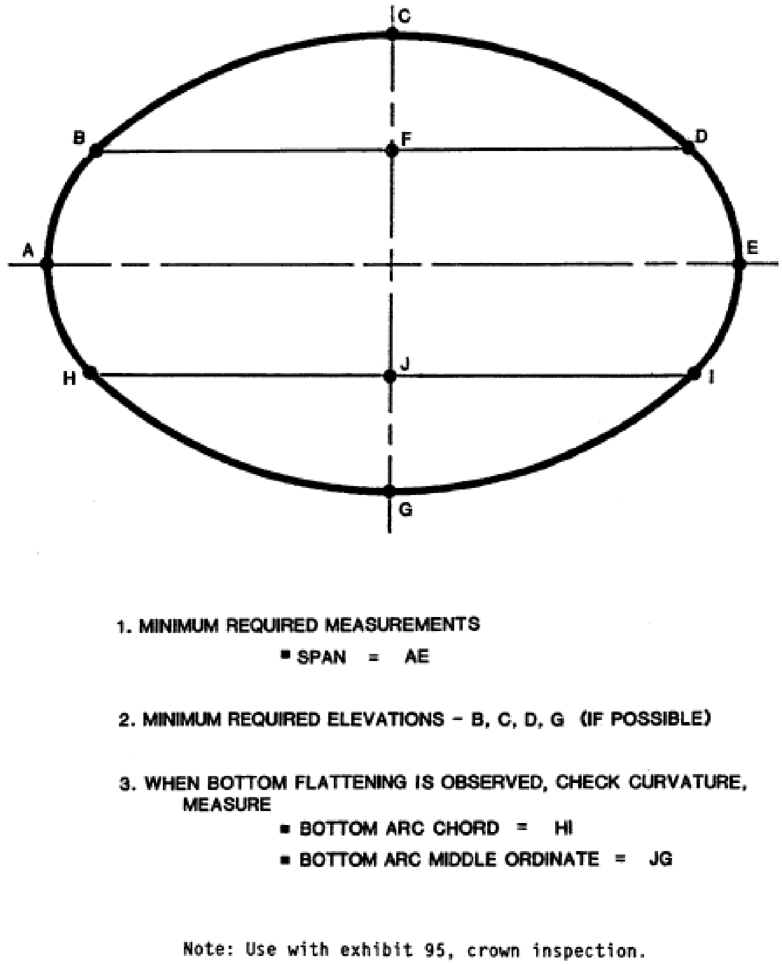


Figure 14.3.45 (Exhibit 103) Shape Inspection Long-Span Horizontal Ellipse

RATING GUIDELINES FOR HORIZONTAL ELLIPSE LONG SPAN CULVERT BARREL

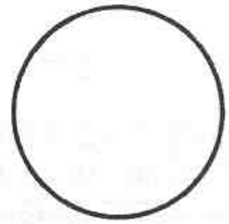
RATING	CONDITION
9	• New condition
8	• Shape: good appearance, smooth symmetrical curvature - Top Arc Mid-Ordinate: within 11 percent of design - Horizontal Span: within 5 percent of design - Bottom Arc: smooth curvature, mid-ordinate within 50 percent of design • Seams: properly made and tight • Metal: minor defects and damage due to construction - Aluminum: superficial corrosion, slight pitting - Steel: superficial rust, no pitting • Footings: good with no erosion
7	• Shape: generally good; curvature is smooth and symmetrical - Top Arc Mid-Ordinate: within 11 percent to 15 percent of design - Horizontal Span: within 5 percent of design - Bottom Arc: bottom flattened, mid-ordinate less than 50 percent of design • Seams: minor cracking at a few bolt holes; minor seam openings, possibility of backfill infiltration exists • Metal: - Aluminum: moderate corrosion, no attack of core alloy - Steel: moderate rust, slight pitting • Footings: minor differential settlement due to erosion; minor hairline cracking in footing
6	• Shape: smooth curvature, shape is non-symetrical - Top Arc Mid-Ordinate: within 15 percent of design - Horizontal Span: more than + or - 5 percent of design - Bottom Arc: bottom flattened and irregular, mid-ordinate less than 50 percent of design • Seams: minor cracking at bolt holes along one seam; evidence of backfill infiltration • Metal: - Aluminum: significant corrosion, minor attack of core alloy - Steel: fairly heavy rust, moderate pitting • Footing: differential settlement due to extensive erosion; moderate cracking of footing
5	• Shape: generally fair; significant distortion and deflection in one section;half top arcs beginning to flatten; mid-ordinate of half top arc 30 percent less than design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Bottom Arc: bottom virtually flat over center half of arc • Seams: major cracking in one location; infiltration of soil causing slight deflection • Metal: - Aluminum: significant corrosion, moderate attack of core alloy - Steel: scattered heavy rust, deep pitting • Footings: significant undercutting of footing and extreme differential settlement; major cracking of footing
4	• Shape: marginal, significant distortion and deflection throughout; mid-ordinate of half top arc less than 50 percent of design - Top Arc Mid-Ordinate: within 15 to 20 percent of design - Horizontal Span: more than + or - 5 percent of design - Bottom Arc: bottom virtually flat over center half of arc and deflected down at corners • Seams: significant seam cracking all along seam; infiltration of soil causing major deflection • Metal: - Aluminum: extensive corrosion, significant attack of alloy - Steel: extensive heavy rust, deep pitting • Footings: rotated due erosion and undercutting; settlement has caused damage to metal arch
3	• Shape: poor extreme distortion and deflection in one section and ordinate of half top arc 50 to 70 percent less than design - Top Arc Mid-Ordinate: 20 to 30 percent less than design - Horizontal Span: more than + or - 6 percent of design - Bottom Arc: bottom reverse curved in center • Seams: cracked 3" or more to either side of bolt; infiltration of backfill causing severe deflection locally • Metal: - Aluminum: extensive corrosion, attack of core alloy, scattered perforations - Steel: extensive heavy rust, deep pitting, scattered perforations • Footing: rotated, severely undercut, major cracking and spalling of footing, significant damage to structure
2	• Shape: critical, extreme distortion and deflection throughout; mid-ordinate of half top arc more than 70 percent less than design - Top Arc Mid-Ordinate: more than 30 percent less than design - Horizontal Span: more then + or - 8 percent of design - Bottom Arc: bottom reversed curved in center and bulged out at sides • Seams: cracked from bolt to bolt; significant amounts of backfill infiltration throughout • Metal: - Aluminum: extensive perforations due to corrosion - Steel: extensive perforations due to rust • Footing: severe differential settlement has caused distortion and kinking of metal arch
1	• Shape: severe due to partial collapse; top arc curvature flat or reverse curved • Seams: failed, backfill pushing in • Road closed to traffic
0	• Structure: completely collapsed • Road: closed to traffic

NOTES: 1. See Coding Guide for description of Rating Scale.
2. As a starting point, select the lowest rating which matches actual conditions.

Figure 14.3.46 (Exhibit 104) Condition Rating Guidelines

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

ROUND



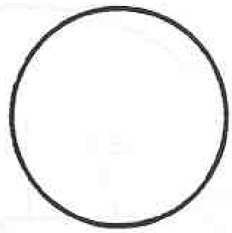
Handling Weight of Corrugated Steel Pipe (2⅔ × ½ in.)
Estimated Average Weights—Not for Specification Use*

Inside Diameter, in.	Specified Thickness, in.	Approximate Pounds per Lineal Ft**			
		Galvanized	Full-Coated	Full-Coated and Invert Paved	Full-Coated and Full Paved
12	0.052	8	10	13	
	0.064	10	12	15	
	0.079	12	14	17	
15	0.052	10	12	15	
	0.064	12	15	18	
	0.079	15	18	21	
18	0.052	12	14	17	
	0.064	15	19	22	
	0.079	18	22	25	
21	0.052	14	16	19	
	0.064	17	21	26	
	0.079	21	25	30	
24	0.052	15	17	20	
	0.064	19	24	30	45
	0.079	24	29	35	50
30	0.052	20	22	25	
	0.064	24	30	36	55
	0.079	30	36	42	60
36	0.052	24	26	29	
	0.064	29	36	44	65
	0.079	36	43	51	75
42	0.052	28	30	33	
	0.064	34	42	51	
	0.079	42	50	59	85
48	0.052	31	33	36	
	0.064	38	48	57	
	0.079	48	58	67	95
54	0.064	44	55	66	95
	0.079	54	65	76	105
60	0.079	60	71	85	
	0.109	81	92	106	140
66	0.109	89	101	117	160
	0.138	113	125	141	180
72	0.109	98	112	129	170
	0.138	123	137	154	210
78	0.109	105	121	138	200
	0.138	133	149	166	230
84	0.109	113	133	155	225
	0.138	144	161	179	240
90	0.109	121	145	167	
	0.138	154	172	192	
	0.168	186	204	224	
96	0.138	164	191	217	
	0.168	198	217	239	

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute)

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

ROUND

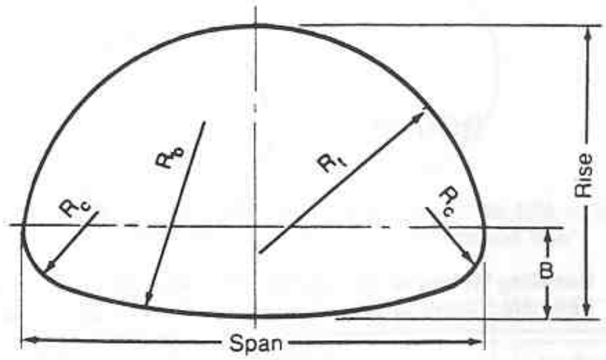


Handling Weight of Corrugated Steel Pipe (3 × 1 in. or 5 × 1 in.)***
Estimated Average Weights—Not for Specification Use

Inside Diameter, in.	Specified Thickness, in.	Approximate Pounds per Lineal Ft**			
		Galvanized	Full-Coated	Full-Coated and Invert Paved	Full-Coated and Full Paved
54	0.064	50	66	84	138
	0.079	61	77	95	149
60	0.064	55	73	93	153
	0.079	67	86	105	165
66	0.064	60	80	102	168
	0.079	74	94	116	181
72	0.064	66	88	111	183
	0.079	81	102	126	197
78	0.064	71	95	121	198
	0.079	87	111	137	214
84	0.064	77	102	130	213
	0.079	94	119	147	230
90	0.064	82	109	140	228
	0.079	100	127	158	246
96	0.064	87	116	149	242
	0.079	107	136	169	262
102	0.064	93	124	158	258
	0.079	114	145	179	279
108	0.064	98	131	166	273
	0.079	120	153	188	295
114	0.064	104	139	176	289
	0.079	127	162	199	312
120	0.064	109	146	183	296
	0.079	134	171	210	329
	0.109	183	220	259	378
126	0.079	141	179	220	346
	0.109	195	233	274	400
132	0.079	148	188	231	363
	0.109	204	244	287	419
138	0.079	154	196	241	379
	0.109	213	255	300	438
144	0.109	223	267	314	458
	0.138	282	326	373	517

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



Sizes and Layout Details—CSP Pipe Arches
2⅔ × ½ in. Corrugation

Equiv. Diameter, in.	Span, in.	Rise, in.	Waterway Area, ft²	Layout Dimensions B in.	R_c in.	R_t in.	R_b in.
15	17	13	1.1	4⅛	3½	8⅝	25⅝
18	21	15	1.6	4⅞	4⅛	10¾	33⅛
21	24	18	2.2	5⅝	4⅞	11⅞	34⅝
24	28	20	2.9	6½	5½	14	42¼
30	35	24	4.5	8⅛	6⅞	17⅞	55⅛
36	42	29	6.5	9¾	8¼	21½	66⅛
42	49	33	8.9	11⅜	9⅝	25⅛	77¼
48	57	38	11.6	13	11	28⅝	88¼
54	64	43	14.7	14⅝	12⅜	32¼	99¼
60	71	47	18.1	16¼	13¾	35¾	110¼
66	77	52	21.9	17⅞	15⅛	39⅜	121¼
72	83	57	26.0	19½	16½	43	132¼

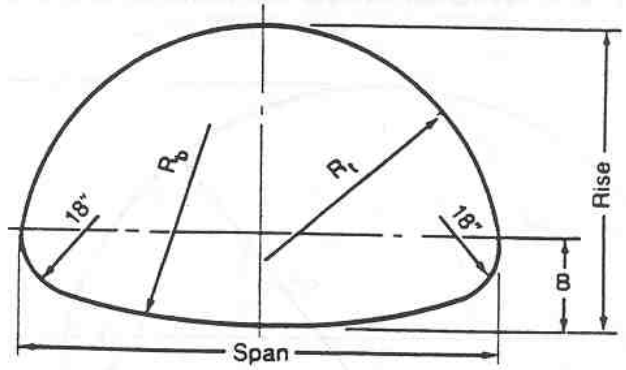
Dimensions shown not for specification purposes, subject to manufacturing tolerances.

Sizes and Layout Details—CSP Pipe-Arches
3 × 1 in. Corrugation

Equiv. Diameter, in.	Size, in.	Span, in.	Rise, in.	Waterway Area, ft²	Layout Dimensions B in.	R_c in.	R_t in.	R_b in.
54	60 × 46	58½	48½	15.6	20½	18¾	29⅜	51⅛
60	66 × 51	65	54	19.3	22¾	20¾	32⅝	56¼
66	73 × 55	72½	58¼	23.2	25⅛	22⅞	36¾	63¾
72	81 × 59	79	62½	27.4	23¾	20⅞	39½	82⅝
78	87 × 63	86½	67¼	32.1	25¾	22⅝	43⅜	92¼
84	95 × 67	93½	71¾	37.0	27¾	24⅜	47	100¼
90	103 × 71	101½	76	42.4	29¾	26⅛	51¼	111⅝
96	112 × 75	108½	80½	48.0	31⅝	27¾	54⅞	120¼
102	117 × 79	116½	84¾	54.2	33⅝	29½	59⅜	131¾
108	128 × 83	123½	89¼	60.5	35⅝	31¼	63¼	139¾
114	137 × 87	131	93¾	67.4	37⅝	33	67⅜	149½
120	142 × 91	138½	98	74.5	39½	34¾	71⅝	162⅜

Figure 14.3.47 Standard Sizes for Corrugated Steel (Source: American Iron and Steel Institute), continued

STANDARD SIZES FOR CORRUGATED STEEL CULVERTS

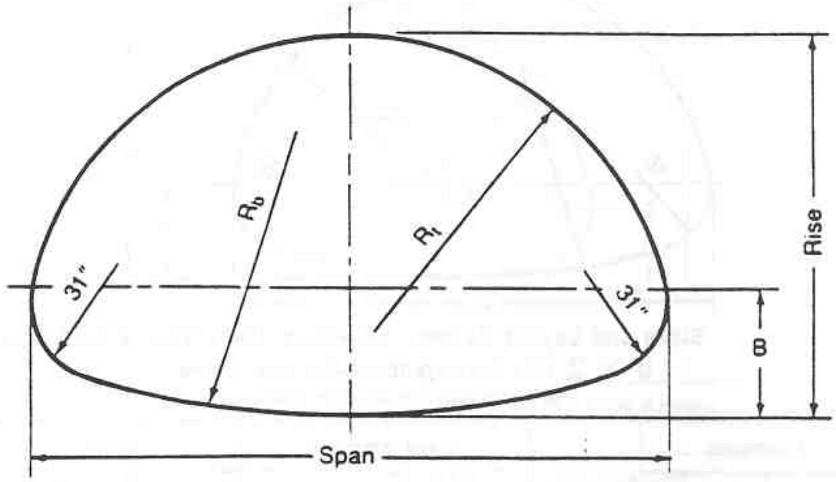


Sizes and Layout Details—Structural Plate Steel Pipe-Arches
6 × 2 in. Corrugations—Bolted Seams
18-inch Corner Radius Rc

Dimensions		Waterway Area, ft²	Layout Dimensions			Periphery		
Span, ft-in.	Rise, ft-in.		B in.	R_t ft	R_b ft	No. of Plates	Total N	Total Pi
6-1	4-7	22	21.0	3.07	6.36	5	22	66
6-4	4-9	24	20.5	3.18	8.22	5	23	69
6-9	4-11	26	22.0	3.42	6.96	5	24	72
7-0	5-1	28	21.4	3.53	8.68	5	25	75
7-3	5-3	31	20.8	3.63	11.35	6	26	78
7-8	5-5	33	22.4	3.88	9.15	6	27	81
7-11	5-7	35	21.7	3.98	11.49	6	28	84
8-2	5-9	38	20.9	4.08	15.24	6	29	87
8-7	5-11	40	22.7	4.33	11.75	7	30	90
8-10	6-1	43	21.8	4.42	14.89	7	31	93
9-4	6-3	46	23.8	4.68	12.05	7	32	96
9-6	6-5	49	22.9	4.78	14.79	7	33	99
9-9	6-7	52	21.9	4.86	18.98	7	34	102
10-3	6-9	55	23.9	5.13	14.86	7	35	105
10-8	6-11	58	26.1	5.41	12.77	7	36	108
10-11	7-1	61	25.1	5.49	15.03	7	37	111
11-5	7-3	64	27.4	5.78	13.16	7	38	114
11-7	7-5	67	26.3	5.85	15.27	8	39	117
11-10	7-7	71	25.2	5.93	18.03	8	40	120
12-4	7-9	74	27.5	6.23	15.54	8	41	123
12-6	7-11	78	26.4	6.29	18.07	8	42	126
12-8	8-1	81	25.2	6.37	21.45	8	43	129
12-10	8-4	85	24.0	6.44	26.23	8	44	132
13-5	8-5	89	26.3	6.73	21.23	9	45	135
13-11	8-7	93	28.9	7.03	18.39	9	46	138
14-1	8-9	97	27.6	7.09	21.18	9	47	141
14-3	8-11	101	26.3	7.16	24.80	9	48	144
14-10	9-1	105	28.9	7.47	21.19	9	49	147
15-4	9-3	109	31.6	7.78	18.90	9	50	150
15-6	9-5	113	30.2	7.83	21.31	10	51	153
15-8	9-7	118	28.8	7.89	24.29	10	52	156
15-10	9-10	122	27.4	7.96	28.18	10	53	159
16-5	9-11	126	30.1	8.27	24.24	10	54	162
16-7	10-1	131	28.7	8.33	27.73	10	55	165

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



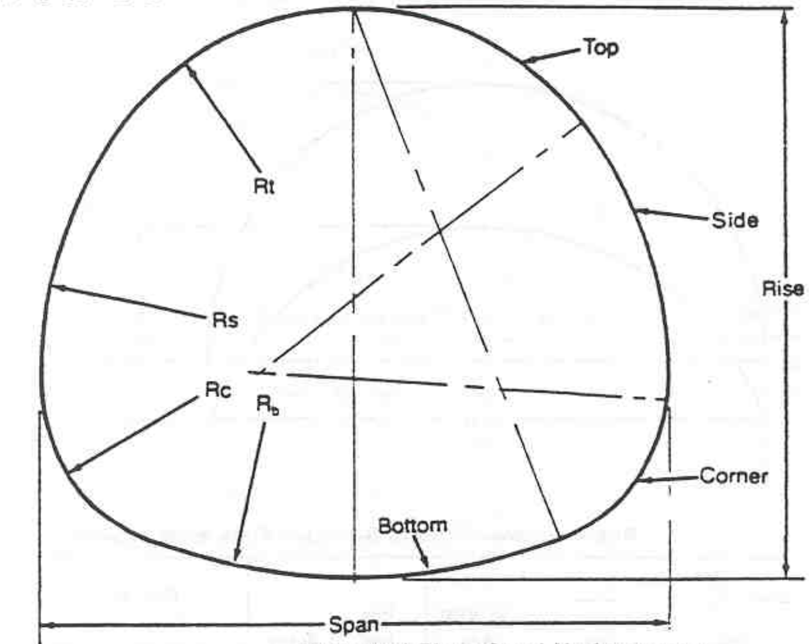
Sizes and Layout Details—Structural Plate Steel Pipe-Arches[11]
6 × 2 in. Corrugations—Bolted Seams
31-in Corner Radius, R_c

Dimensions		Waterway Area, ft²	Layout Dimensions			Periphery		
Span, ft-in.	Rise, ft-in.		B in.	R_t ft	R_b ft	No. of Plates	Total N	Total Pi
13-3	9-4	97	38.5	6.68	16.05	8	46	138
13-6	9-6	102	37.7	6.78	18.33	8	47	141
14-0	9-8	105	39.6	7.03	16.49	8	48	144
14-2	9-10	109	38.8	7.13	18.55	8	49	147
14-5	10-0	114	37.9	7.22	21.38	8	50	150
14-11	10-2	118	39.8	7.48	18.98	9	51	153
15-4	10-4	123	41.8	7.76	17.38	9	52	156
15-7	10-6	127	40.9	7.84	19.34	10	53	159
15-10	10-8	132	40.0	7.93	21.72	10	54	162
16-3	10-10	137	42.1	8.21	19.67	10	55	165
16-6	11-0	142	41.1	8.29	21.93	10	56	168
17-0	11-2	146	43.3	8.58	20.08	10	57	171
17-2	11-4	151	42.3	8.65	22.23	10	58	174
17-5	11-6	157	41.3	8.73	24.83	10	59	177
17-11	11-8	161	43.5	9.02	22.55	10	60	180
18-1	11-10	167	42.4	9.09	24.98	10	61	183
18-7	12-0	172	44.7	9.38	22.88	10	62	186
18-9	12-2	177	43.6	9.46	25.19	10	63	189
19-3	12-4	182	45.9	9.75	23.22	10	64	192
19-6	12-6	188	44.8	9.83	25.43	11	65	195
19-8	12-8	194	43.7	9.90	28.04	11	66	198
19-11	12-10	200	42.5	9.98	31.19	11	67	201
20-5	13-0	205	44.9	10.27	28.18	11	68	204
20-7	13-2	211	43.7	10.33	31.13	12	69	207

Dimensions are to inside crests and are subject to manufacturing tolerances.
$N = 3$ Pi $= 9.6$ in.

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



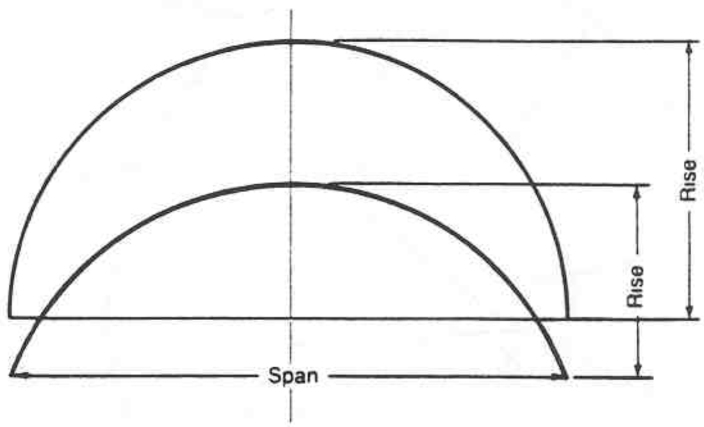
Structural Plate Steel Underpasses
Sizes and Layout Details

Span × Rise, ft and in.		Periphery			Layout Dimensions in In.			
		N	Pi	No. of Plates per Ring	R_t	R_s	R_c	R_b
5-8	5-9	24	72	6	27	53	18	Flat
5-8	6-6	26	78	6	29	75	18	Flat
5-9	7-4	28	84	6	28	95	18	Flat
5-10	7-8	29	87	7	30	112	18	Flat
5-10	8-2	30	90	6	28	116	18	Flat
12-2	11-0	47	141	8	68	93	38	136
12-11	11-2	49	147	9	74	92	38	148
13-2	11-10	51	153	11	73	102	38	161
13-10	12-2	53	159	11	77	106	38	168
14-1	12-10	55	165	11	77	115	38	183
14-6	13-5	57	171	11	78	131	38	174
14-10	14-0	59	177	11	79	136	38	193
15-6	14-4	61	183	12	83	139	38	201
15-8	15-0	63	189	12	82	151	38	212
16-4	15-5	65	195	12	86	156	38	217
16-5	16-0	67	201	12	88	159	38	271
16-9	16-3	68	204	12	89	168	38	246
17-3	17-0	70	210	12	90	174	47	214
18-4	16-11	72	216	12	99	157	47	248
19-1	17-2	74	222	13	105	156	47	262
19-6	17-7	76	228	13	107	158	47	295
20-4	17-9	78	234	13	114	155	47	316

All dimensions, to nearest whole number, are measured from inside crests.
Tolerances should be allowed for specification purposes. 6 × 2 in. Corrugations.

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



Representative Sizes of Structural Plate Steel Arches

Dimensions(¹) Span, ft	Rise, ft-in.	Waterway Area, ft²	Rise over Span(²)	Radius, in.	Nominal Arc Length N(³)	Pi, in.
6.0	1-9½	7½	0.30	41	9	27
	2-3½	10	0.38	37½	10	30
	3-2	15	0.53	36	12	36
7.0	2-4	12	0.34	45	11	33
	2-10	15	0.40	43	12	36
	3-8	20	0.52	42	14	42
8.0	2-11	17	0.37	51	13	39
	3-4	20	0.42	48½	14	42
	4-2	26	0.52	48	16	48
9.0	2-11	18½	0.32	59	14	42
	3-10½	26½	0.43	55	16	48
	4-8½	33	0.52	54	18	54
10.0	3-5½	25	0.35	64	16	48
	4-5	34	0.44	60½	18	54
	5-3	41	0.52	60	20	60
11.0	3-6	27½	0.32	73	17	51
	4-5½	37	0.41	67½	19	57
	5-9	50	0.52	66	22	66
12.0	4-0½	35	0.34	77½	19	57
	5-0	45	0.42	73	21	63
	6-3	59	0.52	72	24	72
13.0	4-1	38	0.32	86½	20	60
	5-1	49	0.39	80½	22	66
	6-9	70	0.52	78	26	78
14.0	4-7½	47	0.33	91	22	66
	5-7	58	0.40	86	24	72
	7-3	80	0.52	84	28	84

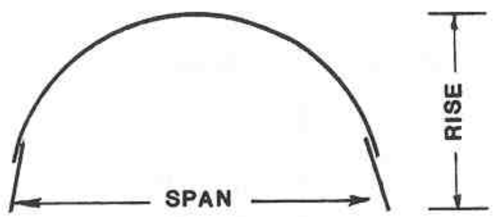
(Table continued on following page)

(¹)Dimensions are to inside crests and are subject to manufacturing tolerances.
(²)R/S ratio varies from 0.30 to 0.52. Intermediate spans and rises are available.
(³)N = 3 Pi = 9.6 in. 6 × 2 in. Corrugations—Bolted Seams.

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

ARCH



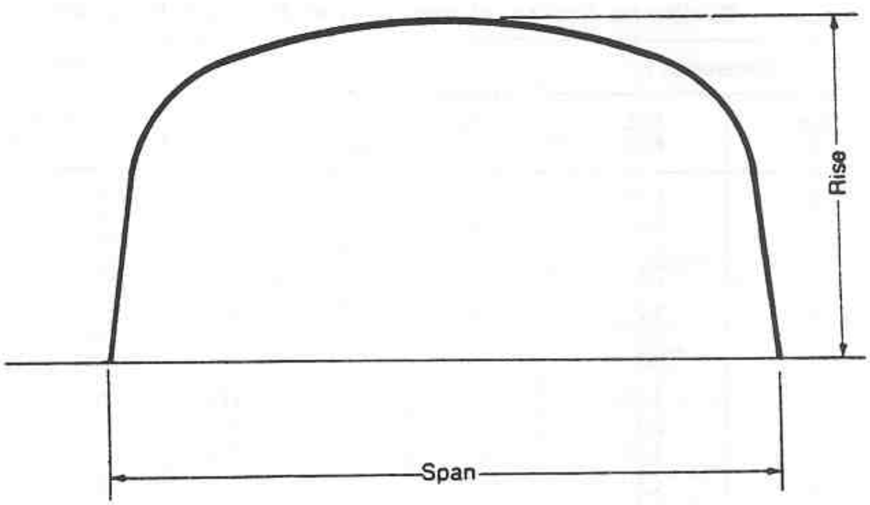
Continued. Representative Sizes of Structural Plate Steel Arches

Dimensions(¹) Span, ft	Rise, ft-in.	Waterway Area, ft²	Rise over Span(²)	Radius, in.	Nominal Arc Length N(³)	Pi, in.
15.0	4-7½	50	0.31	101	23	69
	5-8	62	0.38	93	25	75
	6-7	75	0.44	91	27	81
	7-9	92	0.52	90	30	90
16.0	5-2	60	0.32	105	25	75
	7-1	86	0.45	97	29	87
	8-3	105	0.52	96	32	96
17.0	5-2½	63	0.31	115	26	78
	7-2	92	0.42	103	30	90
	8-10	119	0.52	102	34	102
18.0	5-9	75	0.32	119	28	84
	7-8	104	0.43	109	32	96
	8-11	126	0.50	108	35	105
19.0	6-4	87	0.33	123	30	90
	8-2	118	0.43	115	34	102
	9-5½	140	0.50	114	37	111
20.0	6-4	91	0.32	133	31	93
	8-3½	124	0.42	122	35	105
	10-0	157	0.50	120	39	117
21.0	6-11	104	0.33	137	33	99
	8-10	140	0.42	128	37	111
	10-6	172	0.50	126	41	123
22.0	6-11	109	0.31	146	34	102
	8-11	146	0.40	135	38	114
	11-0	190	0.50	132	43	129
23.0	8-0	134	0.35	147	37	111
	9-10	171	0.43	140	41	123
	11-6	208	0.50	138	45	135
24.0	8-6	150	0.35	152	39	117
	10-4	188	0.43	146	43	129
	12-0	226	0.50	144	47	141
25.0	8-6½	155	0.34	160	40	120
	10-10½	207	0.43	152	45	135
	12-6	247	0.50	150	49	147

(¹)Dimensions are to inside crests and are subject to manufacturing tolerances.
(²)R/S ratio varies from 0.30 to 0.52. Intermediate spans and rises are available.
(³)N = 3 Pi = 9.6 in. 6 × 2 in. Corrugations—Bolted Seams.

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES FOR CORRUGATED STEEL CULVERTS



Layout Details
Corrugated Steel Box Culverts

Rise, ft-in.	Span, ft-in.	Area ft²	Rise, ft-in.	Span, ft-in.	Area ft²
2-7	9-8	20.8	3-9	12-10	41.0
2-8	10-5	23.2	3-10	13-6	44.5
2-9	11-1	25.7	3-10	17-4	55.0
2-10	11-10	28.3	3-11	14-2	48.2
2-11	12-6	31.1	3-11	18-0	59.1
3-1	13-3	34.0	4-1	14-10	52.0
3-2	13-11	37.1	4-1	18-8	63.4
3-3	14-7	40.4	4-2	10-7	36.4
3-4	10-1	28.4	4-2	15-6	55.9
3-5	10-10	31.4	4-3	11-2	39.9
3-5	15-3	43.8	4-3	19-4	67.9
3-6	11-6	34.5	4-4	11-10	43.5
3-6	16-0	47.3	4-4	16-2	60.1
3-8	12-2	37.7	4-5	12-6	47.3
3-8	16-8	51.1	4-6	13-2	51.2

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

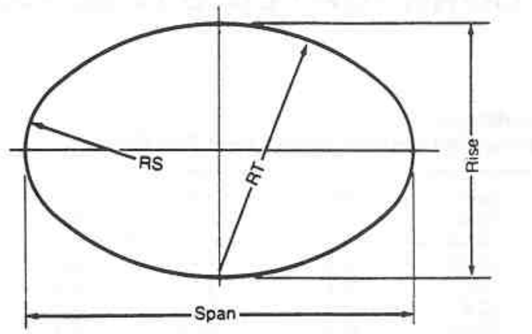
STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

Continued.
Layout Details Corrugated Steel Box Culverts

Rise, ft-in.	Span, ft-in.	Area ft²	Rise, ft-in.	Span, ft-in.	Area ft²
4-6	16-10	64.4	6-9	13-7	77.9
4-7	17-6	68.9	6-9	16-9	99.3
4-7	20-8	77.6	6-10	14-2	83.3
4-8	13-10	55.3	6-10	17-4	105.1
4-9	14-6	59.5	7-0	14-9	88.9
4-9	18-1	73.5	7-0	17-11	111.1
4-10	15-1	63.8	7-0	20-8	127.2
4-11	11-0	44.7	7-1	15-4	94.6
4-11	18-9	78.4	7-2	18-6	117.3
5-0	11-7	48.7	7-3	12-3	71.5
5-0	15-9	68.3	7-3	15-10	100.5
5-1	12-3	52.9	7-4	12-10	77.1
5-1	16-4	73.0	7-4	16-5	106.5
5-1	19-5	83.4	7-4	19-1	123.6
5-2	12-10	57.2	7-5	13-5	82.8
5-3	17-0	77.8	7-6	13-11	88.6
5-4	13-6	61.7	7-6	17-0	112.7
5-5	14-1	66.2	7-8	14-6	94.5
5-5	17-7	82.8	7-8	17-6	119.0
5-5	20-8	94.1	7-9	15-0	100.6
5-6	14-9	71.0	7-9	18-1	125.5
5-7	18-3	88.0	7-11	15-7	106.8
5-8	11-5	53.3	7-11	18-7	132.1
5-8	15-4	75.8	8-0	12-8	81.1
5-8	18-10	93.4	8-0	16-1	113.1
5-9	12-0	57.9	8-1	19-2	138.9
5-9	16-0	80.9	8-2	16-8	119.6
5-10	12-7	62.6	8-2	13-9	93.3
5-10	19-6	98.9	8-3	19-8	145.9
5-11	16-7	86.1	8-4	17-2	126.2
6-0	13-3	67.4	8-5	14-10	106.0
6-1	13-10	72.4	8-5	17-8	133.0
6-1	17-2	91.4	8-7	18-3	139.9
6-2	14-5	77.5	8-7	20-9	160.3
6-2	17-9	96.9	8-8	15-10	119.2
6-2	20-8	110.6	8-9	18-9	147.0
6-4	15-0	82.7	8-11	16-10	132.9
6-4	18-4	102.6	8-11	19-3	154.2
6-5	11-10	62.2	9-1	19-9	161.6
6-5	15-7	88.1	9-3	17-10	147.1
6-6	18-11	108.5	9-5	20-9	176.9
6-7	12-5	67.3	9-6	18-10	162.0
6-7	16-2	93.6	9-10	19-10	177.4
6-8	13-0	72.5	10-2	20-9	193.5
6-8	19-6	114.5			

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

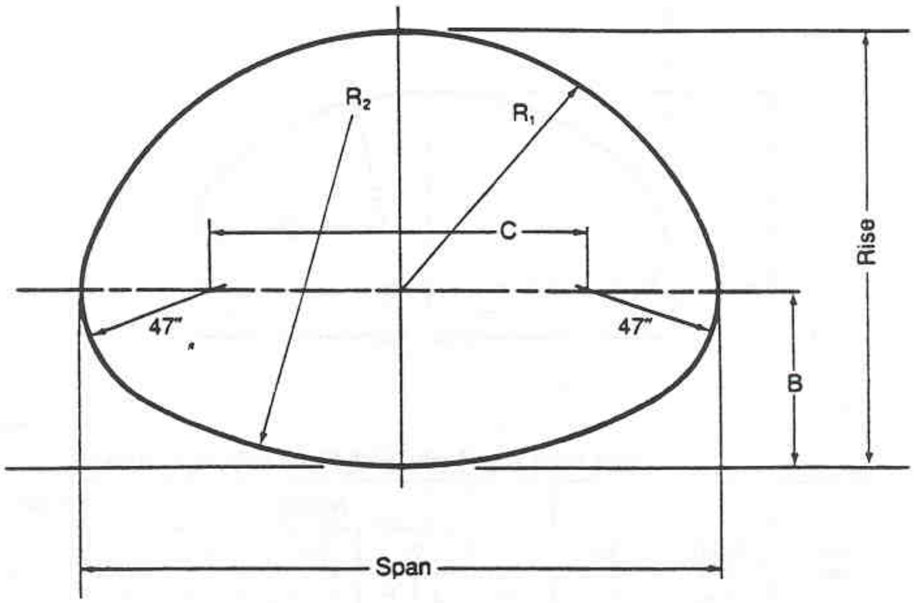


Long Span Horizontal Ellipse Sizes and Layout Details

Span, ft-in.	Rise, ft-in.	Area, ft²	Periphery: Top or Bottom N	Periphery: Top or Bottom Pi	Periphery: Side N	Periphery: Side Pi	Periphery: Total N	Periphery: Total Pi	Inside Radius: Top Rad. in.	Inside Radius: Side Rad. in.
19- 4	12- 9	191	22	66	10	30	64	192	12- 6	4- 6
20- 1	13- 0	202	23	69	10	30	66	198	13- 1	4- 6
20- 2	11-11	183	24	72	8	24	64	192	13- 8	3- 7
20-10	12- 2	194	25	75	8	24	66	198	14- 3	3- 7
21- 0	15- 2	248	23	69	13	39	72	216	13- 1	5-11
21-11	13- 1	221	26	78	9	27	70	210	14-10	4- 1
22- 6	15- 8	274	25	75	13	39	76	228	14- 3	5-11
23- 0	14- 1	249	27	81	10	30	74	222	15- 5	4- 6
23- 3	15-11	288	26	78	13	39	78	234	14-10	5-11
24- 4	16-11	320	27	81	14	42	82	246	15- 5	6- 4
24- 6	14- 8	274	29	87	10	30	78	234	16- 6	4- 6
25- 2	14-11	287	30	90	10	30	80	240	17- 1	4- 6
25- 5	16- 9	330	29	87	13	39	84	252	16- 6	5-11
26- 1	18- 2	369	29	87	15	45	88	264	16- 6	6-10
26- 3	15-10	320	31	93	11	33	84	252	17- 8	4-11
27- 0	16- 2	334	32	96	11	33	86	258	18- 3	4-11
27- 2	19- 1	405	30	90	16	48	92	276	17- 1	7- 3
27-11	19- 5	421	31	92	16	48	94	282	17- 8	7- 3
28- 1	17- 1	369	33	99	12	36	90	270	18-10	5- 5
28-10	17- 5	384	34	102	12	36	92	276	19- 5	5- 5
29- 5	19-11	455	33	99	16	48	98	294	18-10	7- 3
30- 1	20- 2	472	34	102	16	48	100	300	19- 5	7- 3
30- 3	17-11	415	36	108	12	36	96	288	20- 7	5- 5
31- 2	21- 2	512	35	105	17	51	104	312	20- 0	7- 9
31- 4	18-11	454	37	111	13	39	100	300	21- 1	5-11
32- 1	19- 2	471	38	114	13	39	102	306	21- 8	5-11
32- 3	22- 2	555	36	108	18	54	108	324	20- 7	8- 2
33- 0	22- 5	574	37	111	18	54	110	330	21- 1	8- 2
33- 2	20- 1	512	39	117	14	42	106	318	22- 3	6- 4
34- 1	23- 4	619	38	114	19	57	114	342	21- 8	8- 8
34- 7	20- 8	548	41	123	14	42	110	330	23- 5	6- 4
34-11	21- 4	574	41	123	15	45	112	336	23- 5	6-10
35- 1	24- 4	665	39	117	20	60	118	354	22- 3	9- 1
35- 9	25- 9	718	39	117	22	66	122	366	22- 3	10- 0
36- 0	22- 4	619	42	126	16	48	116	348	24- 0	7- 3
36-11	25- 7	735	41	123	21	63	124	372	23- 5	9- 7
37- 2	22- 2	631	44	132	15	45	118	354	25- 2	6-10
38- 0	26- 7	785	44	132	22	66	128	384	24- 0	10- 0
38- 8	27-11	843	42	126	24	72	132	396	24- 0	10-11
40- 0	29- 7	927	43	129	26	78	138	414	27-11	11-10

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES FOR CORRUGATED STEEL CULVERTS



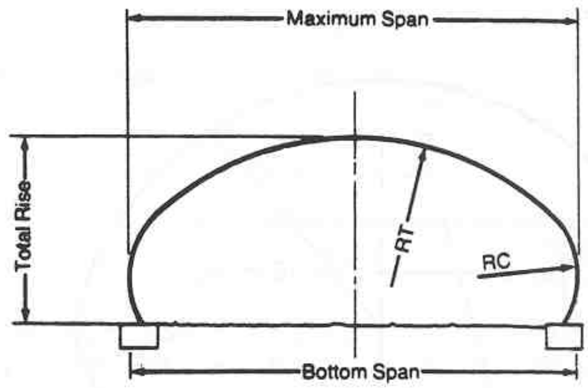
Long Span Pipe Arch Sizes and Layout Details

Span, ft-in.	Rise, ft-in.	Area, ft²	Total No. Plates	Periphery						B, in.	C, in.	Inside Radius	
				Top		Bottom		Total				R1, in.	R2, in.
				N	Pi	N	Pi	N	Pi				
20- 0	13-11	218	10	34	102	20	60	68	204	62.8	146.2	122.5	223.6
20- 6	14- 3	231	10	36	108	20	60	70	210	61.4	152.3	124.7	255.7
21- 5	14- 6	243	11	36	108	22	66	72	216	65.3	162.8	131.4	236.7
21-11	14-11	256	11	38	114	22	66	74	222	63.7	168.9	133.5	268.1
22- 5	15- 3	270	11	40	120	22	66	76	228	62.1	174.6	135.5	307.1
23- 4	15- 7	284	11	40	120	24	72	78	234	66.2	185.5	142.4	280.2
24- 2	15-11	297	12	40	120	26	78	80	240	70.7	196.2	149.7	262.1
24- 8	16- 2	312	12	42	126	26	78	82	246	68.8	202.2	151.4	292.2
25- 2	16- 7	326	12	44	132	26	78	84	252	66.9	207.9	153.2	328.6
25- 7	16-11	342	12	46	138	26	78	86	258	64.8	213.3	155.0	373.3
26- 7	17- 3	357	12	46	138	28	84	88	264	69.4	224.7	162.1	339.4
27- 6	17- 6	372	12	46	138	30	90	90	270	74.2	235.8	169.6	315.8
28- 0	17-10	388	12	48	144	30	90	92	276	72.1	241.5	171.1	350.2
28- 5	18- 3	405	13	50	150	30	90	94	282	69.9	246.8	172.7	392.3
29- 4	18- 6	421	13	50	150	32	96	96	288	74.8	258.2	180.2	361.1
30- 4	18-10	438	14	52	156	34	102	100	300	80.0	269.4	188.2	339.1

*Includes 14N for two N7 corner plates

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



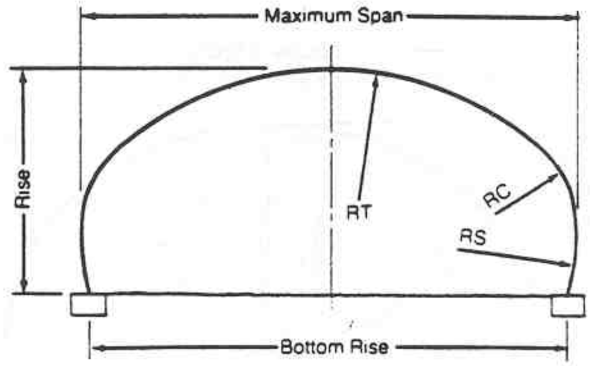
Long Span Low Profile Arch Sizes and Layout Details

Max. Span, ft-in.	Bottom Span, ft-in.	Total Rise, ft-in.	Area, ft²	Periphery Top N	Periphery Top Pi	Periphery Side N	Periphery Side Pi	Periphery Total N	Periphery Total Pi	Inside Radius Top rad. in.	Inside Radius Side rad. in.
20- 1	19-10	7- 6	121	23	69	6	18	35	105	13- 1	4- 6
19- 5	19- 1	6-10	105	23	69	5	15	33	99	13- 1	3- 7
21- 6	21- 4	7- 9	134	25	75	6	18	37	111	14- 3	4- 6
22- 3	22- 1	7-11	140	26	78	6	18	38	114	14-10	4- 6
23- 0	22- 9	8- 0	147	27	81	6	18	39	117	15- 5	4- 6
23- 9	23- 6	8- 2	154	28	84	6	18	40	120	16- 0	4- 6
24- 6	24- 3	8- 4	161	29	87	6	18	41	123	16- 6	4- 6
25- 2	25- 0	8- 5	169	30	90	6	18	42	126	17- 1	4- 6
25-11	25- 9	8- 7	176	31	93	6	18	43	129	17- 8	4- 6
27- 3	27- 1	10- 0	217	31	93	8	24	47	141	17- 8	6- 4
28- 1	27-11	9- 7	212	33	99	7	21	47	141	18-10	5- 5
28- 9	28- 7	10- 3	234	33	99	8	24	49	147	18-10	6- 4
28-10	28- 8	9- 8	221	34	102	7	21	48	144	19- 5	5- 5
30- 3	30- 1	9-11	238	36	108	7	21	50	150	20- 7	5- 5
30-11	30- 9	10- 8	261	36	108	8	24	52	156	20- 7	6- 4
31- 7	31- 2	12- 1	309	36	108	10	30	56	168	20- 7	7- 3
31- 0	30-10	10- 1	246	37	111	7	21	51	153	21- 1	5- 5
32- 4	31-11	12- 3	320	37	111	10	30	57	171	21- 1	7- 3
31- 9	31- 7	10- 3	255	38	114	7	21	52	156	21- 8	5- 5
33- 1	32- 7	12- 5	330	38	114	10	30	58	174	21- 8	7- 3
33- 2	33- 0	11- 1	289	39	117	8	24	55	165	22- 3	6- 4
34- 5	34- 1	13- 3	377	39	117	11	33	61	183	22- 3	8- 2
34- 7	34- 6	11- 4	308	41	123	8	24	57	183	23- 5	6- 4
37-11	37- 7	15- 8	477	41	123	14	42	69	207	23- 5	10-11
35- 4	35- 2	11- 5	318	42	126	8	24	58	174	24- 0	6- 4
38- 8	38- 4	15- 9	490	42	126	14	42	70	210	24- 0	10-11

NOTE: Larger sizes available for special designs.

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS

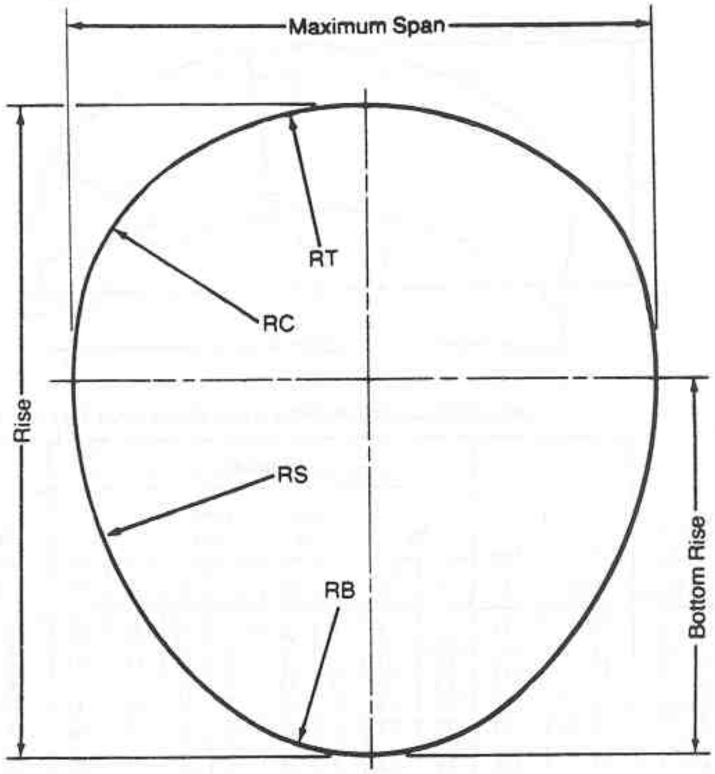


Long Span High Profile Arch Sizes and Layout Details

Max. Span, ft-in.	Bottom Span, ft-in.	Total Rise, ft-in.	Area, ft²	Periphery								Inside Radius		
				Top		Upper Side		Lower Side		Total		Top Radius, ft-in.	Upper Side, ft-in.	Lower Side, ft-in.
				N	Pi	N	Pi	N	Pi	N	Pi			
20- 1	19- 6	9- 1	152	23	69	5	15	3	9	39	117	13- 1	4- 6	13- 1
20- 8	18-10	12- 1	214	23	69	6	18	6	18	47	141	13- 1	5- 5	13- 1
21- 6	19-10	11- 8	215	25	75	5	15	6	18	47	141	14- 3	4- 6	14- 3
22-10	19-10	14- 7	285	25	75	7	21	8	24	55	165	14- 3	6- 4	14- 3
22- 3	20- 7	11-10	225	26	78	5	15	6	18	48	144	14-10	4- 6	14-10
22-11	20- 0	14- 0	276	26	78	6	18	8	24	54	162	14-10	5- 5	14-10
23- 0	21- 5	12- 0	235	27	81	5	15	6	18	49	147	15- 5	4- 6	15- 5
24- 4	21- 6	14-10	310	27	81	7	21	8	24	57	171	15- 5	6- 4	15- 5
23- 9	22- 2	12- 1	245	28	84	5	15	6	18	50	150	16- 0	4- 6	16- 0
24- 6	21-11	13- 9	289	29	87	5	15	8	24	55	165	16- 6	4- 6	16- 6
25- 9	23- 2	15- 2	335	29	87	7	21	8	24	59	177	16- 6	6- 4	16- 6
25- 2	23- 3	13- 2	283	30	90	5	15	7	21	54	162	17- 1	4- 6	17- 1
26- 6	24- 0	15- 3	348	30	90	7	21	8	24	60	180	17- 1	6- 4	17- 1
25-11	24- 1	13- 3	295	31	93	5	15	7	21	55	165	17- 8	4- 6	17- 8
27- 3	24-10	15- 5	360	31	93	7	21	8	24	61	183	17- 8	6- 4	17- 8
27- 5	25- 8	13- 7	317	33	99	5	15	7	21	57	171	18-10	4- 6	18-10
29- 5	27- 1	16- 5	412	33	99	8	28	8	24	65	195	18-10	7- 3	18-10
28- 2	25-11	14- 5	349	34	102	5	15	8	24	60	180	19- 5	4- 6	19- 5
30- 1	26- 9	18- 1	467	34	102	8	24	10	30	70	210	19- 5	7- 3	19- 5
30- 3	28- 2	15- 5	399	36	108	6	18	8	24	64	192	20- 7	5- 5	20- 7
31- 7	28- 4	18- 4	497	36	108	8	24	10	30	72	216	20- 7	7- 3	20- 7
31- 0	29- 0	15- 7	413	37	111	6	18	8	24	65	195	21- 1	5- 5	21- 1
31- 8	28- 6	17- 9	484	37	111	7	21	10	30	71	213	21- 1	6- 4	21- 1
32- 4	27-11	19-11	554	37	111	8	24	12	36	77	231	21- 1	7- 3	21- 1
31- 9	28- 8	17- 3	470	38	114	6	18	10	30	70	210	21- 8	5- 5	21- 8
33- 1	28- 9	20- 1	571	38	114	8	24	12	36	78	234	21- 8	7- 3	21- 8
32- 6	29- 6	17- 4	484	39	117	6	18	10	30	71	213	22- 3	5- 5	22- 3
33-10	29- 7	20- 3	588	39	117	8	24	12	36	79	237	22- 3	7- 3	22- 3
34- 0	31- 2	17- 8	514	41	123	6	18	10	30	73	219	23- 5	5- 5	23- 5
34- 7	30- 7	19-10	591	41	123	7	21	12	36	79	237	23- 5	6- 4	23- 5
35- 3	30- 7	21- 3	645	41	123	8	24	13	39	83	249	23- 5	7- 3	23- 5
37- 3	32- 6	23- 5	747	41	123	11	33	13	39	89	267	23- 5	10- 0	23- 5
34- 8	31-11	17-10	529	42	126	6	18	10	30	74	222	24- 0	5- 5	24- 0
35- 4	31- 5	20- 0	608	42	126	7	21	12	36	80	240	24- 0	6- 4	24- 0
36- 0	31- 5	21- 5	663	42	126	8	24	13	39	84	252	24- 0	7- 3	24- 0
38- 0	33- 5	23- 6	767	42	126	11	33	13	39	90	270	24- 0	10- 0	24- 0

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES
FOR CORRUGATED STEEL CULVERTS



Long Span Pear Shape Sizes and Layout Details

Max. Span, ft-in.	Rise, ft-in.	Rise Bottom, ft-in.	Area	Periphery										Inside Radius			
				Top		Corner		Side		Bottom		Total		Bottom Radius, ft-in.	Side Radius, ft-in.	Corner Radius, ft-in.	Top Radius, ft-in.
				N	Pi	N	Pi	N	Pi	N	Pi	N	Pi				
23- 8	25- 8	14-11	481	25	75	5	15	24	72	15	30	98	294	8-11	16- 7	6- 3	14- 8
24- 0	25-10	15- 1	496	22	66	7	21	22	66	20	60	100	300	9-11	17- 4	7- 0	16- 2
25- 6	25-11	15-10	521	27	81	7	21	20	60	21	63	102	306	10- 7	18- 1	6-11	15-10
24-10	27- 8	16- 9	544	27	81	5	15	25	75	18	54	105	315	9- 3	19- 8	5- 9	15-11
27- 5	27- 0	18- 1	578	30	90	6	18	26	78	16	48	110	330	9- 7	20- 4	4- 7	19-11
26- 8	28- 3	18- 0	593	28	84	5	15	30	90	12	36	110	330	8- 0	20- 1	4- 9	20-11
28- 1	27-10	16-10	624	27	81	8	24	22	66	25	75	112	336	12- 2	19- 0	7- 3	20- 5
28- 7	30- 7	19- 7	689	32	96	7	21	24	72	24	72	118	354	11- 2	24- 0	7- 0	18- 2
30- 0	29- 8	20- 0	699	32	96	8	24	23	69	25	75	119	357	11-11	24- 0	6- 7	21-10
30- 0	31- 2	19-11	736	34	102	7	21	24	72	26	78	122	366	12- 1	24- 0	7- 0	19- 3

Figure 14.3.47 Standard Sizes for Corrugated Steel Culverts (Source: American Iron and Steel Institute), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

Helical Pipe Availability, Weights

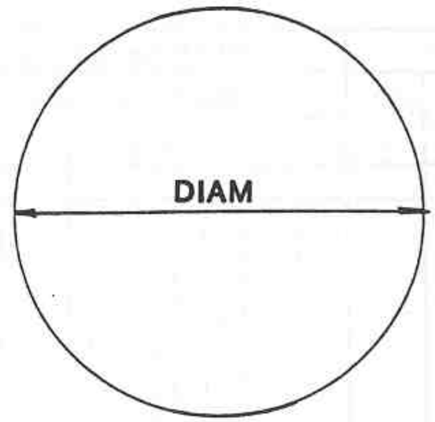
CORR. PATTERN				WEIGHT (Lbs./Lineal Ft.)					
				Equiv. Standard Gauge					
1-1/2 x 1/4	2-2/3 x 1/2	3 x 1	6 x 1	18	16	14	12	10	8
Diameter (In.)									
6				1.4	1.7				
8				1.8	2.2				
10				2.2	2.7				
	12				3.2	4.0	5.5		
	15				3.9	4.9	6.8		
	18				4.7	5.9	8.1		
	21				5.4	6.8	9.4		
	24				6.2	7.8	10.7	13.8	
	27				7.0	8.7	12.1	15.4	
	30				7.8	9.6	13.4	17.1	
		30			8.9	11.2	15.5	19.9	
	36					11.5	16.0	20.5	
		36			10.7	13.4	18.5	23.7	
	42						18.6	23.8	
		42			12.4	15.5	21.5	27.5	
	48						21.2	27.2	32.7
		48			14.1	17.7	24.5	31.4	37.8
			48		12.5	15.6	21.8	28.1	34.1
	54						23.8	30.5	36.7
		54			15.8	19.9	27.5	35.2	42.4
			54		14.0	17.5	24.5	31.5	38.3
	60							33.9	40.8
		60			17.6	22.0	30.5	39.0	47.0
			60		15.5	19.4	27.2	34.9	42.5
	66							37.2	44.8
		66			17.0	21.3	29.8	38.4	46.6
	72								48.8
		72				26.3	36.5	46.7	56.2
			72			23.2	32.5	41.8	50.8
	78								52.9
		78				28.5	39.5	50.5	60.8
			78			25.1	35.2	45.2	55.0
	84								56.9
		84				30.7	42.5	54.3	65.4
			84				37.8	48.7	59.1
		90					45.4	58.2	70.0
			90				40.5	52.1	63.3
		96					48.4	62.0	74.6
			96				43.2	55.5	67.5
		102					51.4	65.8	79.3
			102				45.8	58.9	71.6
		108					54.4	69.7	83.9
			108				48.5	62.4	75.8
		114					57.4	73.5	88.5
			114				51.2	65.8	80.0
		120					60.4	77.3	93.1
			120				53.8	69.2	84.1

NOTES: 1. Sizes 6" thru 10" are available in helical corrugation only.

2. Sizes 12" through 21" in helical configuration have corrugation depth of 7/16" rather than 1/2".

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association)

STANDARD SIZES FOR ALUMINUM CULVERTS



Geometric Data — Structural Plate Pipe

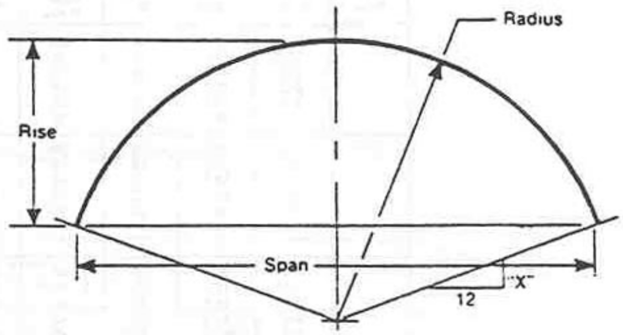
Nom. Diam. In.	Area Sq. Ft.	Total N	Nom. Diam. In.	Area Sq. Ft.	Total N
60	19	20	162	145	54
66	23	22	168	156	56
72	27	24	174	167	58
78	32	26	180	179	60
84	38	28	186	191	62
90	44	30	192	204	64
96	50	32	198	217	66
102	56	34	204	231	68
108	63	36	210	245	70
114	71	38	216	259	72
120	79	40	222	274	74
126	87	42	228	289	76
132	95	44	234	305	78
138	104	46	240	321	80
144	114	48	246	337	82
150	124	50	252	354	84
156	134	52	—	—	—

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

GEOMETRIC DATA – ARCH

"X" Values For Rise/Span Ratio			
R/S Ratio	"X"	R/S Ratio	"X"
.30	6.40	.42	2.10
.31	5.96	.43	1.82
.32	5.54	.44	1.54
.33	5.13	.45	1.27
.34	4.74	.46	1.00
.35	4.37	.47	.74
.36	4.01	.48	.48
.37	3.67	.49	.24
.38	3.33	.50	.00
.39	3.01	.51	.24
.40	2.70	.52	.47
.41	2.40		



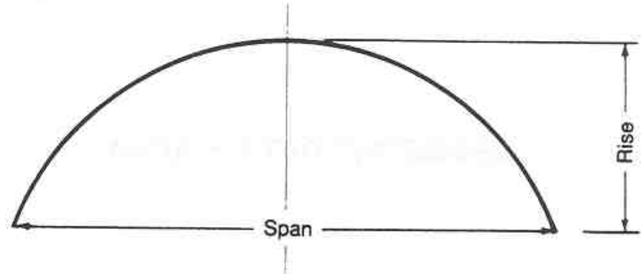
Typical Section

Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
5-0	2-7	10.4	10	.52	30
	2-3	8.5	9	.44	30¼
	1-9	6.5	8	.36	31¼
6-0	3-2	14.9	12	.52	36
	2-9	12.6	11	.46	36¼
	2-4	10.2	10	.38	37¼
	1-10	7.8	9	.30	40½
7-0	3-8	20.3	14	.52	42
	3-3	17.5	13	.46	42
	2-10	14.8	12	.40	43
	2-4	12.0	11	.34	45¼
8-0	4-2	26.4	16	.52	48
	3-9	23.3	15	.47	48
	3-4	20.2	14	.42	48¼
	2-11	17.0	13	.36	50½
	2-5	13.6	12	.30	54½

Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
9-0	4-8	33.4	18	.50	54
	4-3	29.9	17	.48	54
	3-10	26.3	16	.43	54½
	3-5	22.8	15	.38	56
	2-11	19.1	14	.33	59
10-0	5-2	41.2	20	.52	60
	4-10	37.3	19	.48	60
	4-5	33.3	18	.44	60½
	3-11	29.4	17	.40	61½
	3-6	25.3	16	.35	64
	3-0	21.1	15	.30	68½
11-0	5-8	49.8	22	.52	66
	5-4	45.5	21	.48	66
	4-11	41.2	20	.45	66½
	4-6	36.8	19	.41	67½
	4-0	32.4	18	.36	69¼
	3-6	27.8	17	.32	72¾

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS



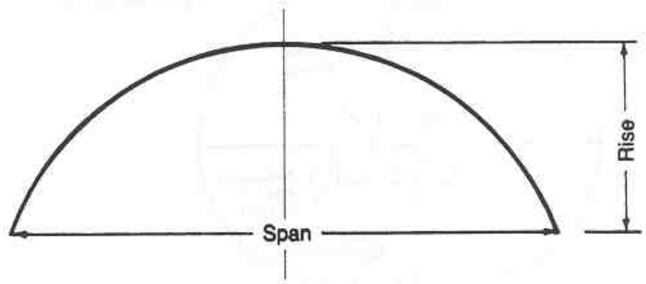
ARCH (CONT'D)

Geometric Data—Arch (Continued)

Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
12-0	6-3	59.3	24	.52	72
	5-10	54.5	23	.49	72
	5-5	49.8	22	.45	72½
	5-0	45.0	21	.42	73¼
	4-7	40.2	20	.38	75
	4-1	35.3	19	.34	77½
13-0	6-9	69.5	26	.52	78
	6-4	64.4	25	.49	78
	5-11	59.3	24	.46	78¼
	5-6	54.1	23	.42	79
	5-1	48.9	22	.39	80½
	4-7	43.6	21	.35	82¾
	4-1	38.1	20	.31	86½
14-0	7-3	80.6	28	.52	84
	6-10	75.1	27	.49	84
	6-5	69.5	26	.46	84¼
	6-0	64.0	25	.43	85
	5-7	58.4	24	.40	86
	5-2	52.7	23	.37	88
	4-8	46.9	22	.33	91¼
15-0	7-9	92.5	30	.52	90
	7-5	86.5	29	.49	90
	7-0	80.6	28	.46	90¼
	6-7	74.7	27	.44	91
	6-1	68.7	26	.41	92
	5-8	62.6	25	.38	93½
	5-2	56.4	24	.34	96¼
	4-8	50.0	23	.31	100½
16-0	8-3	105.2	32	.52	96
	7-11	98.9	31	.49	96
	7-6	92.5	30	.47	96¼
	7-1	86.2	29	.44	96¾
	6-8	79.8	28	.41	97¾
	6-2	73.3	27	.39	99¼
	5-9	66.8	26	.36	101½
	5-3	60.0	25	.32	105
17-0	8-10	118.7	34	.52	102
	8-5	112.0	33	.49	102
	8-0	105.2	32	.47	102¼
	7-7	98.5	31	.45	102¾
	7-2	91.7	30	.42	103½
	6-9	84.9	29	.39	105
	6-3	77.9	28	.37	107
	5-9	70.9	27	.34	110
	5-3	63.5	26	.31	114¼
18-0	9-4	133.1	36	.52	108
	8-11	125.9	35	.50	108
	8-6	118.8	34	.47	108¼
	8-1	111.6	33	.45	108½
	7-8	104.5	32	.43	109¼
	7-3	97.2	31	.40	110½
	6-9	89.9	30	.38	112½
	6-4	82.5	29	.35	115
	5-9	74.8	28	.32	118¾
19-0	9-10	148.2	38	.52	114
	9-5	140.7	37	.50	114
	9-0	133.2	36	.48	114¼
	8-8	125.6	35	.45	114½
	8-2	118.0	34	.43	115¼
	7-9	110.4	33	.41	116¼
	7-4	102.7	32	.38	118
	6-10	94.9	31	.36	120¼
	6-4	86.9	30	.33	123½
	5-10	78.7	29	.31	128¼
20-0	10-4	164.2	40	.52	120
	10-0	156.3	39	.50	120
	9-7	148.3	38	.48	120
20-0 cont.	9-2	140.4	37	.46	120½
	8-9	132.4	36	.44	121
	8-3	124.4	35	.41	122¼
	7-10	116.3	34	.39	123½
	7-4	108.4	33	.37	125¾
	6-10	99.8	32	.34	128½
	6-4	91.2	31	.32	132½
21-0	10-10	181.0	42	.52	126
	10-6	172.7	41	.50	126
	10-1	164.3	40	.48	126
	9-8	156.0	39	.46	126½
	9-3	147.6	38	.44	127
	8-10	139.2	37	.42	128
	8-4	130.7	36	.40	129¼
	7-11	122.2	35	.38	131¼
	7-5	113.5	34	.35	133¾
	6-11	104.6	33	.33	137¼
	6-4	95.4	32	.30	142
22-0	11-5	198.6	44	.52	132
	11-0	189.9	43	.50	132
	10-7	181.1	42	.48	132
	10-2	172.4	41	.46	132½
	9-9	163.6	40	.44	133
	9-4	154.8	39	.42	133¾
	8-11	146.0	38	.40	135
	8-5	137.0	37	.38	135¾
	7-11	127.9	36	.36	139
	7-5	118.7	35	.34	142¼
	6-11	109.2	34	.31	142¼
23-0	11-11	217.1	46	.52	138
	11-6	207.9	45	.50	138
	11-1	198.8	44	.48	138
	10-8	189.5	43	.47	138¼
	10-3	180.5	42	.45	139
	9-10	171.3	41	.43	139½
	9-5	162.0	40	.41	140¾
	8-11	152.7	39	.39	142¼
	8-6	143.2	38	.37	144½
	8-0	133.6	37	.35	147¼
	7-6	123.6	36	.33	151
	6-11	113.8	35	.30	156
24-0	12-5	236.3	48	.52	144
	12-0	226.8	47	.50	144
	11-7	217.2	46	.48	144
	11-3	207.7	45	.47	144¼
	10-10	198.1	44	.45	144¾
	10-4	188.5	43	.43	145½
	9-11	178.9	42	.41	146½
	9-6	169.2	41	.39	148
	9-0	159.3	40	.38	150
	8-6	149.4	39	.36	152½
	8-0	139.2	38	.33	155¾
	7-6	128.9	37	.31	160¼
25-0	12-11	256.4	50	.52	150
	12-6	246.4	49	.50	150
	12-2	236.5	48	.49	150
	11-9	226.6	47	.47	150¼
	11-4	216.6	46	.45	150¾
	10-11	206.6	45	.44	151½
	10-5	196.6	44	.42	152½
	10-0	186.4	43	.40	153¾
	9-6	176.3	42	.38	155½
	9-1	165.9	41	.36	157¾
	8-7	155.4	40	.34	160¾
	8-1	144.7	39	.32	164¾
	7-6	133.7	38	.30	170
26-0	13-5	277.3	52	.52	156
	13-1	266.9	51	.50	156
	12-8	256.6	50	.49	156

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS



ARCH (CONT'D)

Geometric Data—Arch (Continued)

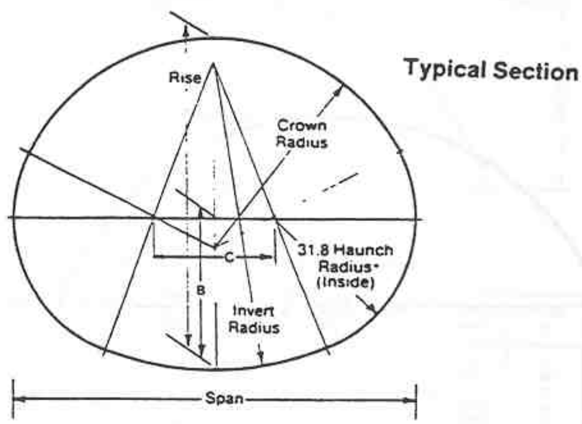
Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
26-0 cont.	12-3	246.2	49	.47	156¼
	11-10	235.9	48	46	156¾
	11-5	225.5	47	44	157¼
	11-0	215.1	46	.42	158¼
	10-6	204.6	45	40	159½
	10-1	194.0	44	.39	161
	9-7	183.3	43	.37	163¼
	9-1	172.4	42	.35	166
	8-7	161.4	41	33	169½
	8-1	150.1	40	.31	174
27-0	14-0	299.0	54	.52	162
	13-7	288.2	53	.50	162
	13-2	277.5	52	49	162
	12-9	266.7	51	.47	162¼
	12-4	256.0	50	46	162¾
	11-11	245.2	49	44	163¼
	11-6	234.4	48	43	164
	11-1	223.5	47	41	165¼
	10-7	212.6	46	39	166¾
	10-2	201.4	45	38	168¾
	9-8	190.2	44	.36	171¼
	9-2	178.8	43	34	174½
	8-7	167.2	42	.32	178½
	8-1	155.3	41	.30	183¾
28-0	14-6	321.5	56	.52	168
	14-1	310.4	55	.50	168
	13-8	299.2	54	49	168
	13-3	288.1	53	47	168¼
	12-10	276.9	52	46	168½
	12-5	265.7	51	44	169¼
	12-0	254.5	50	43	170
	11-7	243.2	49	41	171
	11-1	231.9	48	40	172½
	10-8	220.4	47	38	174¼

Span Ft.In.	Rise Ft.In.	Area Sq.Ft.	Total N	Rise/ Span Ratio	Radius Inches
28-0 cont.	10-2	208.8	46	.36	176½
	9-8	197.1	45	.35	179½
	9-2	185.1	44	.33	183¼
	8-8	172.9	43	.31	188
29-0	15-0	344.8	58	.52	174
	14-7	333.3	57	.50	174
	14-2	321.7	56	.49	174
	13-10	310.2	55	.48	174¼
	13-5	298.6	54	.46	174½
	13-0	287.1	53	.45	175
	12-6	275.4	52	.43	175¾
	12-1	263.8	51	42	176¾
	11-8	252.0	50	40	178¼
	11-2	240.2	49	.39	180
	10-9	228.2	48	.37	182
	10-3	216.1	47	.35	184¾
	9-9	203.8	46	.34	188
	9-2	191.3	45	.32	192¼
	8-8	178.5	44	.30	197¾
30-0	15-6	369.0	60	.52	180
	15-1	357.1	59	50	180
	14-9	345.1	58	49	180
	14-4	333.2	57	48	180¼
	13-11	321.2	56	46	180½
	13-6	309.2	55	45	181
	13-1	297.2	54	44	181¾
	12-7	285.1	53	.42	182¾
	12-2	273.0	52	.41	184
	11-9	260.8	51	39	185½
	11-3	248.5	50	.37	187½
	10-9	236.0	49	.36	190
	10-3	223.3	48	34	193
	9-9	210.5	47	32	197
	9-2	197.3	46	31	201¾

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

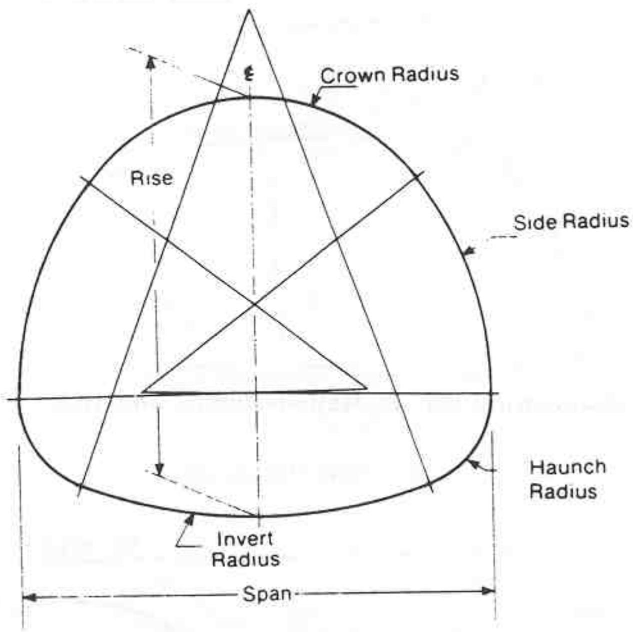
Geometric Data—Pipe Arch



Span Ft.-In.	Rise Ft.-In.	Area Sq. Ft.	Required N				Inside Radius		B	C
			Total	Crown	Invert	Haunch	Crown In.	Invert In.		
6-7	5-8	29.6	25	8	3	7	41.5	69.9	32.5	15.3
6-11	5-9	31.9	26	9	3	7	43.7	102.9	32.4	19.6
7-3	5-11	34.3	27	10	3	7	45.6	188.3	32.2	23.8
7-9	6-0	36.8	28	9	5	7	51.6	83.8	33.8	29.0
8-1	6-1	39.3	29	10	5	7	53.3	108.1	33.5	33.3
8-5	6-3	41.9	30	11	5	7	54.9	150.1	33.2	37.4
8-10	6-4	44.5	31	10	7	7	63.3	93.0	35.6	42.8
9-3	5-5	47.1	32	11	7	7	64.4	112.6	35.2	47.1
9-7	6-6	49.9	33	12	7	7	65.4	141.6	34.7	51.3
9-11	6-8	52.7	34	13	7	7	66.4	188.7	34.2	55.3
10-3	6-9	55.5	35	14	7	7	67.4	278.8	33.5	59.2
10-9	6-10	58.4	36	13	9	7	77.5	139.6	36.8	65.2
11-1	7-0	61.4	37	14	9	7	77.8	172.0	36.1	69.3
11-5	7-1	64.4	38	15	9	7	78.2	222.0	35.3	73.3
11-9	7-2	67.5	39	16	9	7	78.7	309.5	34.4	77.1
12-3	7-3	70.5	40	15	11	7	90.8	165.2	38.4	83.4
12-7	7-5	73.7	41	16	11	7	90.5	200.0	37.5	87.4
12-11	7-6	77.0	42	17	11	7	90.4	251.7	36.5	91.3
13-1	8-2	83.0	43	18	13	6	88.8	143.6	42.0	93.6
13-1	8-4	86.8	44	21	11	6	81.7	300.8	35.8	93.7
13-11	8-5	90.3	45	18	15	6	100.4	132.0	46.0	103.3
14-0	8-7	94.2	46	21	13	6	90.3	215.1	39.4	104.5
13-11	9-5	101.5	47	23	14	5	86.2	159.3	42.8	103.9
14-3	9-7	105.7	48	24	14	5	87.2	176.3	42.0	107.0
14-8	9-8	109.9	49	24	15	5	90.9	166.2	44.0	112.3
14-11	9-10	114.2	50	25	15	5	91.8	183.0	43.2	115.5
15-4	10-0	118.6	51	25	16	5	95.5	173.0	45.3	120.8
15-7	10-2	123.1	52	26	16	5	96.4	189.6	44.4	123.9
16-1	10-4	127.6	53	26	17	5	100.2	179.7	46.6	129.2
16-4	10-6	132.3	54	27	17	5	101.0	196.1	45.7	132.3
16-9	10-8	136.9	55	27	18	5	105.0	186.3	47.9	137.7
17-0	10-10	141.8	56	28	18	5	105.7	202.5	46.9	140.8
17-3	11-0	146.7	57	29	18	5	106.5	221.3	45.9	143.8
17-9	11-2	151.6	58	29	19	5	110.4	208.9	48.2	149.3
18-0	11-4	156.7	59	30	19	5	111.1	227.3	47.2	152.3
18-5	11-6	161.7	60	30	20	5	115.2	215.2	49.6	157.8
18-8	11-8	167.0	61	31	20	5	115.8	233.3	48.5	160.7
19-2	11-9	172.2	62	31	21	5	119.9	221.5	50.9	166.2
19-5	11-11	177.6	63	32	21	5	120.5	239.3	49.8	169.2
19-10	12-1	182.9	64	32	22	5	124.7	227.7	52.3	174.8
20-1	12-3	188.5	65	33	22	5	125.2	245.3	51.1	177.7
20-1	12-6	194.4	66	35	21	5	122.5	310.8	46.2	177.5
20-10	12-7	199.7	67	34	23	5	130.0	251.2	52.5	186.2
21-1	12-9	205.5	68	35	23	5	130.5	270.9	51.2	189.1
21-6	12-11	211.2	69	35	24	5	134.8	257.2	53.9	194.8

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS



Typical Section

Geometric Data—Vehicular Underpass

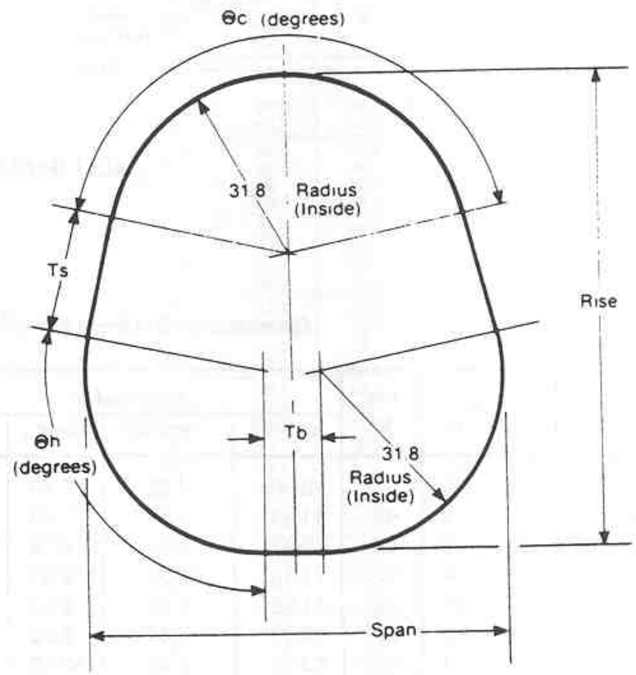
Span Ft	Span In.	Rise Ft	Rise In.	Tot N	Required N Invert	Required N Haunch	Required N Side	Required N Crown	Inside Radius (Inches) Invert	Inside Radius (Inches) Haunch	Inside Radius (Inches) Side	Inside Radius (Inches) Crown
12	1	11	0	47	10.00	4.32	7.69	12.99	135.95	37.95	88.00	67.95
12	10	11	2	49	11.04	4.44	7.50	14.10	148.53	38.53	86.78	74.53
13	0	12	0	51	10.97	4.27	8.79	13.91	160.54	37.54	98.19	72.54
13	8	12	4	53	11.98	4.36	8.67	14.96	167.77	37.77	102.62	76.77
14	0	12	11	55	11.99	4.39	9.62	14.98	182.90	37.90	110.65	76.90
14	6	13	5	57	13.07	4.61	9.26	16.18	174.88	38.88	124.73	78.88
14	8	14	1	59	13.00	4.42	10.58	15.99	192.96	37.96	130.01	78.96
15	5	14	5	61	14.04	4.59	10.33	17.11	201.54	38.54	135.39	83.54
15	6	15	2	63	13.97	4.45	11.61	16.92	211.59	37.59	149.14	81.59
16	2	15	6	65	14.99	4.50	11.52	17.97	216.85	37.85	154.40	85.85
16	6	16	0	67	14.07	4.73	12.10	19.29	272.34	39.34	153.89	89.34
16	8	16	4	68	15.01	4.49	12.49	19.03	246.17	38.17	160.82	89.17
17	3	17	1	70	15.04	5.71	12.20	19.13	214.64	47.64	171.19	90.64
18	5	16	11	72	16.09	5.87	11.95	20.27	249.37	48.37	155.02	100.37
19	0	17	3	74	17.02	5.60	12.36	21.06	262.29	47.29	153.14	105.29
19	7	17	7	76	17.07	5.79	13.06	21.24	296.21	48.21	154.46	108.21
20	5	17	9	78	18.08	5.78	13.05	22.27	317.39	48.39	149.94	115.39

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

Geometric Data—Pedestrian/Animal Underpass

Span Ft.-In.	Rise Ft.-In.	Total N	Tb In.	Ts In.	Θc Degrees	Θh Degrees
6-1	5-9	24	9.2	7.2	100.2	129.9
6-3	6-1	25	11.1	11.0	119.3	120.4
6-3	6-6	26	11.6	15.6	136.5	111.7
6-2	7-0	27	10.2	21.1	152.2	103.9
6-3	7-4	28	11.6	25.2	153.3	103.4
6-1	7-10	29	9.8	30.9	161.7	99.2
6-3	8-2	30	11.3	35.0	161.3	99.3

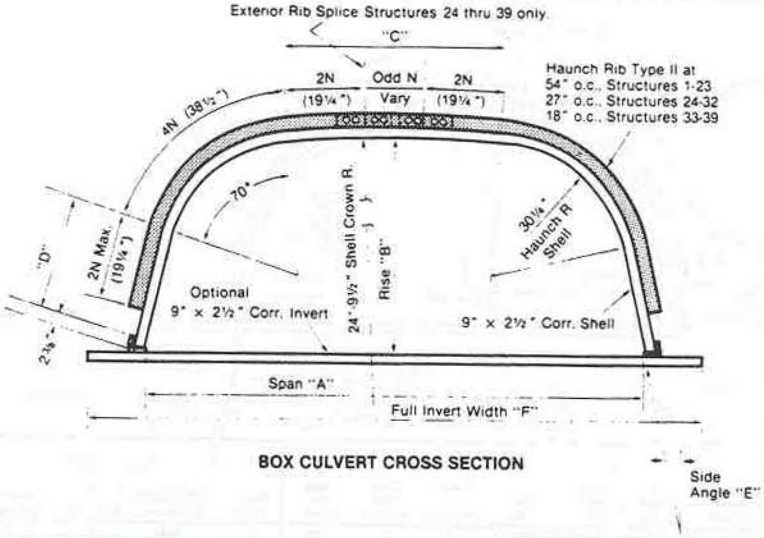


Typical Section

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES
FOR ALUMINUM CULVERTS

Box Culvert Geometric Data



				SHELL							FULL INVERT				
Structure Number	Span "A" (Ft.-In.)	Rise "B" (Ft.-In.)	Area (Sq. Ft.)	Crown Width "C" (N)	Leg Length "O" (N)	Side Angle "E" Deg. Min.	Total N	Haunch Plate Length (N)	Crown Plate Length (N)	Bolts/Ft.	Width "F" (N)	Supplemental/Stub Pl. Thick.	Supplemental/Stub Pl. Width (N)	Weight/Ft.	Bolts/Ft.
1	8- 9	2- 6	18.4	5	.5	15-24	14	1 @ 14	—	6.67	13	—	—	23.06	5.78
2	9- 2	3- 3	25.4	5	1.5	15-24	16	2 @ 8	—	11.56	13	—	—	23.06	5.78
3	9- 7	4- 1	32.6	5	2.5	15-24	18	2 @ 9	—	12.00	14	—	—	24.44	6.00
4	10- 0	4-10	40.2	5	3.5	15-24	20	2 @ 10	—	12.44	14	—	—	24.44	6.00
5	10- 6	5- 7	48.1	5	4.5	15-24	22	2 @ 11	—	12.89	15	—	—	25.82	6.22
6	10-11	6- 4	56.4	5	5.5	15-24	24	2 @ 12	—	13.33	17	—	—	28.58	6.57
7	11- 4	7- 2	65.0	5	6.5	15-24	26	2 @ 13	—	13.78	17	—	—	28.58	6.67
8	10- 2	2- 8	23.0	7	.5	13-33	16	2 @ 8	—	12.89	15	—	—	25.82	6.22
9	10- 7	3- 5	31.1	7	1.5	13-33	18	2 @ 9	—	13.33	15	—	—	25.82	6.22
10	10-11	4- 3	39.5	7	2.5	13-33	20	2 @ 10	—	13.78	17	—	—	28.58	6.57
11	11- 4	5- 0	48.2	7	3.5	13-33	22	2 @ 11	—	14.22	17	—	—	28.58	6.57
12	11- 8	5- 9	57.2	7	4.5	13-33	24	2 @ 12	—	14.67	17	—	—	28.58	6.57
13	12- 1	6- 7	66.4	7	5.5	13-33	26	2 @ 13	—	15.11	17	—	—	28.58	6.57
14	12- 5	7- 4	76.0	7	6.5	13-33	28	2 @ 14	—	15.56	17	—	—	28.58	6.57
15	11- 7	2-10	28.1	9	0.5	11-42	18	2 @ 9	—	14.67	17	—	—	28.58	6.57
16	11-11	3- 7	37.4	9	1.5	11-42	20	2 @ 10	—	15.11	17	—	—	28.58	6.57
17	12- 3	4- 5	46.9	9	2.5	11-42	22	2 @ 11	—	15.56	17	—	—	28.58	6.67
18	12- 7	5- 2	56.6	9	3.5	11-42	24	2 @ 12	—	16.00	19	—	—	32.02	7.11
19	12-11	6- 0	66.6	9	4.5	11-42	26	2 @ 13	—	16.44	19	—	—	32.02	7.11
20	13- 3	6- 9	76.9	9	5.5	11-42	28	2 @ 14	—	16.89	19	—	—	32.02	7.11
21	13- 0	3- 0	33.8	11	0.5	9-52	20	2 @ 10	—	16.44	19	—	—	32.02	7.11
22	13- 4	3-10	44.2	11	1.5	9-52	22	2 @ 11	—	16.89	19	—	—	32.02	7.11
23	13- 7	4- 7	54.8	11	2.5	9-52	24	2 @ 12	—	17.33	19	—	—	32.02	7.11
24	13-10	5- 5	65.6	11	3.5	9-52	26	2 @ 13	—	23.11	19	—	—	32.02	7.11
25	14- 1	6- 2	76.6	11	4.5	9-52	28	2 @ 14	—	23.56	20	—	—	33.34	12.14
26	14- 5	3- 3	40.0	13	0.5	8-1	22	2 @ 11	—	22.67	20	—	—	33.34	12.14
27	14- 8	4- 1	51.5	13	1.5	8-1	24	2 @ 8	8	25.56	21	.100	2	40.23	12.37
28	14-10	4-10	63.2	13	2.5	8-1	26	2 @ 9	8	26.44	21	.100	2	40.23	12.37
29	15- 1	5- 8	75.1	13	3.5	8-1	28	2 @ 10	8	26.89	21	.100	2	40.23	12.37
30	15- 4	6- 5	87.2	13	4.5	8-1	30	2 @ 11	8	27.33	21	.100	2	40.23	12.37
31	15- 6	7- 3	99.4	13	5.5	8-1	32	2 @ 12	8	27.78	22	.100	2	41.61	12.49
32	15- 9	8- 0	111.8	13	6.5	8-1	34	2 @ 13	8	28.22	22	.100	2	41.61	12.39
33	15-10	3- 6	46.8	15	0.5	6-10	24	2 @ 8	8	32.22	22	.100	2	41.61	12.39
34	16- 0	4- 3	59.5	15	1.5	6-10	26	2 @ 9	8	33.56	22	.100	2	41.61	12.49
35	16- 2	5- 1	72.3	15	2.5	6-10	28	2 @ 10	8	34.89	23	.100	2	42.99	13.11
36	16- 4	5-11	85.2	15	3.5	6-10	30	2 @ 11	8	35.33	23	.100	3	45.75	13.11
37	16- 6	6- 8	98.3	15	4.5	6-10	32	2 @ 12	8	35.78	23	.100	3	45.75	13.11
38	16- 8	7- 6	111.5	15	5.5	6-10	34	2 @ 13	8	36.22	23	.100	3	45.75	13.11
39	16-10	8- 3	124.8	15	6.5	6-10	36	2 @ 14	8	36.67	24	.100	3	47.13	13.33

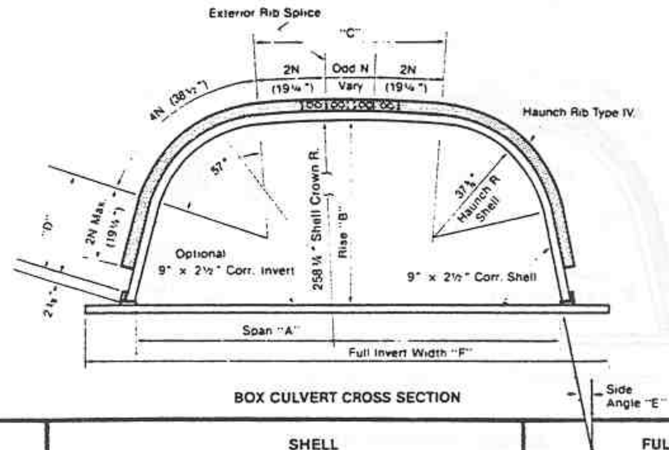
NOTES:

1) "N" equals 9.62".
2) All crowns of shells have Type IV ribs outside at 18" on centers.
3) Weights per foot listed do not include bolt weight.
4) Weight per foot of full invert includes 3½ × 3 × ¼ connecting angle and scalloped closure plate for each side. Inverts for 20N and greater are two-piece.
5) Weight per foot of footing pad includes a 3½ × 3 × ¼-in. connecting angle for each side. Optional wale beam not included.
6) Full invert plates are .100 thick. When reactions to invert require additional thickness supplemental plates of thickness and width listed are furnished to bolt between full invert and side connecting angle.
7) Width of footing pad is for each side.
8) For structures using short footing pads with leg length "D" equal to 3.5 N or more, either wale beam stiffeners should be used to avoid

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

Box Culvert Geometric Data



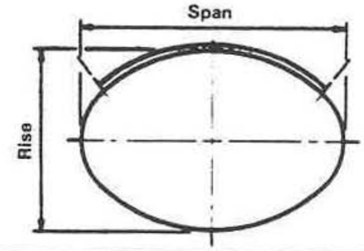
Structure Number	Span "A" (Ft.-In.)	Rise "B" (Ft.-In.)	Area (Sq. Ft.)	SHELL: Crown Width "C" (N)	Leg Length "D" (N)	Side Angle "E" Deg. Min.	Total N	Haunch Plate Length (N)	Crown Plate Length (N)	Bolts Per Foot	FULL INVERT: Width "F" (N)	Supplemental Stub Plates: Thickness	Supplemental Stub Plates: Width (N)	Weight Per Foot	Bolts Per Foot
40	17- 9	3-10	54.4	17	.5	14-54	26	8	10	33.56	25	.100	3	48.51	13.56
41	18- 2	4- 7	68.3	17	1.5	14-54	28	9	10	34.89	25	.100	3	48.51	13.56
42	18- 7	5- 4	82.5	17	2.5	14-54	30	10	10	36.22	26	.100	3	49.88	13.78
43	19- 0	6- 1	97.1	17	3.5	14-54	32	11	10	36.67	27	.100	3	51.26	14.00
44	19- 5	6-11	111.9	17	4.5	14-54	34	12	10	37.11	27	.100	3	51.26	14.00
45	19-10	7- 8	127.1	17	5.5	14-54	36	13	10	37.56	28	.100	3	52.64	14.22
46	20- 3	8- 5	142.6	17	6.5	14-54	38	14	10	38.00	28	.100	3	52.64	14.22
47	19- 1	4- 2	63.3	19	.5	12-47	28	8	12	34.89	27	.100	3	51.26	14.00
48	19- 5	4-11	78.3	19	1.5	12-47	30	9	12	36.22	27	.100	3	51.26	14.00
49	19- 9	5- 8	93.6	19	2.5	12-47	32	10	12	37.56	27	.100	3	51.26	14.00
50	20- 1	6- 6	109.2	19	3.5	12-47	34	11	12	38.00	28	.100	3	52.64	14.22
51	20- 6	7- 3	125.0	19	4.5	12-47	36	12	12	54.44	29	.125	3	56.09	14.44
52	20-10	8- 1	141.2	19	5.5	12-47	38	13	12	54.89	29	.100	3	54.02	14.44
53	21- 2	8-10	157.6	19	6.5	12-47	40	14	12	55.33	30	.150	3	59.54	14.67
54	20- 4	4- 6	73.1	21	.5	10-40	30	8	14	49.56	29	.150	3	58.16	14.44
55	20- 7	5- 3	89.2	21	1.5	10-40	32	9	14	52.22	29	.125	3	56.09	14.44
56	20-11	6- 1	105.5	21	2.5	10-40	34	10	14	54.89	29	.100	3	54.02	14.44
57	21- 3	6-10	122.1	21	3.5	10-40	36	11	14	55.33	30	.150	3	59.54	14.67
58	21- 6	7- 8	139.0	21	4.5	10-40	38	12	14	55.78	30	.125	3	57.47	14.67
59	21-10	8- 5	156.0	21	5.5	10-40	40	13	14	56.22	31	.175	3	62.99	14.89
60	22- 1	9- 3	173.3	21	6.5	10-40	42	14	14	56.67	31	.150	3	60.92	14.89
61	21- 7	4-11	83.8	23	.5	8-32	32	9	14	50.89	30	.125	3	57.47	14.67
62	21-10	5- 8	101.0	23	1.5	8-32	34	10	14	53.56	31	.175	3	62.99	14.89
63	22- 1	6- 6	118.4	23	2.5	8-32	36	11	14	56.22	31	.150	3	60.92	14.89
64	22- 3	7- 3	135.9	23	3.5	8-32	38	12	14	56.67	31	.150	4	65.05	14.89
65	22- 6	8- 1	153.7	23	4.5	8-32	40	13	14	57.11	32	.200	4	71.95	15.11
66	22- 9	8-10	171.6	23	5.5	8-32	42	14	14	57.56	32	.175	4	69.19	15.11
67	23- 0	9- 8	189.8	23	6.5	8-32	44	15	14	58.00	32	.150	4	66.43	15.11
68	22- 9	5- 4	95.5	25	.5	6-25	34	10	14	52.22	32	.175	4	69.19	15.11
69	23- 0	6- 1	113.7	25	1.5	6-25	36	11	14	54.89	32	.150	4	66.43	15.11
70	23- 2	6-11	132.1	25	2.5	6-25	38	12	14	57.56	33	.225	4	76.09	15.33
71	23- 4	7- 8	150.6	25	3.5	6-25	40	13	14	58.00	33	.200	4	73.33	15.33
72	23- 6	8- 6	169.3	25	4.5	6-25	42	14	14	58.44	33	.200	4	73.33	15.33
73	23- 8	9- 3	188.1	25	5.5	6-25	44	15	14	58.89	33	.175	4	70.57	15.33
74	23-10	10- 1	207.0	25	6.5	6-25	46	16	14	59.33	34	.250	4	80.22	15.56
75	24- 0	5- 9	108.2	27	.5	4-18	36	10	16	53.56	34	.225	4	77.46	15.56
76	24- 1	6- 6	127.5	27	1.5	4-18	38	11	16	56.22	34	.225	4	77.46	15.56
77	24- 3	7- 4	146.8	27	2.5	4-18	40	12	16	58.89	34	.200	4	74.71	15.56
78	24- 4	8- 2	166.2	27	3.5	4-18	42	13	16	59.33	34	.200	4	74.71	15.56
79	24- 5	8-11	185.7	27	4.5	4-18	44	14	16	59.78	34	.200	4	74.71	15.56
80	24- 7	9- 9	205.3	27	5.5	4-18	46	15	16	60.22	35	.300	4	87.12	15.78
81	24- 8	10- 6	225.0	27	6.5	4-18	48	16	16	60.67	35	.250	4	81.60	15.78
82	25- 2	6- 2	122.0	29	.5	2-11	38	11	16	54.89	35	.200	4	76.09	15.78
83	25- 2	7- 0	142.2	29	1.5	2-11	40	12	16	57.56	35	.200	4	76.09	15.78
84	25- 3	7- 9	162.4	29	2.5	2-11	42	13	16	60.22	36	.300	4	88.50	16.00
85	25- 4	8- 7	182.6	29	3.5	2-11	44	14	16	60.67	36	.300	4	88.50	16.00
86	25- 4	9- 5	202.9	29	4.5	2-11	46	15	16	61.11	36	.300	4	88.50	16.00
87	25- 5	10- 2	223.3	29	5.5	2-11	48	16	16	61.56	36	.300	4	88.50	16.00

NOTES:

1) "N" = 9.62"
2) All shells have Type IV ribs outside only. Both haunch and crown ribs are 18" on centers for structures 40 through 50 and 9" on centers for structures 51 through 87.
3) Weights per foot listed do not include bolt weight.
4) Weight per foot of full invert includes 3½ x 3 x ¼ connecting angle and scalloped closure plate for each side. Inverts for 20 N width and greater are two piece.
5) Full invert plates are 100" thick. When reactions to invert require additional thickness, supplemental plates of thickness and width listed are furnished to bolt between full invert and side connecting angles. When thickness listed is greater than a .250" supplemental plates will be two pieces equalling the composite thickness required.
6) Weight per foot of footing pads includes 3½ x 3 x ¼ connecting angle for each side. Optional wale beam weight is not included.
7) Width of footing pads is for each side. When thickness listed is greater than .250" the footing pads will be two pieces equalling the composite thickness required.

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS



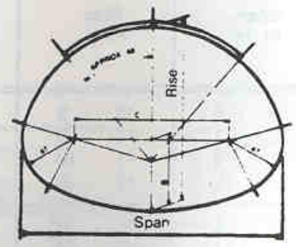
Span Ft.-in		Rise Ft-in.		Area ft²	Required N Crown or Invert	Required N Haunch	Required N Total	Inside Radius Crown & Invert in.	Inside Radius Haunch in.
19	4	12	9	191	22	10	64	150.3	53.9
20	1	13	0	202	23	10	66	157.2	53.9
20	2	11	10	183	24	8	64	164.1	42.8
20	10	12	2	193	25	8	66	171.0	42.8
21	0	15	1	248	23	13	72	157.2	70.4
21	11	13	1	220	26	9	70	177.9	48.4
22	6	15	8	274	25	13	76	171.0	70.4
23	0	14	1	249	27	10	74	184.8	53.9
23	3	15	11	288	26	13	78	177.9	70.4
24	4	16	11	320	27	14	82	184.8	75.9
24	6	14	7	274	29	10	78	198.6	53.9
25	3	14	11	287	30	10	80	205.4	53.9
25	6	16	9	330	29	13	84	198.6	70.4
26	1	18	2	369	29	15	88	198.6	81.4
26	3	15	10	320	31	11	84	212.3	59.4
27	0	16	2	334	32	11	86	219.2	59.4
27	2	19	1	405	30	16	92	205.4	86.9
27	11	19	5	421	31	16	94	212.3	86.9
28	1	17	1	369	33	12	90	226.1	64.9
28	10	17	4	384	34	12	92	233.0	64.9
29	5	19	11	455	33	16	98	226.1	86.9
30	2	20	2	472	34	16	100	233.0	86.9
30	4	17	11	415	36	12	96	246.8	64.9
31	2	21	2	513	35	17	104	239.9	92.5
31	4	18	11	454	37	13	100	253.7	70.4
32	1	19	2	471	38	13	102	260.6	70.4
32	3	22	2	555	36	18	108	246.8	98.0
33	0	22	5	574	37	18	110	253.7	98.0
33	2	20	1	513	39	14	106	267.5	75.9
34	1	23	4	619	38	19	114	260.6	103.5
34	8	20	8	548	41	14	110	281.2	75.9
35	0	21	4	574	41	15	112	281.2	81.4
35	2	24	4	666	39	20	118	267.5	109.0
35	10	25	9	719	39	22	122	267.5	120.0
36	1	22	4	620	42	16	116	288.1	86.9
36	11	25	7	736	41	21	124	281.2	114.5
37	2	22	2	632	44	15	118	301.9	81.4
38	0	26	7	786	42	22	128	288.1	120.0
38	8	28	0	844	42	24	132	288.1	131.0
40	1	29	8	928	43	26	138	295.0	142.1

SOURCE: ALUMINUM ASSOCIATION

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

Geometric Data—Pipe Arch

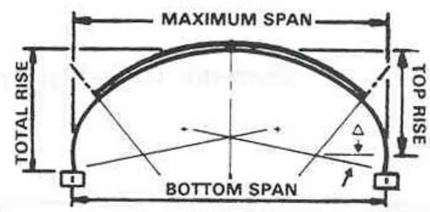


Span Ft-in	Rise Ft-in.	Area ft²	Required N Total	Required N Crown	Required N Invert	Required N Haunch	Inside Radius Crown in.	Inside Radius Invert in.	B	C
20 1	13 11	216	68	34	20	7	122.7	224.2	62.9	146.7
20 7	14 3	229	70	36	20	7	124.9	256.4	61.4	152.8
21 5	14 7	241	72	36	22	7	131.7	237.3	65.4	163.4
21 11	14 11	254	74	38	22	7	133.7	268.8	63.8	169.4
22 8	15 3	267	76	39	23	7	138.2	274.9	65.0	177.8
23 4	15 7	281	78	40	24	7	142.7	281.0	66.3	186.1
24 3	15 10	295	80	40	26	7	150.0	262.8	70.8	196.8
24 9	16 3	309	82	42	26	7	151.7	293.0	68.9	202.9
25 5	16 7	324	84	43	27	7	156.2	299.0	70.2	211.3
26 4	16 10	339	86	43	29	7	163.9	281.3	75.0	222.1
27 0	17 2	354	88	44	30	7	168.6	287.4	76.4	230.5
27 9	17 6	369	90	45	31	7	173.3	293.5	77.9	238.9
28 5	17 10	385	92	46	32	7	178.0	299.6	79.3	247.3
29 4	18 2	401	94	46	34	7	186.6	286.7	84.6	257.9
29 10	18 6	418	96	48	34	7	187.5	311.6	82.3	264.2
30 4	18 10	435	98	50	34	7	188.6	340.1	80.0	270.2

14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

Geometric Data—Low Profile Arch



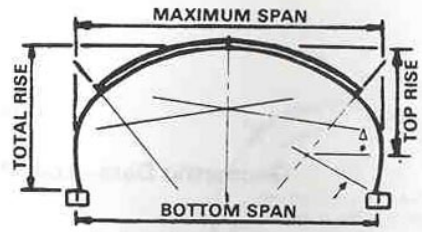
Max. Span Ft-in.		Total Rise Ft-in		Area ft²	Bottom Span Ft-in.		Top Rise Ft-in.		Required N			Inside Radius		Δ	
									Crown	Side	Total	Crown In.	Side In.	Deg.	Min.
20	1	7	6	120	19	10	6	6	23	6	35	157.2	54.0	12	19
19	5	6	9	105	19	2	5	10	23	5	33	157.2	43.0	15	22
21	6	7	9	133	21	4	6	9	25	6	37	171.0	54.0	12	19
22	3	7	11	140	22	1	6	11	26	6	38	177.9	54.0	12	19
23	0	8	0	147	22	10	7	1	27	6	39	184.8	54.0	12	19
23	9	8	2	154	23	6	7	2	28	6	40	191.7	54.0	12	19
24	6	8	3	161	24	3	7	4	29	6	41	196.6	54.0	12	19
25	3	8	5	168	25	0	7	5	30	6	42	205.4	54.0	12	19
26	0	8	7	175	25	9	7	7	31	6	43	212.3	54.0	12	19
27	3	10	0	217	27	1	9	0	31	8	47	212.3	76.0	8	51
28	1	9	6	212	27	11	8	7	33	7	47	226.1	65.0	10	17
28	9	10	3	234	28	7	9	3	33	8	49	226.1	76.0	8	52
28	10	9	8	220	28	8	8	8	34	7	48	233.0	65.0	10	17
30	4	9	11	237	30	2	9	0	36	7	50	246.8	65.0	10	17
31	0	10	8	261	30	10	9	8	36	8	52	246.8	76.0	8	52
31	7	12	1	309	31	2	10	4	36	10	56	246.8	87.0	14	0
31	1	10	1	246	30	10	9	1	37	7	51	253.7	65.0	10	17
32	4	12	3	319	31	11	10	6	37	10	57	253.7	87.0	14	0
31	9	10	2	255	31	7	9	3	38	7	52	260.6	65.0	10	17
33	1	12	5	330	32	8	10	8	38	10	58	260.6	87.0	14	0
33	2	11	0	289	33	0	10	1	39	8	55	267.5	76.0	8	52
34	6	13	3	367	34	1	11	6	39	11	61	267.5	98.0	12	26
34	8	11	4	308	34	6	10	4	41	8	57	281.2	76.0	8	52
37	11	15	7	478	37	8	13	10	41	14	69	281.2	131.0	9	23
35	5	11	5	318	35	3	10	6	42	8	58	288.1	76.0	8	52
38	8	15	9	491	38	4	14	0	42	14	70	288.1	131.0	9	23

See "Notes" Table 5-20A or 5-20B for rib spacing when required.

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

Geometric Data—High Profile Arch



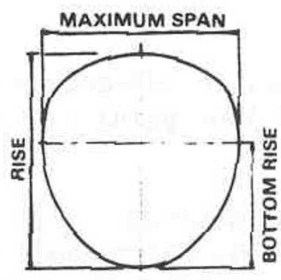
Max Span Ft.-in.		Total Rise Ft.-in.		Area ft²	Bottom Span Ft-in.		Top Rise Ft-in.		Required N Crown	Required N Haunch	Required N Side	Required N Total	Inside Radius Crown in.	Inside Radius Haunch in.	Inside Radius Side in.	Δ Deg.	Δ Min.
20	1	9	1	152	19	6	6	6	23	5	3	39	157.2	54.0	157.2	11	40
20	9	12	1	214	18	10	7	3	23	6	6	47	157.2	65.0	157.2	22	8
21	6	11	8	215	19	10	6	9	25	5	6	47	171.0	54.0	171.0	20	20
22	10	14	6	284	19	10	8	6	25	7	8	55	171.0	76.0	171.0	26	48
22	3	11	9	224	20	7	6	11	26	5	6	48	177.9	54.0	177.9	19	33
22	11	14	0	275	20	1	7	7	26	6	8	54	177.9	65.0	177.9	25	44
23	0	11	11	234	21	5	7	1	27	5	6	49	184.8	54.0	184.8	18	49
24	4	14	10	309	21	7	8	5	27	7	8	57	184.8	76.0	184.8	24	50
23	9	12	1	244	22	2	7	2	28	5	6	50	191.7	54.0	191.7	18	8
24	6	13	8	288	21	11	7	4	29	5	8	55	198.6	54.0	198.6	23	2
25	10	15	1	334	23	3	8	9	29	7	8	59	198.6	76.0	198.6	23	6
25	3	13	1	283	23	3	7	5	30	5	7	54	205.4	54.0	205.4	19	35
26	6	15	3	347	24	0	8	10	30	7	8	60	205.4	76.0	205.4	22	19
26	0	13	3	294	24	1	7	7	31	5	7	55	212.3	54.0	212.3	18	57
27	3	15	5	360	24	10	9	0	31	7	8	61	212.3	76.0	212.3	21	36
27	5	13	6	317	25	8	7	10	33	5	7	57	226.1	54.0	226.1	17	48
29	5	16	5	412	27	1	10	0	33	8	8	65	226.[illegible]	87.0	226.1	20	18
28	2	14	5	348	25	11	8	0	34	5	8	60	233.0	54.0	233.0	19	37
30	2	18	0	466	26	8	10	2	34	8	10	70	233.0	88.0	233.0	23	51
30	4	15	5	399	28	2	9	0	36	6	8	64	246.8	65.0	246.8	18	34
31	7	18	4	497	28	5	10	4	36	8	10	72	246.8	87.0	246.8	23	3
31	1	15	7	412	29	0	9	1	37	6	8	65	253.7	65.0	253.7	18	3
31	8	17	9	483	28	7	9	10	37	7	10	71	253.7	76.0	253.7	22	25
32	4	19	11	554	27	11	10	6	37	8	12	77	253.7	87.0	253.7	26	45
31	9	17	2	469	28	9	9	3	38	6	10	70	260.6	65.0	260.6	21	47
33	1	20	1	571	28	9	10	8	38	8	12	78	260.6	87.0	260.6	26	3
32	6	17	4	484	29	6	9	4	39	6	10	71	267.5	65.0	267.5	21	14
33	10	20	3	588	29	7	10	9	39	8	12	79	267.5	87.0	267.5	25	23
34	0	17	8	514	31	2	9	8	41	6	10	73	281.2	65.0	281.2	20	11
34	8	19	10	591	30	7	10	4	41	7	12	79	281.2	76.0	281.2	24	7
35	4	21	3	645	30	7	11	0	41	8	13	83	281.2	87.0	281.2	26	6
37	3	23	4	747	32	7	13	2	41	11	13	89	281.2	120.0	281.2	26	8
34	9	17	9	529	31	11	9	9	42	6	10	74	288.1	65.0	288.1	19	42
35	5	20	0	608	31	5	10	6	42	7	12	80	288.1	76.0	288.1	23	33
36	1	21	5	663	31	5	11	2	42	8	13	84	288.1	87.0	288.1	25	28
38	0	23	6	767	33	5	13	3	42	11	13	90	288.1	120.0	288.1	25	31

See "Notes" Table 5-20A or 5-20B for rib spacing when required.

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

STANDARD SIZES FOR ALUMINUM CULVERTS

Geometric Data—Pear Shape



Max. Span Ft.-in.	Rise Ft.-in.	Rise Bottom Ft-in.	Area ft²	Required N					Inside Radius			
				Top	Corner	Side	Bottom	Total	Bottom in.	Side in.	Corner in.	Top in.
23 7	25 6	14 10	477	25	5	24	15	98	108.31	198.07	74.07	175.07
24 0	25 10	15 1	497	22	7	22	20	100	119.07	208.07	84.07	194.07
25 4	25 11	15 10	518	27	7	20	20	102	124.23	218.24	84.24	191.24
24 10	27 7	16 9	545	27	5	25	18	105	110.90	236.21	69.21	191.21
28 10	27 3	19 8	590	32	7	27	8	110	79.61	257.96	68.96	252.96
26 8	28 3	18 0	594	28	5	30	12	110	95.45	241.24	57.24	251.24
28 0	27 10	16 9	624	27	8	22	25	112	146.38	227.72	86.72	244.72
28 7	30 7	19 7	690	32	7	24	24	118	133.13	288.45	84.45	218.45
30 0	29 7	20 0	699	32	8	23	25	119	142.41	288.26	79.26	262.26
30 0	31 2	19 11	739	34	7	24	26	122	144.43	·288.58	84.58	231.58

Figure 14.3.48 Standard Sizes for Aluminum Culvert (Source: Aluminum Association), continued

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Chapter 15
Advanced Inspection Methods

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Chapter 15
Advanced Inspection Methods

Topic 15.1 Timber

15.1.1 Introduction

Advanced inspection methods give inspectors the ability to further evaluate suspected defects found during a visual inspection. They can also be used to perform inspections on members that are not accessible. Advanced inspection methods usually require calibrated testing equipment, a professionally trained technician to perform the testing, and a professional that has expertise in interpreting the advanced inspection results. However, it is beneficial to have an understanding of the various advanced inspection methods.

There are two main classifications of advanced inspection methods. The first is labeled nondestructive testing or evaluation (NDT or NDE). This classification pertains to advanced inspection methods that do not impair the usefulness of the member being tested. Other testing, the second main classification, covers advanced inspection methods that affect or destroy the structural integrity of the member being tested.

New technology is making the use of these highly technical systems economically feasible for bridge inspection. From this fact, advanced inspection methods are becoming more popular for the routine inspection of bridge members. Current and future studies have been focusing directly on relating results from advanced inspection methods into Bridge Management Systems ratings.

This Topic describes the different types of nondestructive and other test methods for timber bridge members and the general methods for each.

15.1.2
Nondestructive Testing Methods

Sonic Testing

A Sonic Testing device is used to detect decay or other low density regions in timber members. Starting about six inches below the ground line, probes are pressed on opposite sides of the timber member. A trigger trips a hammer that sends a sound wave down one probe, through the member, and up the other probe to a dial (see Figure 15.1.1).

Figure 15.1.1	Sonic Testing Equipment

This method eliminates the need for making holes in timber members. Members testing positive for decay are then drilled or cored to determine the detailed nature of the deficiency. A dial reading that is low, compared with that of a good member of similar diameter, indicates decay or another low density region that delayed the sound wave within the member. However, it is a good idea to take several readings on the member since the readings are nearly instantaneous, and the Sonic Testing equipment needs to be checked frequently for proper calibration.

Used by trained personnel, Sonic Testing works well with Douglas fir and western red cedar. However, it does not work as well with southern pine members because of the high incidence of ring shakes.

Spectral Analysis

Spectral Analysis, sometimes called stress wave, uses sonic waves to produce stress waves in a timber member. The stress waves are then used to locate decay in timber members. The stress waves travel through the timber member and reflect off the timber surface, any flaws, or joints between adjacent members at the speed of sound. It is known that stress waves travel slower in decayed members than in sound members. If the member's dimensions are known, the amount of time it takes for a stress wave to travel the known distance can indicate that deficiencies are evident due to longer stress wave timings.

Stress waves are also used to determine the in-situ strength of timber members. Sound timber members transmit waves at higher velocity than decayed wood. The velocity of the stress wave can be calculated by obtaining time of flight readings over a set length. The velocity can be converted in to a dynamic modulus of elasticity, which in turn, allows properly trained personnel to estimate the strength properties of the wood.

First, a stress wave is induced by striking the specimen with an impact device that is instrumented with an accelerometer that emits a start signal to a timer. A second accelerometer, which is held in contact with the other side of the specimen, serves to detect the leading edge of the propagating stress wave and sends a stop signal to the timer. The elapsed time for the stress wave to propagate between the accelerometers is displayed on the timer.

The use of stress wave velocity to detect wood decay in timber bridges and other structures is limited only by access to the structural members under consideration. It is especially useful on thick timbers or glulam timbers where hammer sounding is not effective. Note that access to both sides of the timber member is required.

The transmission time is affected by such properties as growth ring orientation, decay, moisture content, and preservative treatment.

Figure 15.1.2 Stress Wave Timer

Ultrasonic Testing

Ultrasonic testing (UT) consists of high frequency sound waves introduced by a sending transducer. Discontinuities in the specimen interrupt the sound wave and deflect it toward a receiving transducer. The magnitude of the return signal allows a measurement of the flaw size. The distance from the transducer to the flaw can be estimated from the known properties of the sound wave and of the material being tested. Ultrasonic testing can be used to detect cracks, internal flaws, discontinuities, and sub-surface damage (see Figure 15.1.3).

Figure 15.1.3 Ultrasonic Testing Equipment

In timber bridge members, ultrasonic testing can be used to determine the in-place strength of timber bridge members, both above and below water. The load-carrying capacity of the member is correlated to the member's wave velocity normal to the grain and to its in-place unit weight.

Vibration

A newer type of nondestructive testing that can determine the condition of timber bridge members deals with the use of vibrations (see Figure 15.1.4). This nondestructive evaluation method is based on the philosophy that sound timber members vibrate at a certain frequency. While testing a timber member, if the member vibrates at a different frequency than the established theoretical frequency, the member may have deterioration present. Vibratory testing methods in timber members are basically used to determine the member's modulus of elasticity. From this, other properties of the timber member can be established.

Figure 15.1.4 Vibration Testing Equipment

15.1.3 Other Testing Methods

Boring or Drilling

While drilling and coring are the most common methods for detecting internal deterioration in bridges, boring is seen as the most dependable and widely used method for detecting internal decay in timber. Drilling and coring are used to detect the presence of voids and to determine the thickness of the residual shell when voids are present. Boring permits direct examination of an actual sample from a questionable member. A timber boring tool is used to extract wood cores for examination (see Figure 15.1.5).

Figure 15.1.5 Timber Boring Tool

Drilling is performed using a rechargeable drill or a brace and bit. An abrupt decrease in drilling resistance indicates either decay or a void. However, wet wood and natural voids can falsely suggest decay. Decay can be based on how the auger type drill bit pulls its way through the wood or on measuring the torque resistance on the bit as it penetrates the wood. Drilling is usually done with a power drill or hand-crank drill equipped with a 3/8 inch to 3/4 inch diameter bit. If decay is detected, the inspection hole can be used to add remedial treatments to the wood. While samples are generally not attainable, observation of the wood particles removed during the drilling process can provide valuable information about the member. The depth of preservative penetration, if any, can be determined, and regions of discolored wood may indicate decay.

Coring with timber boring tools also provides information on the presence of decay pockets and other voids, and coring produces a solid wood core that can be carefully examined for evidence of decay. The use of increment cores for assessing the presence and damage due to bacterial and fungal decay requires special care. Cleaning of the timber boring tools is necessary after each core extraction to eliminate transfer of organisms. There are several cleaning agents available to clean the timber boring tools or drill bits that work well. Core samples that do not show visible signs of decay can be cultured to detect the presence of potential decay hazards. Many laboratories can provide this service. Core samples are more commonly used to detect the presence of internal decay pockets and to measure the depth of preservative penetration and retention. Culturing provides a simple method for assessing the potential decay hazard and many laboratories provide routine culturing services. Because of the wide variety of fungi near the surface, culturing is not practical for assessing the hazard of external decay.

A decay detection device is a newer drilling and logging tool. It operates upon the principle that a drill moving through sound wood encounters more resistance than a drill moving through decayed, and/or soft wood. It records the resistance, using a pen, paper, and rotary drum arrangement so that a permanent graphic record of the test is generated. Sound wood produces a series of near vertical markings on the record, however, when decayed wood is encountered, the resistance drops and the markings assume a more horizontal or diagonal pattern. By studying the resulting record, an experienced operator can determine if decay exists and can estimate the approximate location and size of the decayed area (see Figure 15.1.6).

Figure 15.1.6 Inspector Using Decay Detection Device

Bore holes can provide an entrance for bacterial and fungal decay to gain access to the member. Inspection methods that destroy or remove a portion of the wood, splinters, probe holes, and borings may become avenues for decay entry if not properly treated at the conclusion of the inspection. As such, the holes need to be treated with a preservative and plugged after testing. Failure to properly treat the wood may result in accelerated decay development or deterioration in the structure.

Moisture Content

Moisture meters can be used to determine moisture content in a timber member (see Figure 15.1.7). Moisture contents exceeding 20% indicate the condition of the wood is conducive to decay. As a sliding hammer drives two electrodes into the wood, a ruler emerging from the top of the hammer measures the depth. These electrodes can measure moisture content to a depth of approximately 2 1/2 inches. Because the high moisture content of decaying wood causes steeper than normal moisture gradients, the meter is useful for determining the extent of decay.

Figure 15.1.7 Moisture Content Equipment

Probing

Probing consists of inserting a pointed tool, such as an ice pick, into the wood and comparing its resistance with that of sound wood. Lack of resistance or excessive softness to probe penetration may reveal the presence of decay. Two forms of probing are a pick test and a shell-thickness indicator.

A pick test consists of removing a small piece of wood with a pick or pocketknife (see Figure 15.1.8). If the wood splinters, it is probably sound wood, and if it breaks abruptly, it is probably decayed wood. Since the pick test only removes a small portion of the timber member, it may be considered a physical testing method. See Topic 6.1.7 for more information.

Figure 15.1.8 Pick Test: Sound Wood, Decayed Wood

A shell-thickness indicator is a thin, metal, hooked rod used to determine the thickness of solid, but not necessarily sound, wood. The rod is inserted into a hole made by coring or drilling and is then pulled back with pressure against the side of the hole. The hook easily attaches to the edge of a decay pocket, making it possible to determine the depth of the decay and the solid wood.

Field Ohmmeter

The field ohmmeter measures electrical resistance to detect decay in timber members (see Figure 15.1.9). It is best used in wood with a moisture content of at least 27 percent, a value indicative of decaying wood. A probe is used consisting of two twisted, insulated wires with the insulation removed near the tip. This probe is inserted to various depths into a hole with a diameter of 3/32 of an inch. If the electrical resistance changes as the probe goes deeper, this indicates decay or a defect.

While this device effectively detects decay, it can also produce misleading readings on sound timber. Consequently, drilling or coring needs to be done on suspect members to verify results. Like sonic testing, the field ohmmeter needs to be recalibrated frequently.

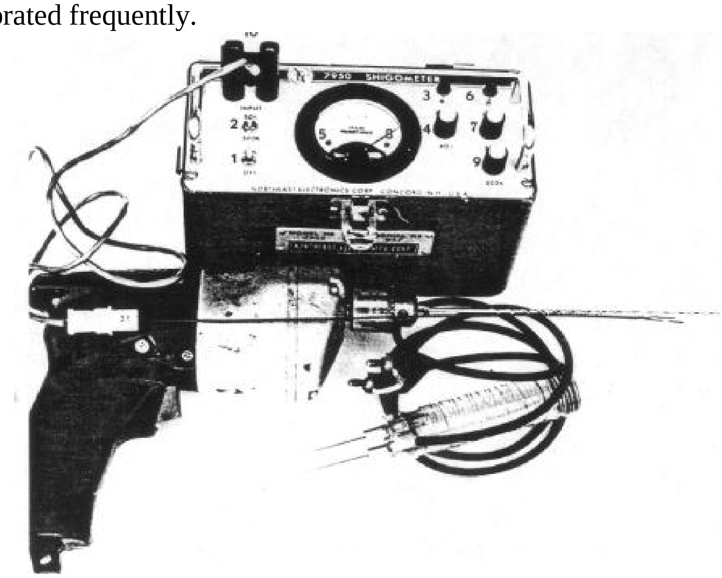

Figure 15.1.9 Field Ohmmeter Equipment

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Chapter 15
Advanced Inspection Methods

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Topic 15.2 Concrete

15.2.1 Introduction

Advanced inspection methods give inspectors the ability to further evaluate suspected deficiencies found during a visual inspection. They can also be used to perform inspections on members that are not accessible. Advanced inspection methods usually require calibrated testing equipment, a professionally trained technician to perform the testing and a professional that has expertise in interpreting the advanced inspection results. Bridge inspectors need to have an understanding of the various advanced inspection methods.

There are two main classifications of advanced inspection methods. The first is labeled nondestructive testing or evaluation (NDT or NDE). This classification pertains to advanced inspection methods that do not impair the usefulness of the member being tested. Other testing, the second main classification, covers advanced inspection methods that may affect the structural integrity of the member being tested.

New technology is making the use of these highly technical systems more economically feasible for bridge inspection. From this fact, advanced inspection methods are becoming more popular for supplementing visual inspection methods predominately used for routine inspection of bridge members. Current studies have been focusing directly on relating results from advanced inspection methods into Bridge Management Systems ratings.

This Topic describes the different types of nondestructive and other test methods for concrete bridge members and the general methods for each.

15.2.2 Nondestructive Testing Methods

Acoustic Wave Sonic/Ultrasonic Velocity Measurements

An evaluation of concrete decks can be accomplished with sonic or ultrasonic acoustic wave velocity measurements. This method delineates areas of internal cracking (including delaminations) and deteriorated concrete, including the estimation of strength and elastic modulus. A mobile automated data acquisition device with an impact energy source and multiple sensors is the principle part of a computer-based monitoring and recording system for detailed evaluation of bridge decks. Bridge abutments and concrete support members are tested using the same recording system with a portable, hand-held sensor array (see Figure 15.2.1 and 15.2.2). The system works directly on either bare concrete or through wearing surfaces such as asphalt. It can distinguish between debonded asphalt and delaminations, and it is effective for a detailed evaluation of large areas.

Figure 15.2.1 Portable Hand Held Sonic/Ultrasonic Testing Sensor Array System

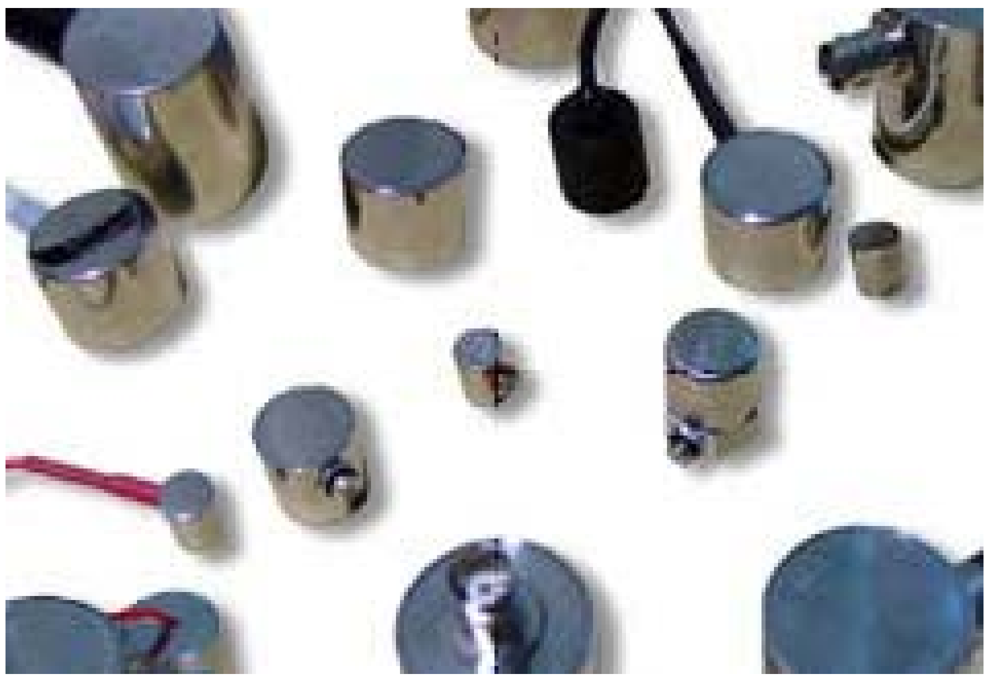

Figure 15.2.2 Acoustic Emission Sensors

Electrical Methods

Half-cell potentials are used to evaluate the corrosion activity of reinforcing steel embedded in concrete (See Figure 15.2.3). Commonly known as CSE (Copper Sulfate Electrode) tests, reinforcing bar networks are physically accessed and wired for current detection. Half-cell electrical potentials of reinforcing steel are measured by moving the CSE about the concrete surface. As the CSE contacts concrete over an actively corroding rebar, voltage is registered. Measured potential values reflect levels of corrosion activity in the rebar. Higher potential measurements indicate corrosion activity. This kind of survey can be used to determine core sample locations.

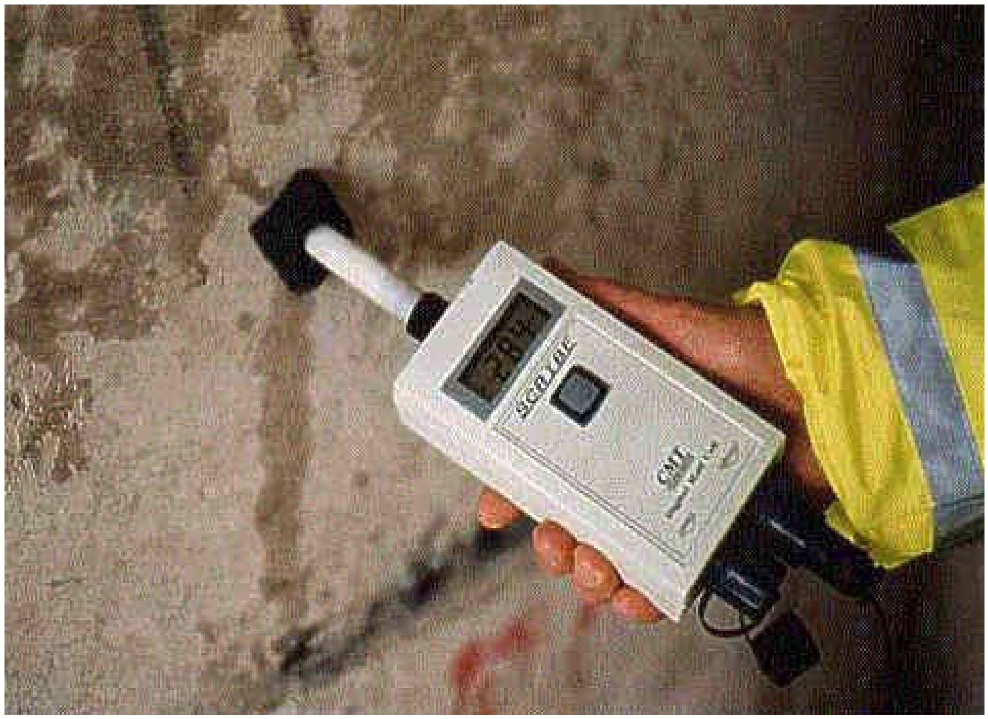

Figure 15.2.3 Half-Cell Potential

Delamination Detection Machinery

Delamination detection machinery is based on sonic responses and can be used to inspect concrete decks (see Figure 15.2.4). The portable electronic instrument consists of three components: a tapping device, a sonic receiver, and a signal interpreter. The instrument is moved across the deck as acoustic signals are passed through the deck. These signals are then received and electronically interpreted, and the output is used to generate a plan of the deck showing delaminated areas. This method can be used on concrete decks with asphalt covered surfaces, although accuracy decreases.

Figure 15.2.4 Delamination Detection Machinery

Ground-Penetrating Radar

Ground-Penetrating Radar (GPR) is a geophysical method that uses high-frequency pulsed electromagnetic waves to acquire subsurface information. An important benefit of this method is the ability to measure the thickness of asphalt covering. It can also be used to examine the condition of the top flange of box beams that may otherwise be inaccessible.

An electromagnetic wave is radiated from a transmitting antenna, and travels through the material at a velocity which is determined primarily by the electrical properties of the material. The wave spreads out and travels downward; however, materials with different electrical properties can alter its path. Upon encountering a buried object or boundary with different electrical properties, part of the wave energy is reflected or scattered back to the surface while part of its energy continues its downward path. The wave that is reflected back to the surface is captured by a receiving antenna, and recorded on a digital storage device for later interpretation (see Figure 15.2.5). The most common display of GPR data is one showing signal versus amplitude, and is referred to as a trace. A single GPR trace consists of the transmitted energy pulse followed by pulses that are received from reflecting objects or layers.

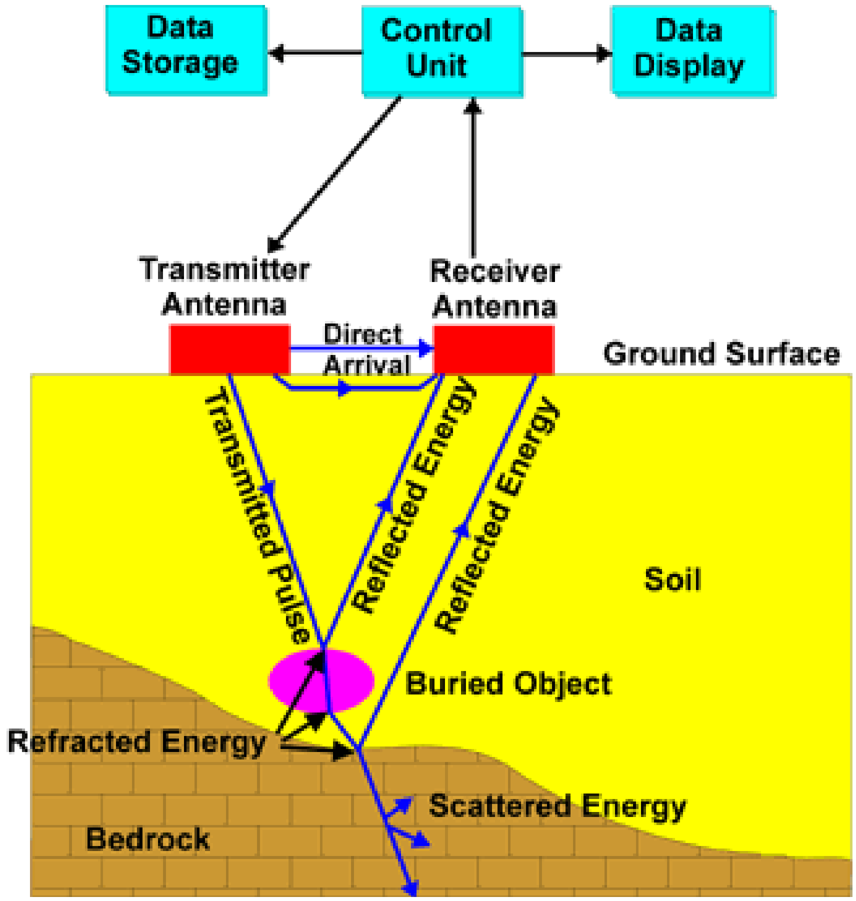

Figure 15.2.*5* Schematic of Ground-Penetrating Radar

GPR is used to map geologic conditions that include depth to bedrock, depth to the water table, depth and thickness of soil and sediment strata on land and under fresh water bodies, and the location of subsurface cavities and fractures in bedrock.

Other applications include the detection of delaminations and flaws in reinforced concrete bridge elements; location of post-tensioning ducts in prestressed concrete elements; location of objects such as pipes, drums, tanks, cables, and boulders; mapping landfill and trench boundaries; mapping contaminants; and conducting archeological investigations. GPR has been used for concrete bridge element and tunnel lining inspections.

Ground Penetrating Radar for Bridge Decks

Ground penetrating radar (GPR) technology is nearing full acceptance as a method to assess the condition of bridge decks, in particular, delaminations between concrete and rebar.

Importantly, GPR is an NDE technology, as opposed to cutting core samples from concrete decks. It can provide information on asset condition that can be used to plan and execute effective and efficient repair programs. The principal issue with GPR technology is slow rate of data capture when the depth of evaluation is more than approximately 3 inches.

Electromagnetic Methods Advancements in ground penetrating radar have led to the development of the High Speed Electromagnetic Roadway Measurement and Evaluation System (HERMES) Bridge Inspector. This system was built by the Lawrence Livermore National Library to detect delaminations in concrete decks caused by reinforcement corrosion. The HERMES Bridge Inspector sends high frequency electromagnetic pulses from sixty-four radar antennas into a bridge deck while travelling over the structure. The device is set up in a trailer mounted towing vehicle and is made up of a computer workstation, storage device, survey wheel, control electronics, and the sixty-four antenna modules or transceivers (see Figures 15.2.6 and 15.2.7). The system can inspect up to a 6 foot 3 inch width at a time with maximum speeds of up to sixty miles per hour. At speeds of around twenty miles per hour, the system can sample the concrete deck every 9/16 inch in the direction of travel. Output information can be reconstructed to show cross-sections of the deck being inspected. The depth of penetration depends on time and the material type. An 11-13/16 inch penetration in concrete can be accomplished in about six nanoseconds.

Figure 15.2.6 The HERMES Bridge Inspector (Outside)

Figure 15.2.7 The HERMES Bridge Inspector (Inside)

Pulse Velocity

Pulse velocity methods are used to evaluate relative quality of concrete and estimate compressive strength. The pulses pass through the concrete and the transit time is then measured. The pulse velocity is then interpreted to evaluate the quality of the concrete and to estimate in-place concrete compressive strength.

This equipment analyzes concrete in decks by measuring velocity of sound waves. Some equipment generates sound waves by shooting BB's on to the deck. The time for the waves to return depends on the integrity of the concrete.

Flat Jack Testing

The flat jack method was originally developed to test the in situ stress and deformation of rock and is now being applied to masonry structures. A portion of the horizontal mortar joint is removed, and the flat jack (an envelope made of metal) is inserted and pressurized to determine the state of stress. For deformation testing, two flat jacks are inserted, one directly above the other and separated by five or six courses.

Impact-Echo Testing

Sound wave reflection is a method for nondestructive evaluation of concrete and masonry, based on the use of impact-generated stress (sound) waves that propagate through the structure and are reflected by internal flaws and external surfaces.

This method can be used to determine the location and extent of flaws such as cracks, delaminations, voids, honeycombing and debonding in plain, reinforced and post-tensioned concrete structures. It can locate voids in the subgrade directly beneath slabs and pavements. It can be used to determine member thickness or locate cracks, voids and other defects in masonry structures where the brick or block units are bonded together with mortar. This method is not adversely affected by the presence of steel reinforcing bars.

A short-duration mechanical impact, produced by tapping a small steel sphere against a concrete or masonry surface, produces low-frequency stress waves that propagate into the structure and are reflected by flaws and/or external surfaces (see

Figure 15.2.8). The wavelengths of these stress waves propagate through concrete almost as though it were a homogeneous elastic medium. Multiple reflections of these waves within the structure excite local modes of vibration, and the resulting surface displacements are recorded by a transducer located adjacent to the impact. The piezoelectric crystal in the transducer produces a voltage proportional to displacement, and the resulting voltage-time signal (called a waveform) is digitized and transferred to a computer, where it is transformed mathematically in to a spectrum of amplitude vs. frequency. Both the waveform and spectrum are plotted on the computer screen. The dominant frequencies, which appear as peaks in the spectrum, are associated with multiple reflections of stress waves within the structure, or with flexural vibrations in thin or delaminated layers.

Figure 15.2.8 Impact-Echo Testing Equipment

Infrared Thermography

NDE inspection using thermography is based on imaging surface temperatures of a specimen in order to infer subsurface delaminations or defects (see Figure 15.2.10). The basic theory is that heat conduction through a material is altered if a delamination is present (see Figure 15.2.11). In this example the temperature of the deck is greater than the surrounding air. With no internal defect, heat flow through the deck is relatively uniform. An image of the surface temperature of the deck then produces an image that is relatively uniform. If a delamination is now present inside the specimen, the heat flow is altered. In this example, the surface of the deck above the delamination appears to be higher in temperature than the remainder of the deck. The rest of the deck that is not delaminated appears cooler than the delaminated area (see Figure 15.2.9).

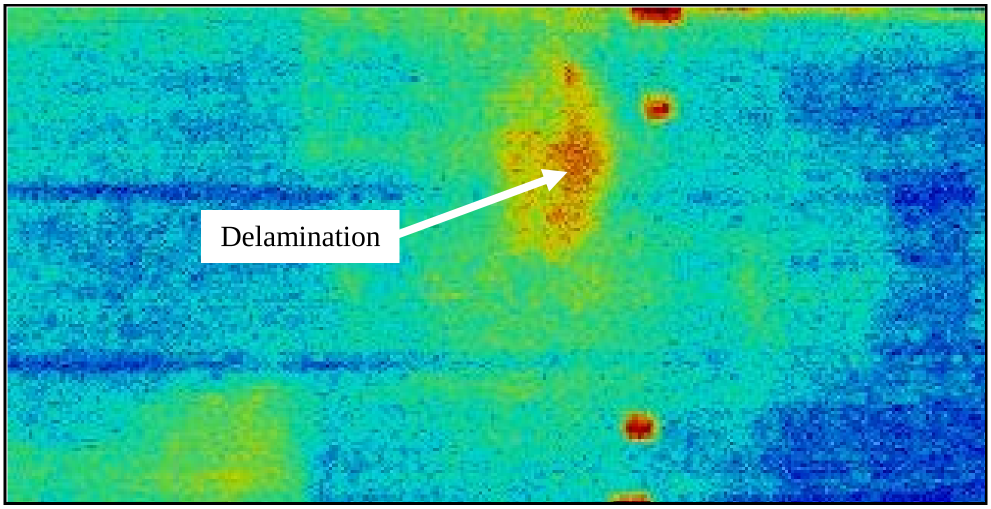

Figure 15.2.9 Deck with Area of Delamination (Warmer Colors)

Figure 15.2.10 Infrared Thermography Testing Equipment

Thermographic measurements are complicated by a number of issues. Probably the most significant is that a thermal camera does not directly measure the temperature of a specimen. The camera measures radiant flux that needs to be converted to temperature. The measured radiant flux is not only a function of the surface temperature, but is a function of the emissivity of the specimen. Emissivity is a material property that describes how well an object emits or absorbs energy. Two objects at the same temperature but with different emissivities appear as different intensities in an infrared image. Shadows or other uneven heating of a specimen are also a concern. Other environmental factors, such as water, snow, or ice on a specimen, alter results as well. Also, the method is sensitive to material property differences on the specimen surface. Surface defects, such as oil stains, water, and skid marks, show up in the infrared data.

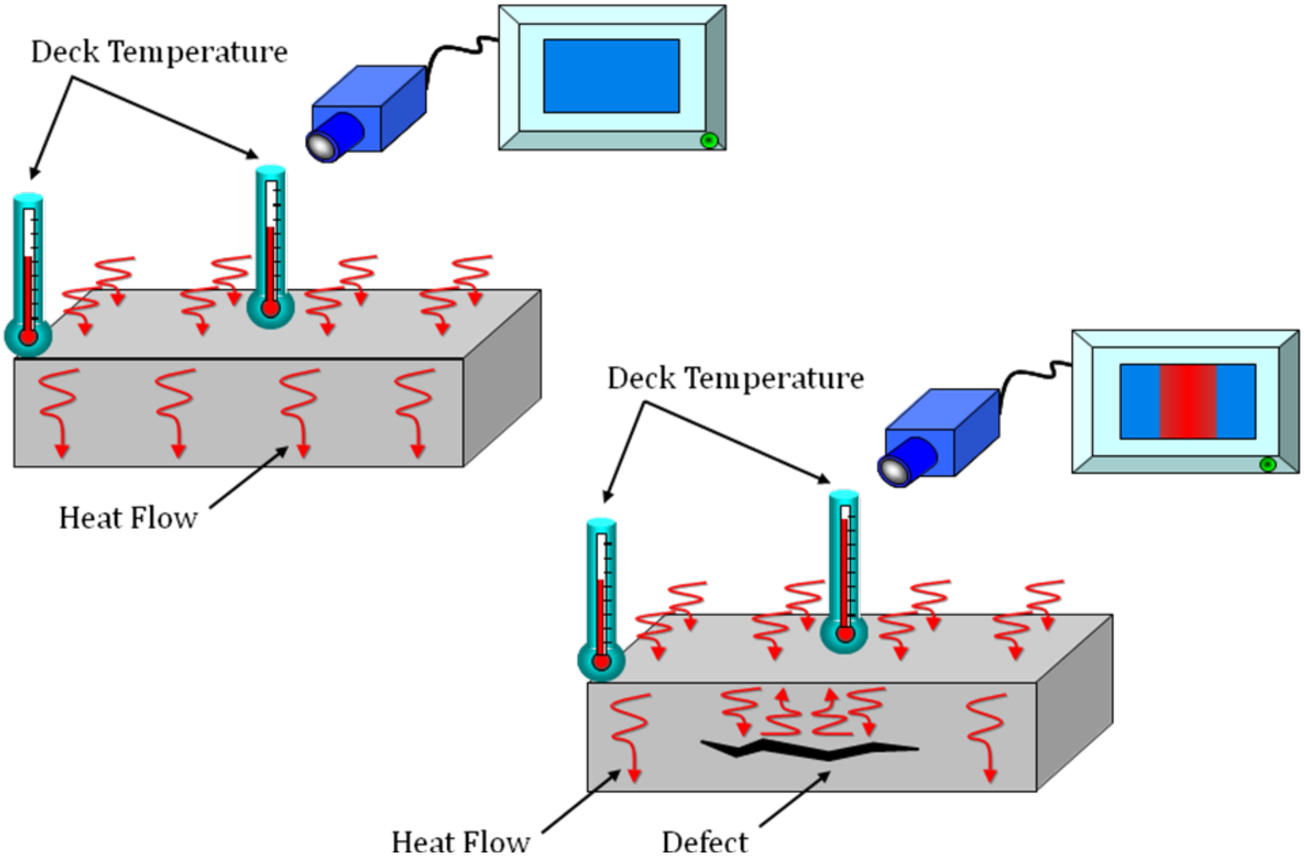

Figure 15.2.11 Schematic of Thermal Imaging

Laser Ultrasonic Testing

Laser ultrasonic testing provides information about flaws in concrete and about the position of steel reinforcement bars, which cannot be obtained with the non-laser ultrasonic testing described in this Subtopic. Laser-generated acoustic wave measurements with high stress amplitudes provide information about the quality of the concrete at various depths from the surface. Reinforcing steel does not cause misleading results in laser ultrasonic testing as it does in non-laser ultrasonic testing.

Magnetic Field Disturbance

Advanced inspection methods have been developed that can evaluate fatigue damage to steel reinforcement in concrete members. The device is known as the magnetic field disturbance (MFD) system and can be used on reinforced and prestressed concrete. The system maps the magnetic field across the bottom and sides of the beam. A discontinuity in magnetized steel, such as a fracture in a rebar or a broken wire in a steel strand, produces a unique magnetic signal. Research has been encouraging for detecting fatigue-related damage due to the significantly different magnetic signals for corroded reinforcing.

Neutron Probe for Detection of Chlorides

A neutron probe can be used to detect chlorides in construction materials. The materials are bombarded with neutrons from a small portable source. Measuring the gamma rays bouncing back provides a spectrum showing different elements, one of which is chloride. A major potential application that remains to be tested is measuring chlorides in reinforced concrete to determine corrosion hazard. Another potential application includes inspecting suspension bridge cables.

Nuclear Methods

The primary use of nuclear methods is to measure the moisture content in concrete by neutron absorption and scattering methods. These moisture measurements are then used to determine if corrosion of reinforcement is likely to occur. A more direct measurement of the rate of corrosion is more useful to the bridge inspector,

Pachometer

A pachometer is a magnetic device used in determining the position of reinforcement (see Figure 15.2.12). Magnetic methods do not detect concrete defects or deterioration directly. However, they can detect regions of inadequate cover, which is often associated with corrosion-induced deterioration. Magnetic methods can be used to measure cover in the range of 0 to 3 inches to an accuracy of about 1/4 inch.

Figure 15.2.12 Pachometer Testing Equipment

Rebound and Penetration Methods

Rebound and penetration methods measure the hardness of concrete and can be used to predict the strength of concrete. The rebound hammer (also known as the Swiss hammer) is probably the most commonly used device to measure the penetration resistance of hardened concrete. A spring-loaded device strikes the surface of the concrete, and based on the response, the compressive strength of the concrete can be determined. This inspection method can be used to compare the quality of the concrete in different parts of concrete bridge components. However, only the surface of the concrete is being tested, and the strength value is relative.

Another common penetration device utilizes a pistol-like driving device that fires a probe into the surface of the concrete. The probe is specifically designed to crack aggregate particles and to compress the concrete being tested.

Both of these tests are considered practical primarily with concrete that is less than one year old. However, when used in conjunction with core sampling, these tests can also be used to determine significant differences in concrete strength of older bridges.

Ultrasonic Testing

Ultrasonic testing can provide valuable information regarding the condition of concrete bridge members. However, the method can be difficult to use with reinforced concrete members, and some skill is required to obtain usable results.

Large cracks and voids can be detected since the path of the pulse travels around any cavity in the concrete and time of transmission is therefore lengthened. The presence of steel parallel to the line of transmission provides a path along which the pulse can travel more rapidly, causing misleading results. Therefore, it is generally desirable to choose paths that avoid the influence of reinforcing steel.

Smart Concrete

Carbon fiber-reinforced cement can be used as a strain-sensing coating on conventional concrete. This coating allows the sensing of strain similar to strain gauges. The resistance can be measured by having electrical contacts attached to the member.

Strain gauges are expensive compared to the structural material, and they often become detached during use. This method could be much more reliable in sensing strain in structures.

Smart concrete is in early stages of development.

Radiography

Radiographic inspection is a nondestructive testing technique used to evaluate concrete for signs of hidden flaws which could interfere with its function. It is accomplished with the use of radiographs, images generated by bombarding the concrete under inspection with radiation. X-ray and gamma ray radiographic inspection are the two most common forms of this inspection method.

Carbonation

Carbonation of concrete is the result of the reaction of carbon dioxide and other acidic gases in the air, and it can cause a loss of protection of the reinforcing steel against corrosion. The depth of carbonation in a concrete bridge member can be measured by exposing concrete samples to a solution. Uncarbonated concrete areas change color, while carbonated concrete areas remain colorless.

15.2.3 Other Testing Methods

Core samples can be used for many of the following other advanced inspection tests. Usable cores can normally be obtained only if the concrete is relatively sound. If possible, cores need to have a diameter three times the maximum aggregate size. Core holes need to be filled with non-shrink concrete grout. Since removing a concrete core may weaken the member, exercise caution and do not remove from high stress areas.

Concrete Permeability

Air and water permeability can be measured by drilling a small hole into the concrete, sealing the top with liquid rubber, and inserting a hypodermic needle. Air permeability can then be determined by filling the hole with water and measuring the flow in to the concrete at a pressure similar to that of rainfall. However, this method is seldom used in bridge inspections.

Concrete Strength

Actual concrete strength and quality can be determined only by removing a concrete core and performing such laboratory tests as:

- Compressive strength
- Cement content
- Air voids
- Static modulus of elasticity
- Dynamic modulus of elasticity
- Splitting tensile strength

Endoscopes and Videoscopes

Endoscopes and videoscopes are viewing tubes that can be inserted into holes drilled into a concrete bridge member (see Figure 15.2.13). Light can be provided by glass fibers from an external source. Some applications of this method include the inspection of the inside of a box girder and the inspection of hollow post-tensioning ducts. Although this is a viewing method, it is considered to be a destructive method because some destruction is necessary for its proper use in concrete.

Figure 15.2.13 Remote Video Inspection Device

Moisture Content

Moisture content in concrete serves as an indicator of corrosion activity. Moisture content can be determined using nuclear methods (refer to Topic 15.2.2) or from concrete samples taken from the bridge and oven dried in a laboratory

Petrographic Examination

Petrographic examination is a laboratory method for determining various characteristics of hardened concrete, which are useful in determining the existing condition and predicting future performance. This advanced inspection method is able to detect Alkali-Silica Reaction (ASR) products.

Reinforcing Steel Strength

The actual properties of reinforcing steel can only be determined by removing test samples. Such removal of reinforcing steel can be detrimental to the capacity of the bridge and needs to be done only when such data is essential.

Chloride Test

One of the current standard test methods used to assess the resistance of concrete to penetration of chloride ions is the rapid chloride permeability test. This test, officially known as AASHTO T 277-93, “Electrical Indication of Concrete’s Ability to Resist Chloride,” measures the charge passed through a concrete specimen subjected to sixty volts (direct current) for six hours. Variable results have been reported with the rapid chloride permeability test when certain mineral admixtures such as silica fume were included in the concrete mixture and when calcium nitrite (included in some corrosion inhibitors) or reinforcing steel have been present. The test specimens are two inches long and four inches in diameter in the rapid chloride test. The rapid chloride test uses sodium hydroxide ponded on the top of the specimen, and a solution of sodium chloride at the bottom of the specimen. The specimen is initially subjected to thirty volts (direct current), and the resulting current determines the voltage to be applied for the duration of the test. The voltage is applied for three different time periods varying anywhere from 2 to 96 hours. Following the test, the specimen is split in half and a silver nitrate spray is applied to identify the depth of chloride penetration in to the specimen.

ASR Evaluation

One test for ASR evaluation, often referred to as the accelerated mortar bar test, has been accepted by ASTM and AASHTO. The test involves casting mortar bars that contain the subject aggregate (either coarse or fine), which is processed to a standard gradation. The mortar bars are then removed from their molds after 24 hours and placed in water at room temperature. The temperature of the water is then raised to 176 degrees Fahrenheit in an oven, and the mortar bars are stored in this condition for the next 24 hours. After the bars are removed from the water, they are measured for initial length and then submersed in a 1 normal (N) NaOH solution at 176 degrees Fahrenheit, where they are then stored for 14 days. Length change measurements are made periodically during this storage period. The total expansion at the end of the 14-day soaking period typically is used in specifications, although the expansion limits specified by different agencies vary.

Another method is a qualitative ASR field test that utilizes colored dyes. This test is performed on a broken surface of a concrete core, where reagents are then applied. If ASR is present, the reagents turn different colors indicating if ASR is just beginning or if ASR is in an advanced stage. This field test is relatively inexpensive and can be carried out completely on-site with easy-to-interpret results.

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Chapter 15
Advanced Inspection Methods

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Topic 15.3 Steel

15.3.1 Introduction

Advanced inspection methods give inspectors the ability to further evaluate suspected deficiencies found during a visual inspection. They can also be used to perform inspections on members that are not accessible. Advanced inspection methods usually require calibrated testing equipment, a professionally trained technician to perform the testing and a professional that has expertise in interpreting the advanced inspection results. However, bridge inspectors will have an understanding of the various advanced inspection methods.

There are two main classifications of advanced inspection methods. The first is labeled nondestructive testing or nondestructive evaluation (NDT or NDE). This classification pertains to all advanced inspection methods that do not impair the usefulness of the member being tested. Other testing, the second main classification, covers all advanced inspection methods that affect or destroy the structural integrity of the member being tested.

New technology is making the use of these highly technical systems economically feasible for bridge inspection. From this fact, advanced inspection methods are becoming more popular for the inspection of bridge members. Current and future studies have been, and will be focusing directly on relating results from advanced inspection methods into Bridge Management Systems ratings.

This Topic describes the different types of nondestructive and other test methods for steel bridge members and the general methods for each.

15.3.2 Nondestructive Testing Methods

Acoustic Emission Testing

Acoustic emission (AE) testing has been used for many years, but is now becoming a more standardized and available method.

This inspection method detects elastic waves generated by the rapid release of energy from within a test object by such mechanisms as plastic deformation, fatigue and fracture. When a structure is under certain load levels, it will produce an acoustic sound that ranges between 20 KHz and 1 MHz. The sound that is generated is known as acoustic emissions. Acoustic emission testing uses ultrasonic microphone to listen for sounds from active deficiencies and is very sensitive to deficiency activity when a structure is loaded beyond its service load in a proof test. This process can detect flaws and imperfections such as the initiation, growth and growth rate of fatigue cracks in steel structural members, friction, corrosion, deformation, cracks opening and closing, weld discontinuities, the failure of bonds, fibers and filaments in composite materials and the appearance of potentially hazardous flaws in metal or synthetic pressure vessels.

Most sounds produced by materials under stress are inaudible; however there may be a portion that exists as audible sound, based on the magnitude and type of deformation, flaw growth or failure.

Bridges contain a large number of joints, welds and connections that are potential initiation points for fatigue cracks. Acoustic emission monitoring is used for early detection of fatigue cracks in fracture critical bridge members and to monitor the relative activity of existing fatigue cracks. Advanced signal processing and correlations to parametric measurements are used to separate noises generated by dynamic loading, loose connections, rivets and crack growth (see Figure 15.3.1).

Commercial systems are available, based on wave propagation properties. When energy is released (for example: high-tensile wire failures or concrete cracks), waves propagate in the material. Acoustic sensors distributed along the structure can detect and record the signal. Computer processing of the signal will then provide valuable information about the event including: location, origin, energy, and frequency (see Figure 15.3.1).

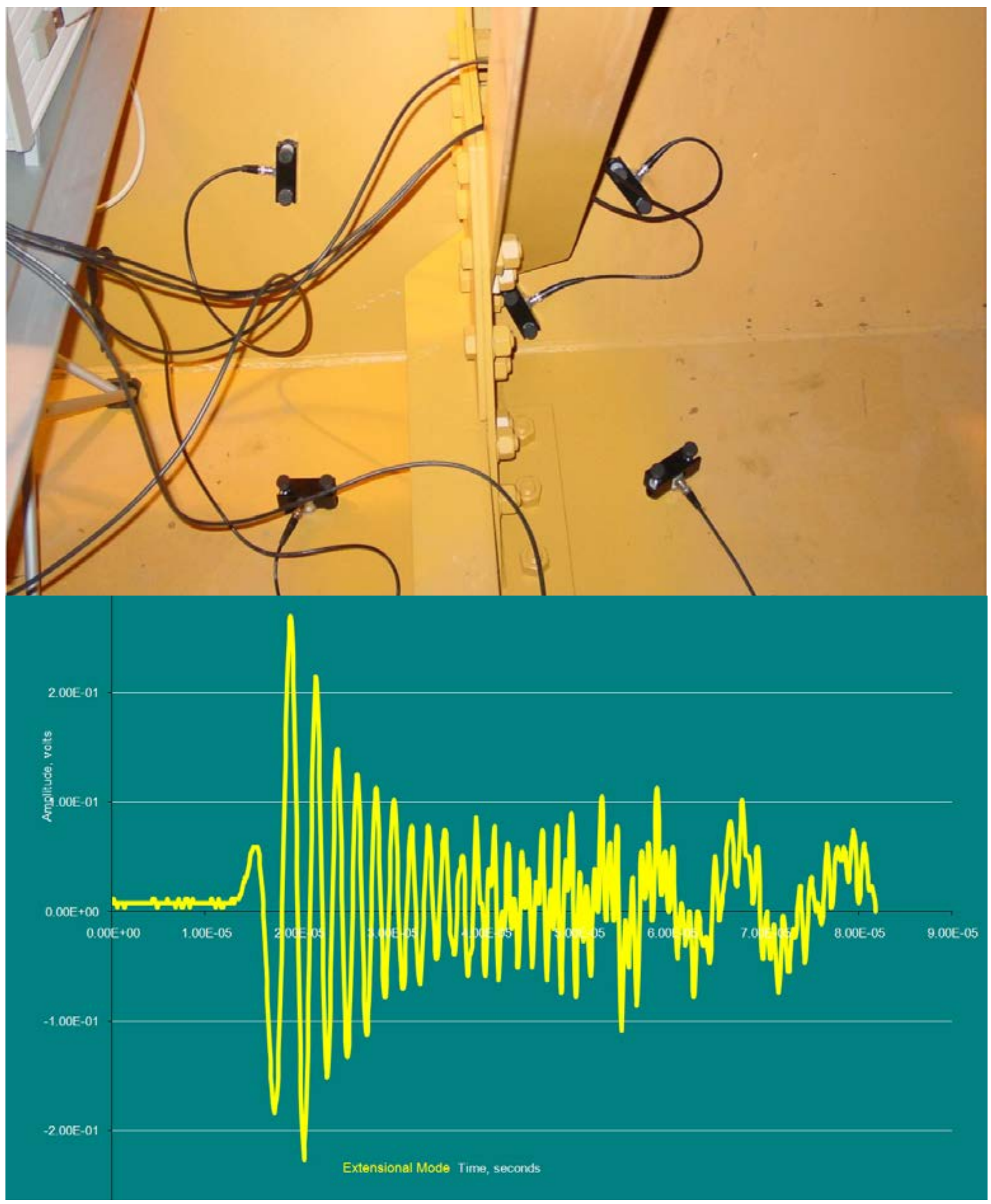

Figure 15.3.1 Acoustic Sensors Used to Determine Crack Propagation

The main advantage of these systems is the recording and real-time analysis of the waves themselves, allowing automatic filtering by the acquisition unit according to preset criteria. The events of interest are stored in the acquisition unit and automatically available for analyses.

Devices can be used to monitor areas that already are cracked or cracked areas that have been retrofitted. The device is a portable, modular multi-channel system that can be mounted close to the area being monitored (see Figure 15.3.2). The system can be directly connected to a computer or it can be accessed through wired or wireless modems for data collection.

Figure 15.3.2 Inspector Using Acoustic Emissions to Determine Crack Propagation

Limitations of acoustic emission testing include AE being a non-repeatable test. Once a test is completed, it cannot be repeated due to the flaw growth being an irreversible process. Also, deficiencies are not detectable if they are not growing or flexing. Therefore, if the deficiency is not increasing in size, acoustic emission testing will not be able to locate the crack. The bridge may also cause interference with testing. If background noise exists, that noise would be similar to the sound energy released by a flaw. For this reason, the wires need to be properly shielded against background noise. Lastly, the acoustic emission testing unit is relatively expensive and also requires the additional cost of an operator.

Corrosion Sensors

Corrosion sensors are being developed that use environmental variables such as dirt and duration of wetness to indicate the degree of corrosion of a steel structure.

Smart Coatings

The National Science Foundation’s Advanced Technology for Large Structural Systems (ATLSS) Engineering Research Center has developed “Smart Paint” – paint with microencapsulated dyes that outline a fatigue crack in a bridge or other highway structure as the crack forms and propagates.

Japanese scientists have also developed paint that sends out electrical signals which are picked up by electrodes placed on either side of the paint's resin layer if the structure or material begins to vibrate. The greater the vibration, the greater the electrical signal. This paint could enable engineers to monitor vibrations throughout the lifetime of a structure, allowing them to calculate much more accurately when fatigue is becoming a problem. The new paint is a much easier way of measuring vibrations than conventional strain gauges.

Dye Penetrant

A dye penetrant test (PT) can be used to define the extent and size of surface flaws in steel members (see Figure 15.3.3). The test area is cleaned to bare metal to remove all contaminants, a penetrant is applied to the surface by spray or brush, and excess penetrant is removed by wiping or water rinsing. Once the penetrant has dried, a white developer is applied, which draws the dye out of the irregularities and defines the extent and size of surface flaws. Bridge inspectors commonly use this method since it does not require extensive training or expensive equipment. A limitation of this method, however, is that it reveals neither the depth of cracks nor any subsurface flaws. Another important factor when performing dye penetrant testing is the penetrant dwell time. This is the amount of time that the penetrant is allowed to remain on the surface before the excess is wiped off. Factors that effect the dwell time include:

- Temperature of the member being tested and the penetrant type
- Ambient air temperature (higher temperatures require shorter dwell times)
- Humidity (low humidity causes penetrant to dry out rapidly)
- Size and shape of the discontinuity (hairline cracks need more time than large ones)
- Material type
- Penetrant removal type and manufacturer's recommendations

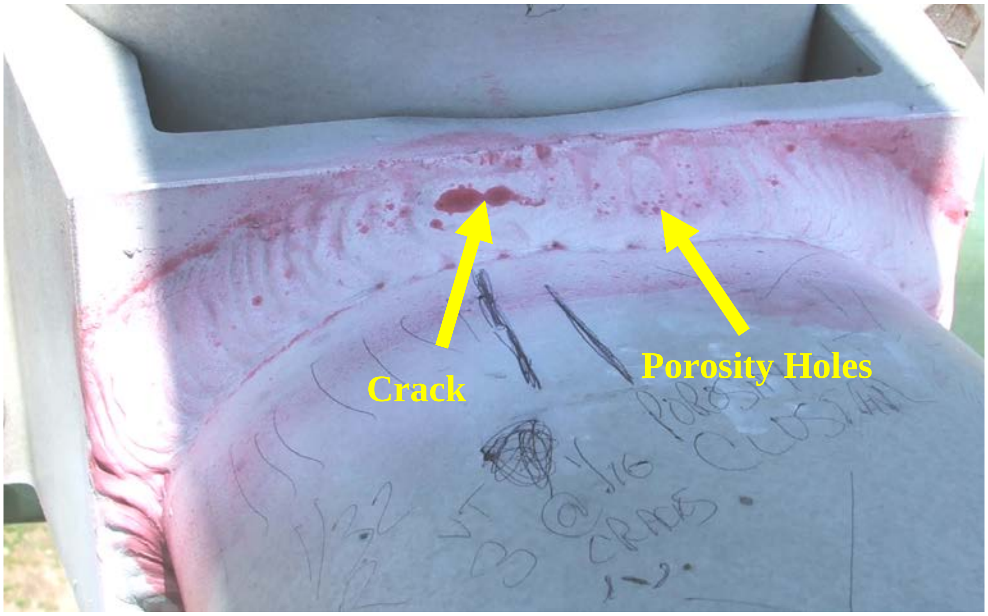

Figure 15.3.3 Detection of a Crack Using Dye Penetrant

Limitations of dye penetrant testing include only showing what defect is on the surface and nothing further in on the specimen. For a proper reading, clean the surface, which includes the paint. In order to see if there is a crack present, the inspector needs good visual acuity. In addition, a recommended temperature of over 40 degrees Fahrenheit is required to achieve acceptable results using dye penetrant testing.

Magnetic Particle

Magnetic particle testing is useful in detecting surface gouges, cracks, and holes in ferromagnetic materials. It can also detect subsurface deficiencies, such as voids, inclusions, lack of fusion, and cracks, which lie near the surface. Magnetic particle inspection is primarily used to find surface breaking flaws (see Figure 15.3.4) and can also be used to located subsurface flaws. Its effectiveness, however, diminishes quickly depending on the depth and type of flaw. The method consists of magnetizing the member, applying iron filings, and then interpreting the pattern formed by the filings, which are attracted by the magnetic leak.

A magnetic field is induced into the member, and cracks or other irregularities in the surface of the member cause irregularities in the magnetic field (see Figure 15.3.5). This method is also referred to as magnetic field disturbance.

Figure 15.3.4 Magnetic Particle Device Used to Detect Subsurface Flaws

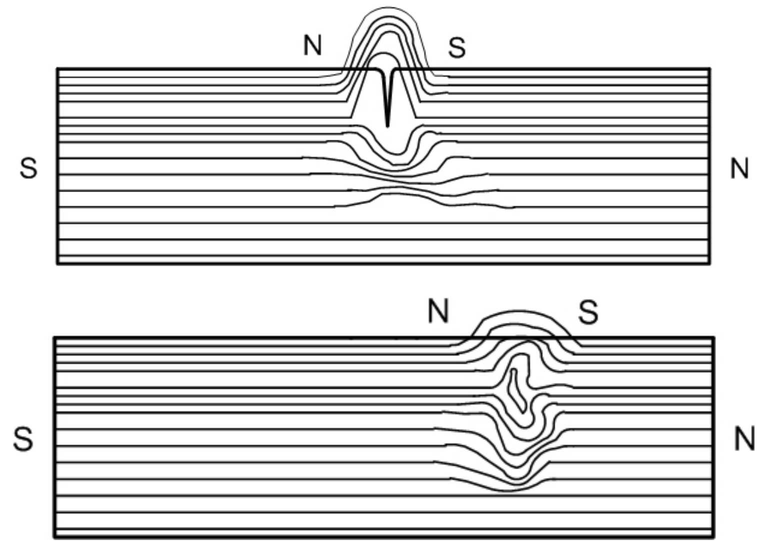

Figure 15.3.5 Schematic of Magnetic Field Disturbance

Limitations of magnetic particle testing include applicability only for members composed of a ferromagnetic material. For some test pieces, removal of the residual magnetism is necessary for an additional expense. The magnetic field requires perpendicular orientation to the principle plane of the defect for detection. In addition, smaller subsurface deficiencies are generally harder to detect than larger deficiencies. Lastly, clean unpainted surfaces help to ensure the maximum sensitivity of the magnetic particle testing unit.

Radiography Testing

Radiography testing (RT) is used to detect and locate subsurface deficiencies such as cracks, voids, and inclusions throughout the internal structure of the material in the fabrication shop and in the field.

Radiography testing requires that the inspector have access to both sides of the structure, with the radiation source on one side and the film on the other side. X-rays or gamma rays are passed through the member and are absorbed differently by the various flaws. When a piece of radiographic film is exposed to the rays, the deficiencies appear as shadows on the film (see Figures 15.3.6). This type of advanced inspection is typically used for full penetration groove welds during fabrication and construction.

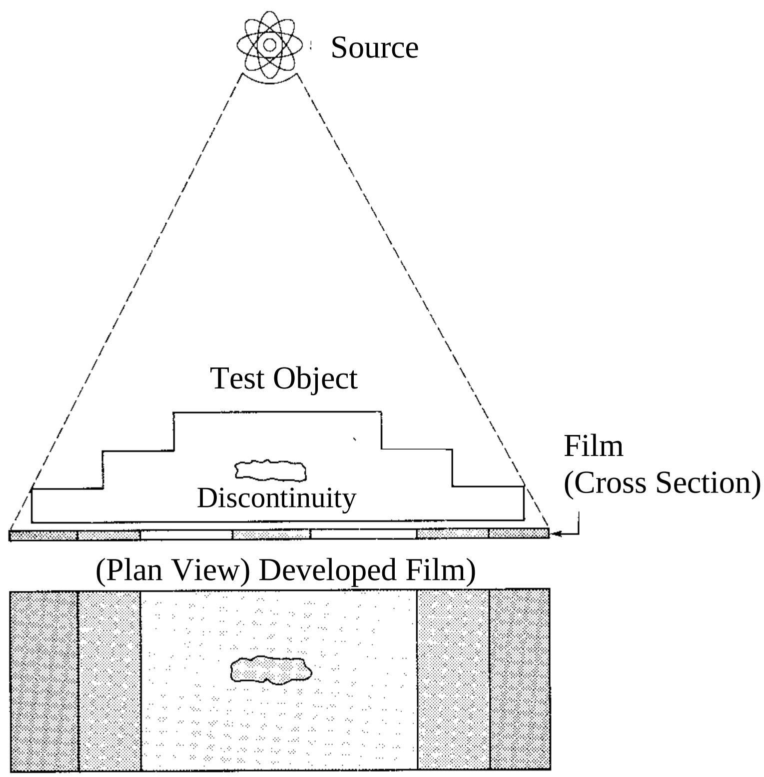

Figure 15.3.6 Radiography Testing

Limitations of radiography testing include requiring access to both sides of the test member for a proper reading. It is necessary to have a high skill level to interpret the readings. Lastly, radiography testing is relatively expensive, especially for thicker members. The cost of an operator needs to be factored into the total cost of radiography testing.

Computed Tomography

Computed tomography uses X-ray and gamma radiation to visualize and produce 2-D and 3-D cross-sectional images of the interior deficiencies of a steel member. The image is captured by a detector array, it is processed by a computer, and it is then reconstructed. This method is similar in many ways to medical CAT scans, and it has great potential for locating discontinuities of all types in steel members (as well as concrete members).

Robotic Inspection

Several companies are currently developing and marketing systems which use high-resolution video cameras on robotic arms attached to permanent falsework underneath the bridge. By remote telescanning, details can be visually monitored, with magnification if needed, without the inspector having to climb to gain access to a detail each time an inspection is desired. While the primary material application for robotic inspection is steel, it can also be used on timber and concrete bridges.

In recent years, the California Department of Transportation (Caltrans) has been working on an aerial robotic inspection system. This system, in the testing and development stage, can allow bridge inspectors to view elevated bridge members from the ground. It is controlled by a remote control that is connected to the system through a 100 ft electrical cord. A fiber optics cable transfers information and images from the aerial device to the ground station. This type of inspection may reduce traffic delays and increase the level of safety for motorists and bridge inspectors.

There have also been robotic inspections performed by unmanned vehicles. Since Hurricane Ike in 2008, the Texas Transportation Institute and the Texas Department of Transportation (TxDOT) gave permission to the Center for Robot-Assisted Search and Rescue for bridge inspection using robotic means. This is achieved by using an unmanned surface vehicle and underwater vehicles (see Figure 15.3.7) that were used to inspect the substructures. The surface vehicle is battery powered and can run for four to six hours based upon the current. It contains an acoustic camera for subsurface inspections and three video cameras to record above the water. The underwater inspection robots can be used for the underwater visual inspection of bridges and debris and mapping debris fields.

Figure 15.3.7 Robotic Inspection: Unmanned and Underwater Inspection Vehicles

Ultrasonic Testing

Ultrasonic testing is frequently used in steel applications and can be used to detect cracks in flat, relatively smooth members, as well as pins by using high-frequency sounds (range of 20 KHz to 25 MHz) pulsed through a material to generate images (see Figure 15.3.8). It can also be used to measure the thickness of steel members, providing detailed information concerning loss of cross section. Ultrasonic testing also has many applications in the inspection of welds, detecting porosity, voids, inclusions, corrosion, cracks, and other discontinuities. This method will involve applying a couplant to the area that is to be inspected and then scanning the area with a transducer, which is attached to the UT machine. Refer to Topic 15.1 for further details about the principles of ultrasonic testing.

Figure 15.3.8 Ultrasonic Testing of a Pin in a Moveable Bridge

Limitations of ultrasonic testing include inaccurate readings for members with a rough surface or complicated geometry. Parallel plates or angles (including built-up members) with a small gap between the elements may also produce inaccurate readings. In additional, flaws that are parallel to the sound waves will not be detected. Skilled operators are required to administer ultrasonic testing, adding to the cost of this NDE method.

Ultrasonic thickness depth meters (D-meters) are miniature versions of an ultrasonic tester which uses a dedicated straight beam transducer (see Figure 15.3.9). The primary difference between an average ultrasonic tester and a D-meter is that ultrasonic testers can determine internal flaws while D-meters can only detect the thickness of the part being tested.

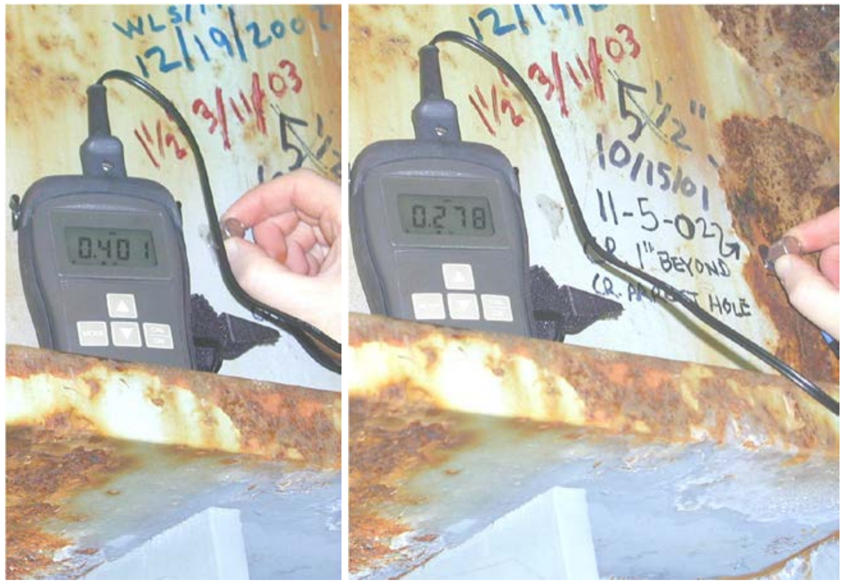

Figure 15.3.9 Ultrasonic Thickness Depth Meter (D-meter)

Portable ultrasonic testing (UT) or phased array units are another form of ultrasonic testing that can be used to test for discontinuities on steel members (see Figure 15.3.10). They are arrays that consist of a series of individual elements, transducers, that are separately pulsed, time delayed and processed. Software will allow the operator to modify the beams time delay or phasing. The portable UTs and phased arrays can be controlled electronically to scan, sweep, steer, and focus the beam.

Figure 15.3.10 Ultrasonic Testing of a Gusset Plate Using a Portable UT

Advantages of this type of testing include considerably faster scanning rates (5 to 10 times faster) compared to traditional ultrasonic testing. Multiple angles and frequencies also produce better images, which results in less user-interpretation required by the operator.

Limitations of portable units include some uncertainty in the technology, as this method is relatively new and not completely proven. For this reason, this method may not be universally accepted by bridge owners as a legitimate way to test for deficiencies. Training courses are also required, due to the difficulty in using this equipment. Lastly, the units are expensive and require an additional cost for a qualified operator.

Eddy Current

This type of electromagnetic method uses AC currents. Eddy current testing (ET) can only be performed on conductive materials and is capable of detecting cracks and flaws as well as member dimensions and variations. This method can be used on painted or untreated surfaces. The system works by monitoring the voltage across a coil that has an AC current flowing through it. When the coil is placed next to the conductive member, the member produces eddy currents that flow opposite to the direction of flow from the coils. Deficiencies in the member disturb the eddy currents, which, in turn, affect the induced current. The affected induced current is monitored through the voltage across the coil. Eddy current testing devices can be hand held devices (see Figure 15.3.11).

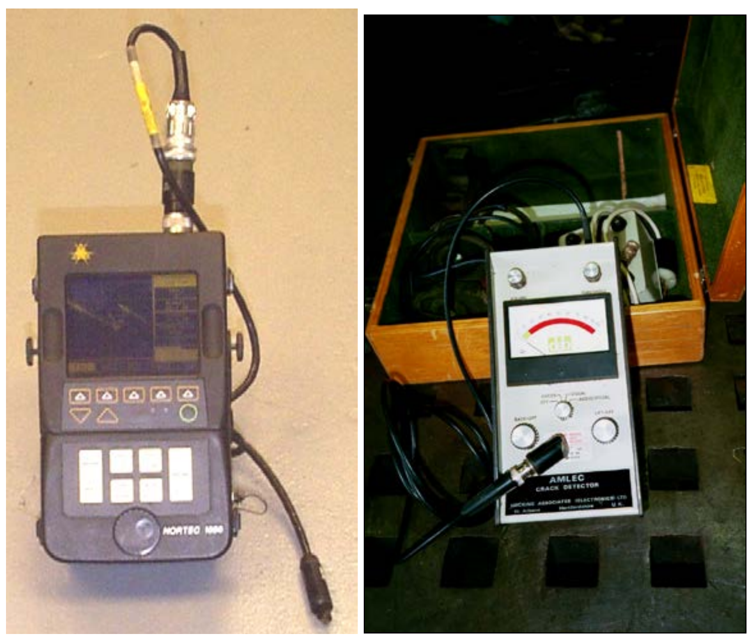

Figure 15.3.11 Hand Held Eddy Current Testing (ET) Instruments

Limitations in eddy current testing include the inability to determine the depth of the crack. This NDE method also does not work with galvanized steel members. Lastly, eddy current testing requires operators with the proper training to correctly interpret the test results, which adds to the cost of the method.

Electrochemical Fatigue Sensor (EFS)

Electrochemical Fatigue Sensors (EFS) is a new nondestructive evaluation method that is used to determine if actively growing fatigue cracks are present in the steel. Data collection and analysis software is provided within the EFS system. The system also consists of an electrolyte, sensor array, and potentiostat. These components are used to apply a constant polarizing voltage between the bridge and the sensor. The sensor is placed near the suspected fatigue crack location on the bridge and then injected with the electrolyte. A small voltage is then applied. The current response of the sensor array, comprised of a crack measurement sensor and a reference sensor, is collected, analyzed and compared with the software. The software will automatically indicate the level of any fatigue crack activity (see Figure 15.3.12).

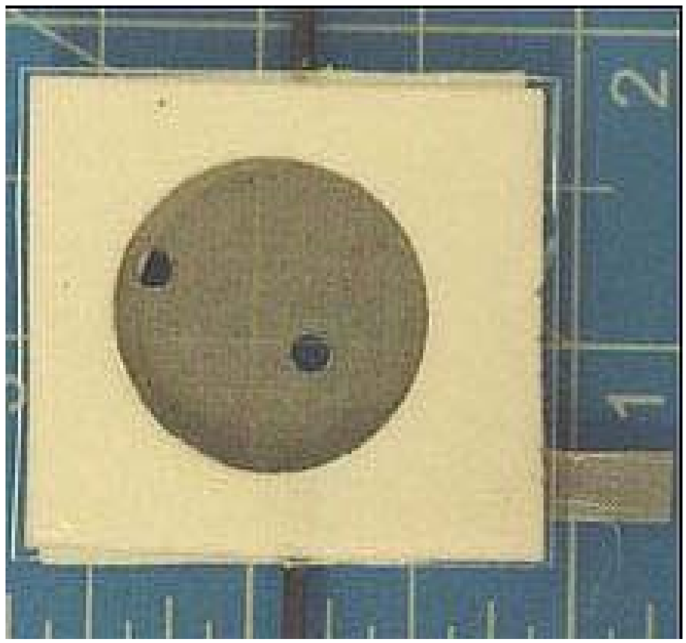

Figure 15.3.12 Electrochemical Fatigue Sensor

Magnetic Flux Leakage

Magnetic flux leakage testing, similar to magnetic particle testing, is a form of nondestructive evaluation developed in Great Britain that has been used over the past few decades on stay cables. This method uses a magnet to magnetize the steel to help detect any corrosion and pitting in steel. Any crack will be detected by sensors that will pick up any distortion of the magnetic field which causes the field to leak from the rope. Various types of sensors, placed close to the rope, are used to help sense and measure any magnetic flux leakage. The types of sensors include coils, Hall sensors or fluxgate sensors.

Laser Vibrometer

Laser vibrometers are used to measure small non-contact vibrations of a stay cable from a large distance. Nothing special needs to be placed or done to the cable. Instead, a low-power laser beam is used, directed at the cables. The response that is measured will be vibration amplitude and frequencies which will be used to determine any vibration of the cables. Those frequencies detected will then be used to calculate the forces that the vibrations are placing upon the cables.

Information concerning various nondestructive testing can be found on the American Society of Nondestructive Testing website: **www.asnt.org**.

15.3.3 Other Testing Methods

Strength tests are considered destructive since they usually involve removing pieces of steel from the bridge. Small pieces cut out of steel members are called test "coupons." The removal method and coupon size have to be suitable for the planned tests. If a coupon is required, consult the bridge engineer to determine the most suitable area of removal. For instance, an inspector will not remove a coupon from the web area over a bearing. An inspector will not recommend removal of a coupon from a high stress zone such as the bottom flange at midspan. Tests may be necessary to determine the strength or other properties of existing iron or steel on bridges for which the steel type is unknown.

The following tests can be conducted only by the destructive method of removing a sample and evaluating it in a laboratory.

Brinell Hardness Test

The Brinell hardness test measures the resistance to penetration of the steel. A hardened steel ball is pressed into the test coupon by a machine-applied load. The applied load and the surface area of the indentation are used to calculate the hardness of the steel. For steel that has not been hardened by cold work, its hardness is directly related to its ultimate tensile strength.

Charpy Impact Test

An impact test determines the amount of energy required to fracture a specimen. A common impact test for steel coupons is the Charpy V-notch test (see Figure 15.3.13). A notched test coupon is placed in a vise, and a hammer is then released from an elevated position, swinging down and hitting the coupon. Since the force of the hammer is concentrated in a notch in the coupon, the stress goes into fracturing the specimen and not into strain. The energy required for fracture is determined based on the mass of the hammer and the distance that it fell and is recorded on the dial located on the striking hammer. This test can be performed at different temperatures to determine if the steel is susceptible to brittle failure.

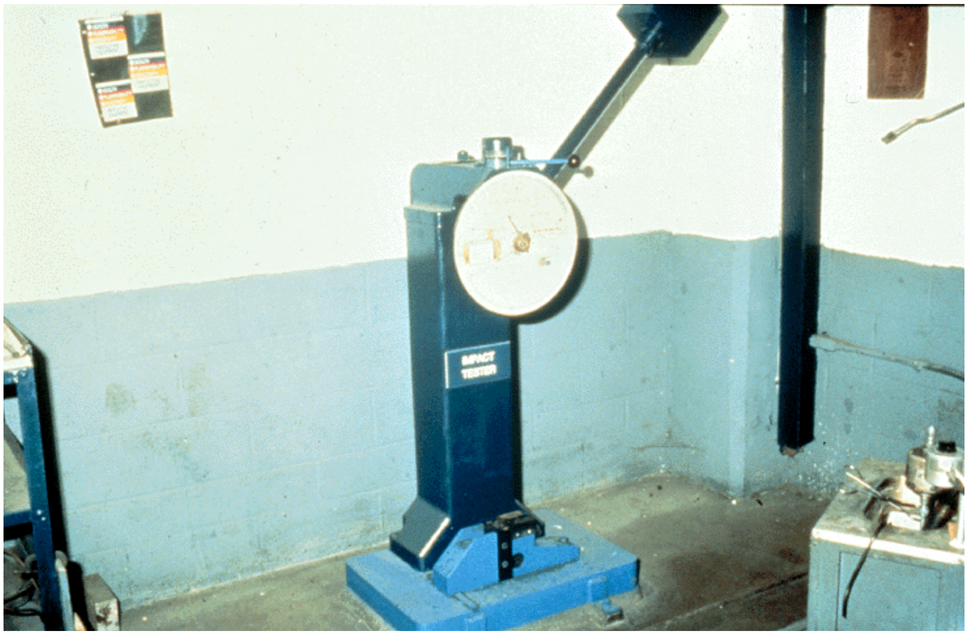

Figure 15.3.13 Charpy V-Notch Test

Chemical Analysis

The chemical composition of the steel is an important indication of whether a weld will crack, either from cold cracking or hot cracking. Tests can be performed on coupons to determine the chemical composition of the steel.

Cold, or delayed, cracking can be approximated using a carbon equivalent (C.E.) equation that is based on the chemical composition of the steel. One such equation, based on the relative proportions of various elements in the steel, is presented in the ASTM A706 rebar specification:

$$C.E. = C\% + \frac{Mn\%}{6} + \frac{Cu\%}{40} + \frac{Ni\%}{20} + \frac{Cr\%}{10} - \frac{Mo\%}{50} - \frac{V\%}{10}$$

C – Carbon	Cr – Chromium
Mn – Manganese	Mo – Molybdenum
Cu – Copper	V – Vanadium
Ni – Nickel	

When the C.E. is below 0.55, the steel is generally not susceptible to cold cracking, and no special precautions are required for welding. However, when the C.E. is above 0.55, the steel is susceptible to cold cracking, and special precautions are required for welding.

Hot cracking occurs as the weld begins to solidify. Hot cracks have almost been eliminated today due to modern welding material formulation.

Tensile Strength Test

The tensile strength is the highest stress that can be applied to the coupon before it breaks Once the test is complete, the tensile strength of the steel can be easily determined. See Topic 5.1, Bridge Mechanics.

The ends of the test coupon are placed in vises on a testing machine. The machine then applies a tensile load to the ends of the coupon. The machine measures the load at which the coupon fails or breaks. This load and the cross-sectional area of the coupon determine the tensile strength of the steel.

Brittle fractures occur without plastic deformation once the yield strength is exceeded. Since there is no plastic deformation, there is no warning that a fracture will occur. The fracture that is formed on a brittle fracture will be flat (see Figure 15.3.14).

Ductile fractures occur once the yield strength has been exceeded, causing the specimen to elongate and "neck down" (also known as plastic deformation) and eventually breaking if the load is not removed (see Figure 15.3.15). Plastic deformation results in distortion of the member, which will provide a visual warning before the member would fracture. The reduced cross section is caused by plastic distortion rather than section loss. The fracture produces shear lips that are tilted at 45 degrees.

Figure 15.3.14 Brittle Failure of Cast Iron Specimen

Figure 15.3.15 Ductile Failure of Cold Rolled Steel

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Topic 15.4 Advanced Bridge Evaluation

15.4.1 Introduction

Today's sensing devices capture and report highly accurate and objective data, which can be used to make "fact-based" evaluations for bridge condition and provide decision support for serviceability, repair or replacement actions, optimizing the owner's overall bridge management plan.

Advanced bridge evaluation technologies allow the owner to more objectively capture and evaluate known or suspect deficiencies found during a visual inspection. They may also be used to perform periodic or continuous inspections on members that are not readily accessible. Advanced bridge evaluation technologies usually require customized hardware and software, an experienced technician to install sensing devices or perform the testing, and an engineering professional that has expertise in interpreting the results (see Figure 15.4.1). Generally, bridge inspectors and engineers have a basic understanding of the advanced bridge evaluation technologies available. This basic knowledge allows them to participate in the selection and use of the appropriate technologies to better determine the bridge's condition.

Figure 15.4.1 Installation of Sensors

There are two main classifications of advanced bridge evaluation technologies. The first is known as nondestructive evaluation (NDE). This classification pertains to technologies that do not impair the usefulness (short term or long term) of the member being tested. The other classification consists of advanced bridge evaluation technologies that negatively affect the member by reducing the structural integrity of the member being tested. For example, removing a part of a member for testing therefore reduces its capacity. Most practitioners and owners today prefer the nondestructive technologies for obvious reasons.

The proper use of these advanced bridge evaluation technologies can supplement routine bridge inspections and can be useful for optimizing an owner's bridge management program. Methods are being developed to transfer near real-time results from these technologies directly into Bridge Management Systems ratings and bridge management protocols (e.g. overload permitting.) (see Figure 15.4.2).

Near real-time solutions are made possible by the combination of a variety of sensing devices, wireless communication and internet technologies. The ability to capture data on member strains, relative movement between members, crack growth and propagation, and other relevant structural parameters are the result of digital technology being applied to structural bridge evaluations.

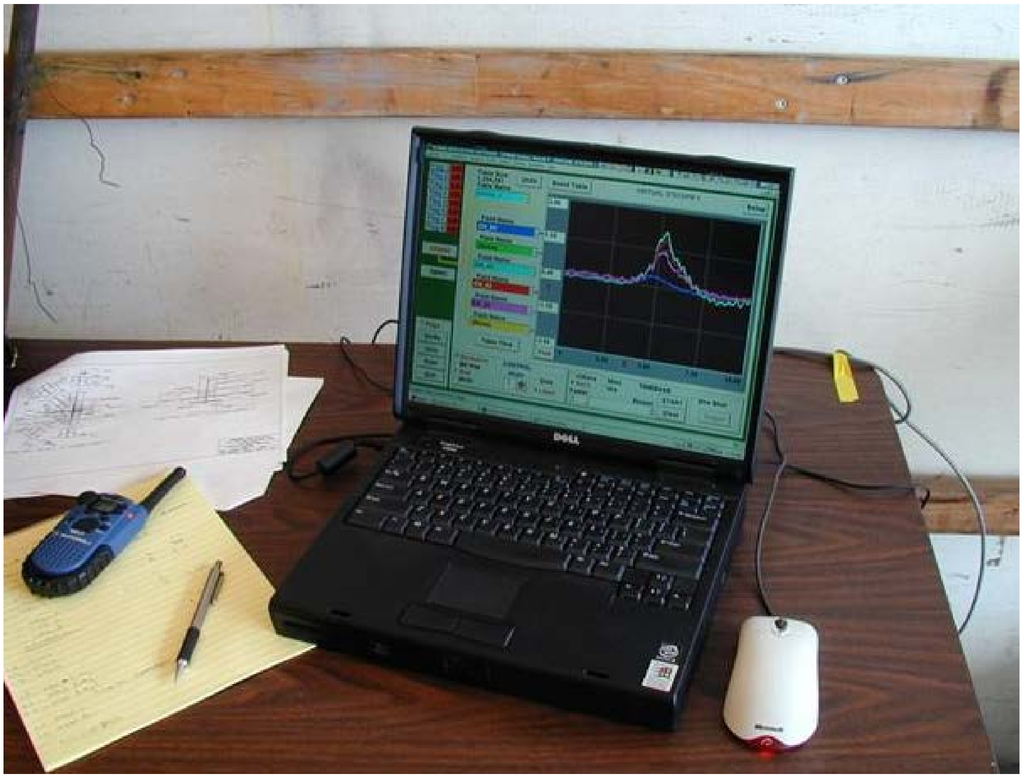

Figure 15.4.2 Viewing Real Time Data

15.4.2 Advanced Bridge Evaluation Methods

Strain or Displacement Sensors

Strain or displacement sensors can be used to monitor the response of a member to a live load and/or temperature changes. These type sensors are available and include foil type, vibrating wire, fiber optic, and a sensor that measures both current and peak strains in one device (see Figure 15.4.3). Foil-type sensors are only used in the axial direction of flat members. Single wire filament "vibrating wire" sensors can be used on flat members or cables. Portable strain reading instruments can be used to monitor sensors from a central location on or near the bridge in a manual data collection mode or fully automatic monitoring can be installed, allowing readings to be taken at user defined intervals and sent wirelessly to a central location for viewing over the internet.

Figure 15.4.3 Strain Gage Used on the Hoan Bridge, Milwaukee, Wisconsin

Locations for strain sensors are selected based on the condition of individual members, accessibility, and the objectives of the monitoring program. Strain/displacement sensors can provide valuable information about:

- The actual transverse load distribution through the structural member
- The load sharing between elements of a multi-element member
- The effectiveness of the various members of the primary structural system
- The influence of deteriorated or defective members
- The growth and/or propagation of cracks in steel or concrete members
- The relative movement of members to fixed points due to loss of section (chemical) or load-induced deterioration

The principal use of strain sensors today is to ascertain the actual condition of a member or series of members and use that information to infer the safe load-carrying capacity of the structure. In essence, sensors are used to provide data to allow additional decision making combined with a visual inspection. Strain sensor data can be used to ascertain the weight of vehicles crossing a bridge. This is known as a "weigh-in-motion" system.

Sensing devices, coupled with electronic control equipment, can be used to update owners and bridge inspectors about ongoing deterioration of the structure. Such configured solutions, which can be integrated into one system, generally include strain or displacement sensors, a system controller on the structure, wireless data transmission, customized software, and other features. This allows for secure data capture, data graphing, viewing over the internet and alerts (by e-mail, cell phone or other method) if strains or displacements exceed predetermined values.

Other Available Sensors

To complement strain or displacement sensors, newer sensors are being developed and deployed to enhance bridge evaluation. Typical sensing devices include tiltmeters (foundation movements), accelerometers (earthquake-induced movements), temperature and humidity sensors, and even global positioning satellite (or GPS) systems to monitor movement of piers, towers, and decks on long bridges to an accuracy of 3/16 of an inch. Other, more esoteric sensing devices include those to detect onset of fatigue cracking, actual stress in cables via electromagnetic fields, corrosion, and other member condition parameters.

Generally speaking, price and functionality are directly related. That is, sensors meant to be used in outdoor environments for long periods of time (years) are more expensive than those meant for controlled environments (laboratories) or short duration use (weeks). Sensing devices can be utilized individually or as part of a system that is configured to provide a total solution. Specialized personnel are required to integrate the variety of sensing devices with controller hardware and software for advanced bridge evaluations.

Dynamic Load Testing

In recent years, an increasing number of short-span bridges have been evaluated using measured response data from known loads. These bridge evaluations have provided useful information and, in some instances, have revealed bridges that required closure or restrictions and those that could be safely upgraded (load restrictions removed).

Use of this method involves a combination of strain sensors, on-site data capture, and response modeling. A known load (weighed dump truck) is driven across a short-span bridge with no other traffic (see Figure 15.4.4). GPS technology is used to precisely spot the truck's position while strain sensors capture member displacements/strains. Data capture typically occurs in one day or less. The data is then used to "build" a rudimentary structural model for evaluation of actual load-carrying capacity. The model is fitted to the actual structural response, allowing engineers to determine actual load-carrying capacity.

This technology gains advantage over current load capacity protocols in that it can consider composite action of the members and contributions to load-carrying capacity from other structural components (sidewalks and parapets) that are typically ignored with traditional analysis methods. Dynamic load testing has been used for over twenty years and has proven its ability to provide accurate load-carrying capacity determinations.

Figure 15.4.4 Dynamic Load Testing Vehicle

System Identification

Using actual structural response data, the properties of the structure (e.g., areas and moments of inertia of structural members) can be calculated. The process of building a structural model from response data is called system identification. The primary use of system identification in structural engineering has been for earthquake engineering research. The historical accuracy achieved in this advanced bridge evaluation methodology indicates that system identification can also provide a tool for detecting unseen structural flaws.

System identification can be performed using a variety of response data, such as modal and time history response. For modal response, the frequencies and mode shapes of the structure are obtained either from ambient vibration data or from the results of harmonic excitation. A time history response is the response (i.e., displacements or acceleration) of one or more points on the structure as a function of time due to a known loading function. For either type of response data, the results are used to determine structural parameters representing the structural integrity of the bridge.

Initially, system identification is used to create a structural model, which accurately represents the in-service condition of the structure (see Figure 15.4.5). Subsequent analyses are then performed to determine which parameters are changing. Since the parameters represent structural properties (e.g., areas and moments of inertia), the changes are indicative of structural deterioration.

Since bridge inspections focus on individual members and system identification considers the entire structure, they are complementary processes. Therefore, system identification can be used to define the structural integrity of the entire bridge structure.

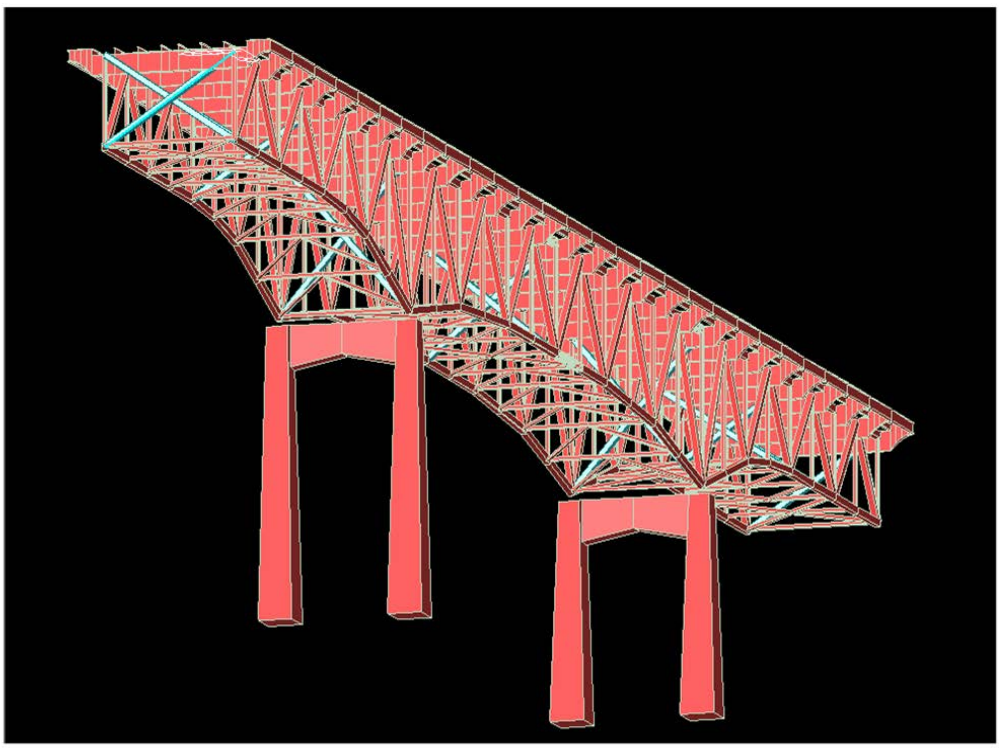

Figure 15.4.5 Structural Model

Practical Considerations for Selection and Use of Advanced Bridge Evaluation Technologies

Any advanced bridge evaluation technology typically provides a reasonable return on investment. There is no reason to pay for technology unless that technology can provide a sufficient return for the Bridge Owner. Over the past several years, significant research has been conducted to demonstrate new advanced bridge evaluation technologies. Some worked well; others did not. But given the current advanced bridge evaluation technologies, owners can typically expect adequate returns on projects. Returns can be calculated using a variety of financial metrics, but some of the more useful are provided below:

- Safe extension of bridge life span, lowering life cycle cost of ownership
- Safe deferral of bridge maintenance or repair programs
- Safe deferral of bridge replacement programs
- Improved prioritization of limited funds
- Removal of unnecessary load restrictions to support commercial traffic and reduce detours, congestion and air pollution
- Identification of bridges that are to be replaced or repaired immediately, thereby lowering liability exposure and increasing safety

Other issues to consider before utilizing an advanced bridge technology:

- Is the advanced bridge evaluation technology being used for a few bridges or across the entire system?
- Is the advanced bridge evaluation technology capturing the “right” information to aid decision making and not a lot of extraneous information?
- Can the advanced bridge evaluation solution be expanded easily and cost effectively if it is later decided to capture more data?
- Should a solution provider be used, capable of system configuration and installation, or integrate the hardware and software internally?
- Should the captured information be able to integrate with the existing information system?
- How long is the technology expected to be deployed – what is the reliability and durability of the hardware and software?
- Can the confidentiality of captured data, both on-site and for later viewing and downloading, be assured?
- Who has the responsibility for conversion of the structural data into useful information and subsequent analysis of that information?
- Can the hardware be used on other structures after project completion?

In summary, the use of advanced bridge evaluation technologies can provide owners with information that promotes “fact-based” decisions. Care and judgment are utilized when specifying and purchasing improved technologies, as well as use in the field. To obtain the best return on investment, defer to those with experience and earned reputation to provide alternative solutions for consideration

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Complex Bridges

Topic 16.1 Cable-Supported Bridges

16.1.1 Introduction

There are several bridge types which feature elements which require special inspection procedures. The most notable bridge types are:

- Suspension bridges (see Figure 16.1.1)
- Cable-stayed bridges (see Figure 16.1.2)

Figure 16.1.1 Golden Gate Bridge

This topic is limited to the cable and its elements. All other members of a cable-supported bridge have been described in earlier topics and are to be referred to for the appropriate information. For each of the above bridge types, this topic provides:

- A general description
- Identification of special elements
- An inspection procedure for special elements
- Methods of recordkeeping and documentation

Figure 16.1.2 Maysville Cable-Stay Bridge

16.1.2 Design Characteristics

A cable-supported bridge is a bridge that is supported by or "suspended from" cables.

Suspension Bridges

A suspension bridge has a deck, which is supported by vertical suspender cables that are in turn supported by main suspension cables. The suspension cables can be supported by saddles atop towers and are anchored at their ends or self-anchored to the bridge superstructure. Suspension bridges are normally constructed when intermediate piers are not feasible because of long span requirements (see Figure 16.1.3). Modern suspension bridge spans are generally longer than 1400 feet.

Cable-Stayed Bridges

A cable-stayed bridge is another long span cable-supported bridge where the superstructure is supported by cables, or stays, passing over or anchored to towers located at the main piers. Cable-stayed bridges are the more modern version of cable-supported bridges. Spans generally range from 700 to 1400 feet (see Figure 16.1.4). Evolving for approximately 400 years, the first vehicular cable-stayed bridge in the United States was constructed in Alaska in 1972 (John O'Connell Memorial Bridge at Sitka, Alaska).

In suspension bridges, vertical suspender cables attach the deck and floor system to the main suspension cables. Cable-stayed bridges are much stiffer than suspension bridges. In cable-stayed bridges, the deck and floor system is supported directly from the tower with fairly taut stay cables.

Figure 16.1.3 Roebling Bridge

Figure 16.1.4 Sunshine Skyway Cable-Stayed Bridge in Tampa Bay, Florida

Types of Cables

A cable may be composed of one or more structural wire ropes, structural wire strands, locked coil strands, parallel wire strands, or parallel wires.

Parallel Wire Cable

Parallel wire cable consists of a number of parallel wires (see Figure 16.1.5 and Figure 16.1.10). The diameter varies depending on the span length and design loads. Parallel wire cables used in cable-stayed bridges conforms to ASTM A421, Type BA, low relaxation. It is basically stress-relieved wire used for prestressed concrete.

Structural Wire Strand

Structural wire strand is an assembly of wires formed helically around a center wire in one or more symmetrical layers. Sizes normally range from 2 to 4 inches in total diameter (see Figure 16.1.6).

Structural Wire Rope

Structural wire rope is an assembly of strands formed helically around a center strand (see Figure 16.1.7).

Parallel Strand Cable

Parallel strand cable is a parallel group of strands (see Figure 16.1.8). Seven-wire strand commonly used for cable-stayed bridges conforms to ASTM A416, low relaxation steel (see Figure 16.1.11). It is basically seven-wire stress-relieved strand for prestressed concrete.

Locked Coil Strand

Locked coil strand is a helical type strand composed of a number of round wires, and then several layers of wedge or keystone shaped wires and finally several layers of Z- or S-shaped wires (see Figure 16.1.9). Locked coil strand has not been used for cable-stayed bridges in this country, but it is commonly used for cable-stayed bridges in Europe.

Several types of cables have been used for cable-stayed bridges. The three most common are locked-coil strand, parallel wire, and parallel seven-wire strand. The majority of existing cable-stayed bridges in the world, other than the United States, use preformed prestretched galvanized locked-coil strand. The cable-stayed bridges in the United States incorporate parallel wire or seven-wire prestressing strand in the cables.

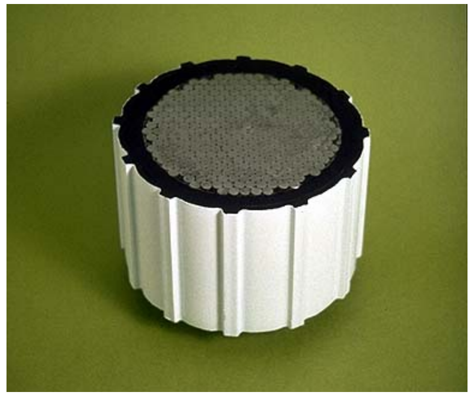


Figure 16.1.5 Parallel Wire

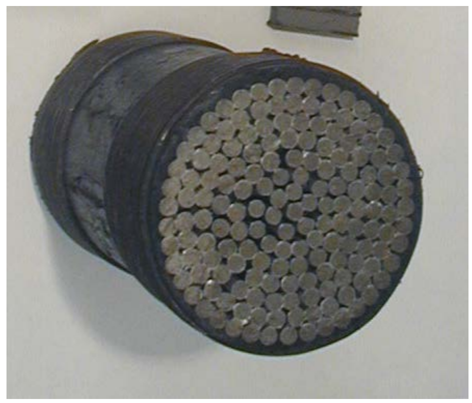


Figure 16.1.6 Structural Wire Strand

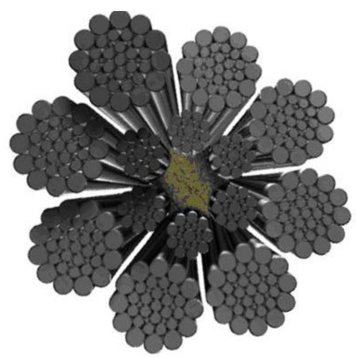


Figure 16.1.7 Structural Wire Rope

Figure 16.1.8 Parallel Strand Cable

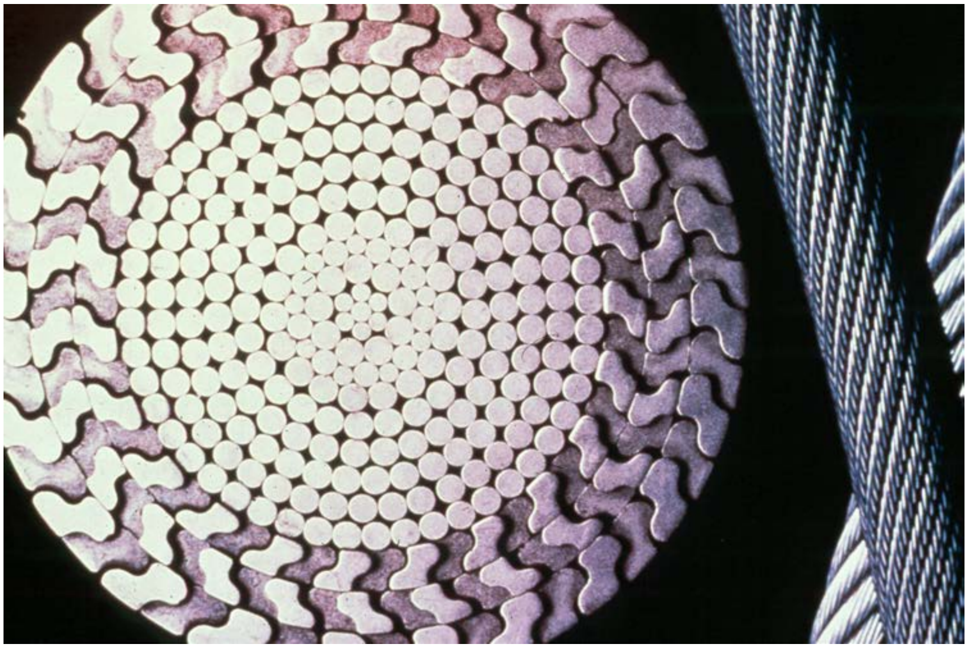


Figure 16.1.9 Locked Coil Strand Cross-Section

Figure 16.1.10 Parallel Wire

Figure 16.1.11 Parallel Strand

Corrosion Protection of Cables

Methods used for corrosion protection include:

- Galvanizing the individual wires
- Painting the finished cable
- Wrapping the finished cable with spirally wound soft galvanized wire, neoprene, or plastic wrap type tape
- Polyethylene sheathing filled with cement grout or grease
- Polyethylene sheathing filled with no grouting
- Any combination of the above systems (see Figure 16.1.12).

Figure 16.1.12 Cable Wrapping on the Wheeling Suspension Bridge

Types of Towers

- Portal tower – typical of suspension bridges (see Figure 16.1.13 (a))
- Towers fixed to pier (see Figure 16.1.13 (b))
- Towers fixed to superstructure (see Figure 16.1.13 (c))
- Single column tower (see Figure 16.1.13 (d))
- A-frame tower (see Figure 16.1.13 (e))
- Laterally offset tower fixed to pier (see Figure 16.1.13 (f))
- Diamond shaped tower (see Figure 16.1.13 (g))

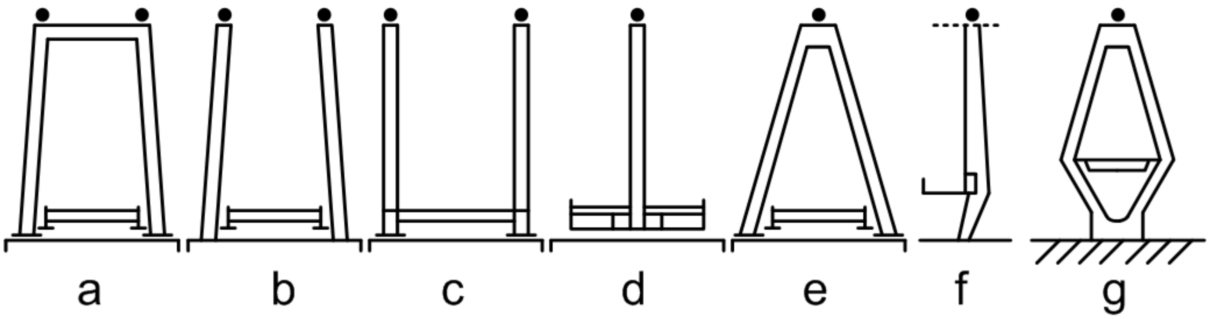


Figure 16.1.13 Shapes of Towers Used for Cable-Stay Bridges

Towers are constructed of reinforced concrete or steel or a combination of the two materials (see Figures 16.1.14 and 16.1.15).

Figure 16.1.14 Tower Types: Concrete "Portal Tower" and "A-Frame Tower"

Figure 16.1.15 Tower Types: Steel "Portal Tower" and Concrete "Single Column Tower"

The deck structures are also constructed of concrete or steel.

16.1.3 Suspension Bridges

In this subtopic, only those bridge elements that are unique to suspension bridges are presented. Refer to the appropriate topic for other bridge elements that are common to similar bridge types (i.e. floor systems, open web girders, box sections, etc.).

Main Suspension Cables and Suspender Cables

Main suspension cables are generally supported on saddles at the towers and are anchored at each end. Sometimes, main suspension cables are referred to as catenary cables. Suspender cables are vertical cables that connect the deck and floor system to the main suspension cables (see Figure 16.1.16). .

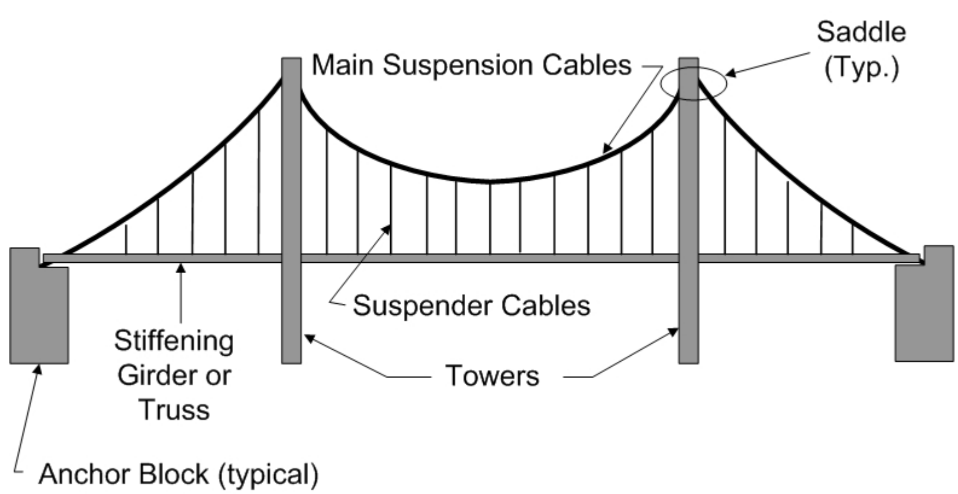


Figure 16.1.16 Three-Span Suspension Bridge Schematic

If a suspension bridge has only two main suspension cables, the cables are considered to be fracture critical members since there is no load path redundancy. Refer to Topic 6.4 for a detailed description of fracture criticality and redundancy.

Another type of suspension bridge uses a self-anchoring system where the main suspension cables are anchored into the edge girders that span continuously from end to end in the suspension spans. The force from the main suspension cables puts the edge girders into compression. The edge girders support the floor system and the suspender cables support the edge girders in this arrangement.

This type suspension bridge may be used to create long clear spans for navigation and not have to continue the suspension spans to the shorelines for anchorage. The alignment for the approach spans can be different than the suspension spans. These benefits are seen in the new Oakland Bay View Bridge in California, where the approach alignment is on a curve, the suspension span creates the wide navigation channel and the anchoring is self contained within the superstructure of the suspension spans.

Anchorage and Connections

Anchorages

In bridges with common earth anchored cable systems, either above or below ground, the total force of the main suspension cable has to be transferred into the anchor block (see Figure 16.1.17). The void area inside the anchor block is referred to as the Chain Gallery. The force from the main cable is distributed through the splay saddle, bridge wires, strand shoes and anchor bars. The anchor bars are embedded and secured in the concrete of the anchor block. The anchor bars may consist of steel bars, rods, pipes, or prestressed bars / strands.

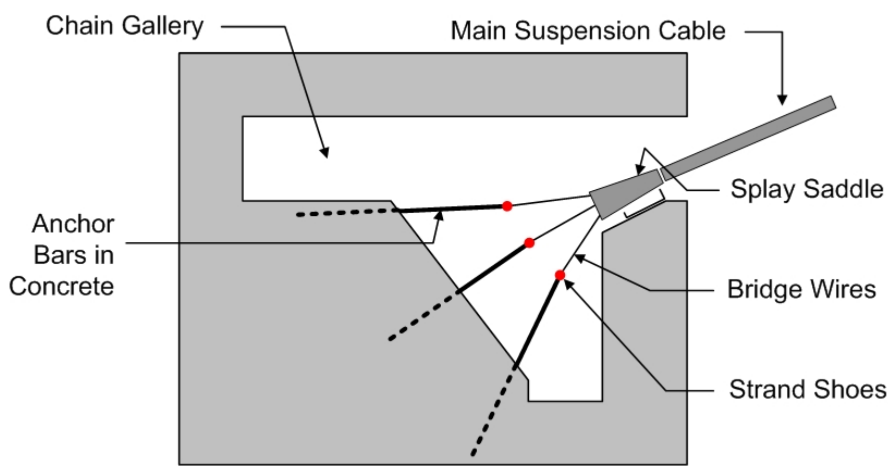


Figure 16.1.17 Anchor Block Schematic

Saddles

The connection between main cable and tower is usually made through saddles. The saddle supports the main cable as it crosses over the tower (see Figure 16.1.18). Saddles are commonly made from fabricated steel or castings.

Figure 16.1.18 Cable Saddles for Manhattan Bridge, NYC (Main Span 1,480 ft)

Suspender Cable Connections

The connection between the main suspension cable and suspender cable is made by means of a cable band. The cable band consists of two semi-cylindrical halves connected by high-tensile steel bolts to develop the necessary friction.

Grooved cable bands have been used in the majority of suspension bridges (see Figure 16.1.19). The top surfaces of the bands are grooved to receive the suspender cables, which are looped over the band.

Instead of looping the hanger cables around the main suspension cable, the hanger might be socketed at the upper end and pin connected to the cable band. This connection is called an open socket (see Figure 16.1.20). Connection to the deck and floor system can also be a similar open socket arrangement or it can be connected directly to a floorbeam - similar to the tied arch bridge.

Figure 16.1.19 Grooved Cable Bands

Figure 16.1.20 Open Socket Suspender Cable Connection

Vibrations

The flexibility of cable-supported structures, associated with high stress levels in the main load carrying members, makes these structures especially sensitive to dynamic forces caused by earthquake, wind, or vehicular loads. The term local vibration is used when dealing with the vibration in an individual member (see Figure 16.1.21). When the vibration of the entire structure as a whole is analyzed, it is known as global vibration (see Figure 16.1.22). Due to the amount of vibration in cable-supported structures, it may be common to see various types of damping systems attached to cables. Damping systems may be a tie between two cables, neoprene cushions, shock absorbers mounted directly to the cables, or other systems that act to dampen the cable vibrations (see figures 16.1.23 and 16.1.24).

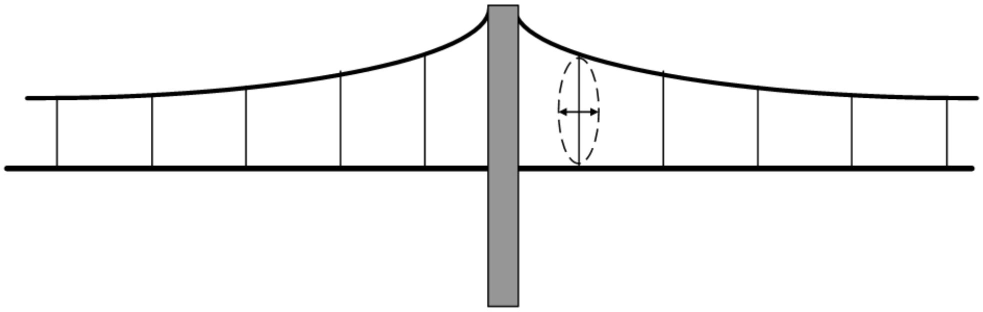


Figure 16.1.21 Cable Vibrations Local System Schematic

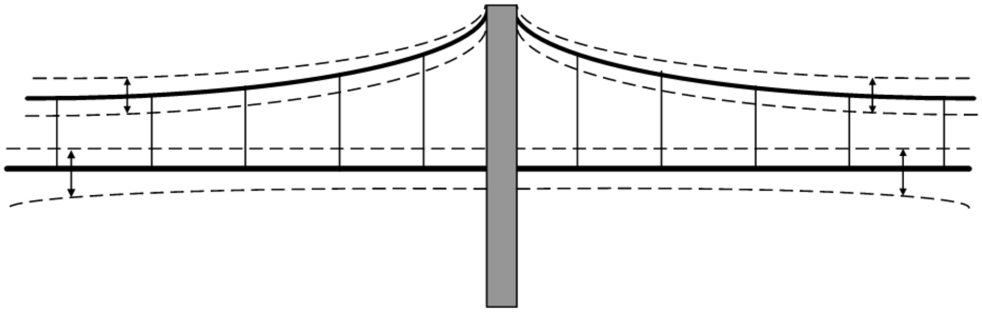


Figure 16.1.22 Cable Vibrations Global System Schematic

Vibrations can affect suspension cables in several ways. Vibration opens cable wires allowing entry of corrosive chemicals and accelerates corrosion. Vibrations create fretting, cracks in the protective coating and cement grout, and accelerate corrosion and possibly fatigue.

Figure 16.1.23 Cable Damping System - Wheeling Suspension Bridge
(Photo Courtesy of Geoffrey H. Goldberg, 1999)

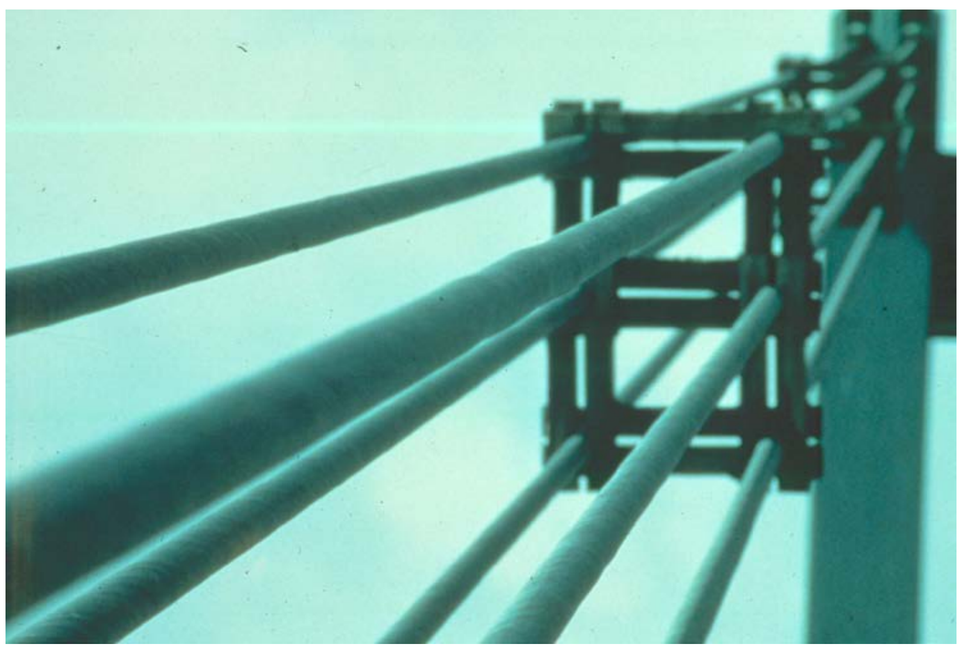


Figure 16.1.24 Cable Tie Damper System

16.1.4 Cable-Stayed Bridges

Only the cable and its elements are described in this subtopic. Refer to the appropriate topic for other bridge elements that are common to similar bridge types (i.e. floor systems, open web girders, box sections, etc.).

Due to the complexity of the various cable arrangements and systems, fracture criticality for individual cable-stayed structures can only be determined through a detailed structural analysis.

Cable Arrangements and Systems

Cable-stayed bridges may be categorized according to the various longitudinal cable arrangements. These cable arrangements are categorized into the following four basic systems:

- Radial or Converging Cable System
- Harp Cable System
- Fan Cable System
- Star Cable System

Radial or Converging Cable System

In this system, all cables are leading to the top of the tower at a common point. Structurally, this arrangement is the most effective. By anchoring all the cables to the tower top, the maximum inclination to the horizontal is achieved (see Figure 16.1.25).

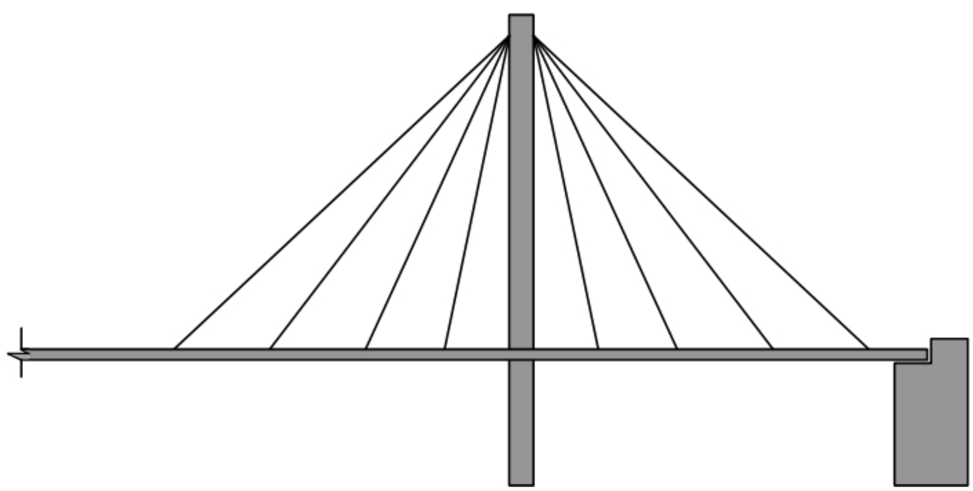


Figure 16.1.25 Radial or Converging Cable System Schematic

Harp Cable System

The harp system, as the name implies, resembles harp strings. In this system, the cables are parallel and equidistant from each other. The cables are also spaced uniformly along the tower height and connect to the deck floor system or superstructure at the same spacing (see Figure 16.1.26).

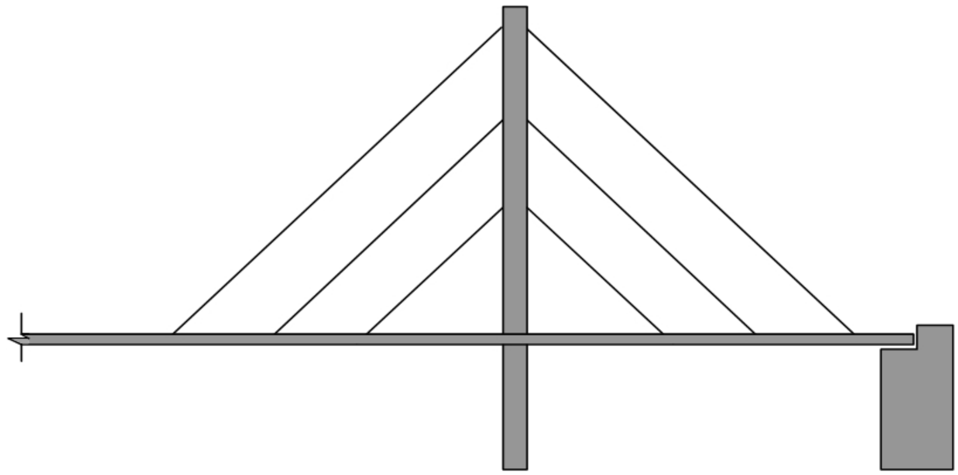


Figure 16.1.26 Harp or Parallel Cable System Schematic

Fan Cable System

The fan system is a combination of the radial and the harp systems. The cables emanate from the top of the tower at equal spaces and connect to the superstructure at larger equal spaces (see Figure 16.1.27).

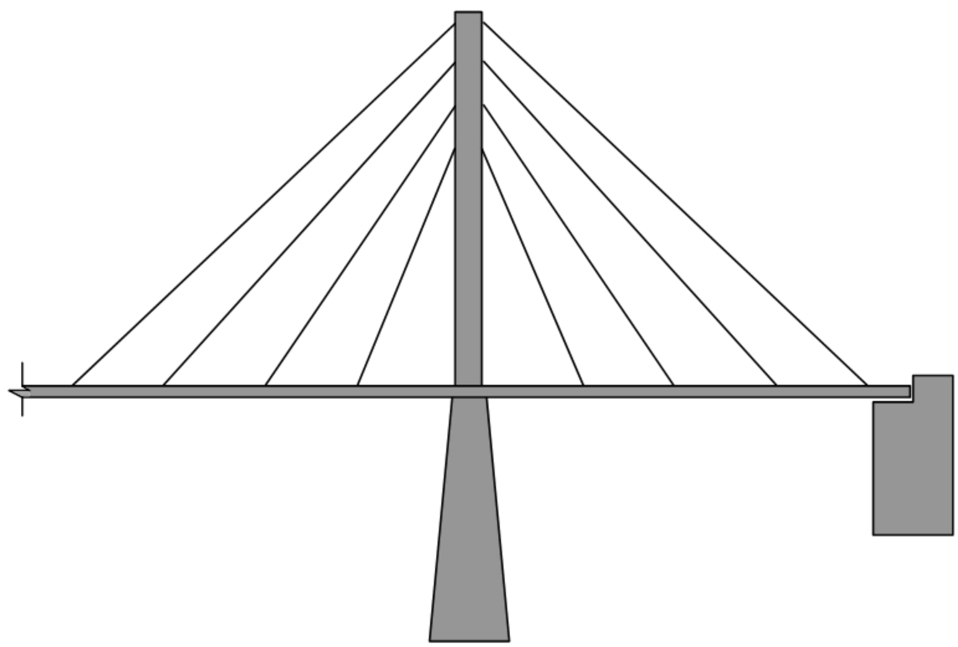


Figure 16.1.27 Fan or Intermediate Cable System Schematic

Star Cable System

In the star system, the cables intersect the tower at different heights and then converge on each side of the tower to intersect the deck structure at a common point. The common intersection in the anchor span is usually located over the abutment or end pier. The star system is uncommon compared to the three systems previously presented. The star system requires a much stiffer superstructure since the cables are not distributed longitudinally along the deck and superstructure (see Figure 16.1.28).

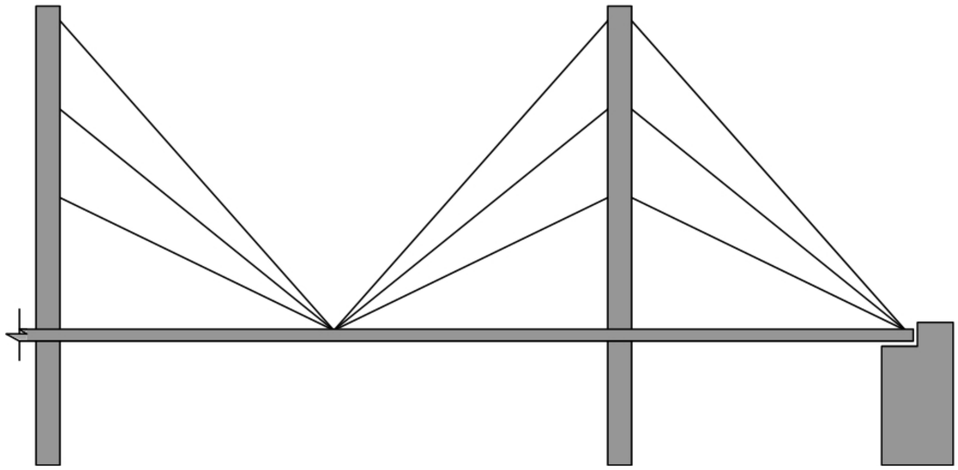


Figure 16.1.28 Star Cable System Schematic

Cable Planes

The cables may lie in either a single or a double plane, may be symmetrical or asymmetrical, and may lie in oblique or vertical planes.

Single Plane

The single-plane cable arrangement is used with a divided deck structure with the cables passing through the median area and anchored below the deck. A single-plane cable system generally utilizes single column or A-frame towers (see Figure 16.1.29).

Figure 16.1.29 Single Vertical Plane Cable System

Double Vertical Plane

The double vertical plane system incorporates two vertical cable planes connecting the tower to the edge girders along the deck structure. The structure may utilize twin towers or a portal frame tower (see Figure 16.1.30). The portal frame tower is a twin tower with a connecting strut at the top. Wider bridges may utilize a triple plane system that is basically a combination of the single and double plane systems.

Figure 16.1.30 Double Vertical Plane Cable System

Double Inclined Plane

In this two plane system the cable planes are oblique, sloping toward each other from the edges of the deck and intersecting at the tower along the longitudinal centerline of the deck (see Figure 16.1.31). Generally the tower is an A-frame type, receiving the sloping cables that intersect close to the centerline on the tower.

Figure 16.1.31 Double Inclined Plane Cable System

Anchorages and Connections

The cables may be continuous and pass through or over the tower or be terminated at the tower. If continuous across the tower, a saddle is incorporated.

Saddles

The cable saddles are constructed from fabricated plates or steel castings with grooves through which the cables pass (see Figure 16.1.32). Between the end and center spans differential forces will occur at the cable saddles unless they are supported by rollers or rocker bearings. When the saddles are fixed, the rigidity of the system is at the maximum.

Figure 16.1.32 Cable Saddle

End Fittings

If terminated at the tower, an end fitting or anchorage is incorporated. A similar end fitting is utilized at the edge girder (see Figure 16.1.33).

Figure 16.1.33 Cable Deck Anchorage

Socket

A socket widely used for the anchoring of parallel-wire strands is a poured zinc socket. The wires are led through holes in a locking plate at the end of the socket and have the bottom heads providing the resistance against slippage of wires. The cavity inside the socket is filled with hot zinc alloys. To improve the fatigue resistance of the anchor, a cold casing material is used. The zinc cools and locks the wire strands into the socket.

Figure 16.1.34 Anchor Inspection on Veterans Bridge

The problems encountered with low fatigue strength of zinc-poured sockets lead to the development of HiAm sockets in 1968 for use with parallel wire stays.

This anchorage incorporates a flat plate with countersunk radial holes to accommodate the geometry of flared wires that transition from the compact wire bundle into the anchorage. The anchorage socket is filled with zinc dust and with an epoxy binder. This method of anchoring the stays increases the magnitude of fatigue resistance to almost twice that for the zinc-poured sockets.

A common anchorage type for strands is the Freyssinet type anchor.

In the Freyssinet socket the seven wire strand is anchored to an anchor plate using wedges similar to prestressing wedges. This wedge anchor is used during erection. After application of the permanent dead load, the anchor tube is filled with an epoxy resin, zinc dust, and steel ball composition. Under transient live load, the additional cable force will be transformed by shear from the cable strand to the tube.

Vibrations

Several of the primary causes of vibration in stay cables consist of rain-wind induced vibrations, sympathetic vibration of cables with other bridges elements excited by wind, inclined cable galloping, and vortex excitation of single cable or groups of cables. Due to the amount of vibration in cable-supported structures, it may be common to see various types of damping systems attached to cables. Damping systems may be a tie between two cables, neoprene cushions, shock absorbers mounted directly to the cables, or other systems that act to dampen the cable vibrations (see Figure 16.1.35).

Vibrations can affect stay cables in several ways. Vibration opens cable wires allowing entry of corrosive chemicals and accelerates corrosion. Vibrations create fretting, cracks in the protective coating and cement grout, and accelerate corrosion and possibly fatigue.

Figure 16.1.35 Damper on Cable-Stayed Bridge

16.1.5 Overview of Common Deficiencies

Common deficiencies that can occur on steel cable members:

- Corrosion
- Fatigue Cracking
- Overloads
- Collision Damage
- Heat Damage
- Paint Failure

Refer to Topic 6.3.5 for a more detailed presentation of the properties of steel, types and causes of steel deficiencies, and the examination of steel.

16.1.6 Inspection Locations and Methods for Suspension Bridge Cable System Elements

The inspection and maintenance methods presented in this Topic are not exhaustive, but are unique to the particular bridge type. Therefore, include both the procedures presented in this Topic as well as the general procedures previously presented in this manual during the inspection of special bridges.

These bridges are considered to be complex according to the NBIS regulations. The NBIS requires identification of specialized inspection procedures, and additional inspector training and experience required to inspect these complex bridges. The bridges are then to be inspected according to these procedures.

Due to the specialized nature of these bridges and because no two cable-supported bridges are identical, the inspection should be led by someone very familiar with the particular bridge. Many major bridges, such as cable-supported bridges, will have individual inspection and maintenance manuals developed specifically for that bridge, like an "owner's" manual. If available, use this valuable tool throughout the inspection process and verify that specified routine maintenance has been performed. Use customized, preprinted inspection forms wherever possible to enable the inspector to report the findings in a rigorous and systematic manner.

Main Cable Anchorage Elements

The anchorage system, at the ends of the main cables, consists of a number of elements that require inspection (see Figure 16.1.36).

- Splay saddle
- Bridge Wires
- Strand shoes or sockets
- Anchor bars
- Chain Gallery

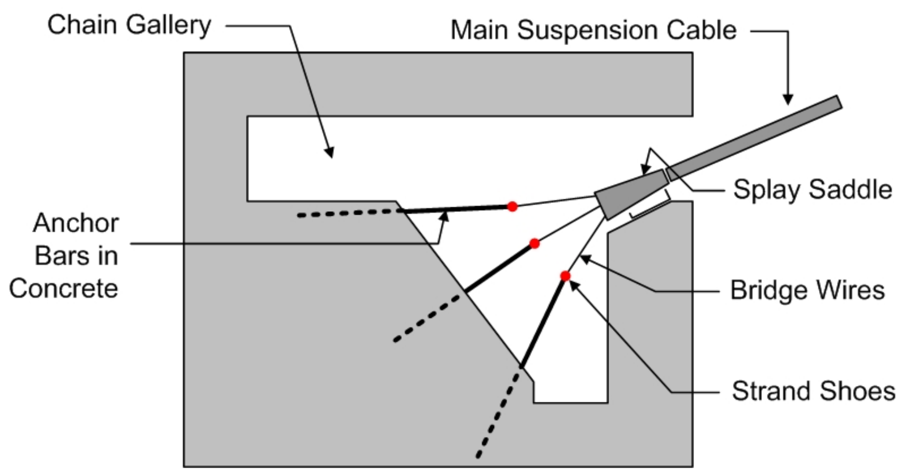


Figure 16.1.36 Anchor Block Schematic

Splay Saddle

Inspect the splay saddles for missing or loose bolts and the presence of cracks in the casting itself. There is a possibility of movement up the cable away from the splay. Signs of this movement may be the appearance of unpainted strands on the lower side or “bunched up” wrapping on the upper side

Bridge Wires

In parallel wire type suspension bridges, inspect the unwrapped wires between the strand shoes and the splay saddle. Carefully insert a large screwdriver between the wires and apply leverage. This will help reveal broken wires. Inspect the wires for abrasion damage, corrosion, and movement.

Strand Shoes or Sockets

At the anchorages of parallel wire type suspension bridges, inspect the strand shoes for signs of displaced shims, along with movement, corrosion, misalignment, and cracks in the shoes.

At the anchorages of prefabricated strand type suspension bridges, inspect the strand sockets for signs of movement, slack or sag, corrosion, and broken sockets. Unpainted or rusty threads at the face of the sockets may indicate possible “backing off” of the nuts.

Anchor Bars

Inspect the anchor bars or rods for corrosion (section loss), deficiencies, or movement at the face of their concrete embedment. Check for corrosion or other signs of distress over the entire visible (unencased) portion.

Chain Gallery

Inspect the interior of the anchorage for corrosion and deficiencies of any steel hardware, and cracks and spalls in the concrete anchor. Note if there is protection against water entering or collecting where it may cause corrosion, and also if there is proper ventilation (see Figure 16.1.37).

Figure 16.1.37 Anchorage Interior of Ben Franklin Bridge, Philadelphia, PA

Main Suspension Cables

Inspect the main suspension cables as follows:

Cables

Inspect the main suspension cables for indications of corroded wires. Inspect the condition of the protective covering or coating, especially at low points of cables, areas adjacent to the cable bands, saddles over towers, and at anchorages.

Cable Wrapping

Inspect the wrapping wire for cracks, staining, and dark spots. Check for loose wrapping wires. If there are cracks in the caulking where water can enter, this can cause corrosion of the main suspension cable. Check for evidence of water seepage at the cable bands, saddles, and splay castings (see Figure 16.1.38).

Figure 16.1.38 Tape and Rubber Seal Torn Around Cable Allowing Water Penetration into Top of Sheath

Hand Ropes

Inspect the hand ropes and connections along the main cables for loose connections of stanchion (hand rope supports) to cable bands or loose connections at anchorages or towers. Check also for corroded or deteriorated ropes or stanchions, bent or twisted stanchions, and too much slack in rope.

Vibration

Note and record all excessive vibrations.

Saddles

Inspect the saddles for missing or loose bolts, and corrosion or cracks in the casting. Check for proper connection to top of tower or supporting member and possible slippage of the main cable.

Suspender Cables and Connections

Inspect the suspender cables for corrosion or deficiencies, broken wires, and kinks or slack. Check for abrasion or wear at sockets, saddles, clamps, and spreaders. Be sure to note excessive vibrations.

Sockets

Inspect the suspender rope sockets for:

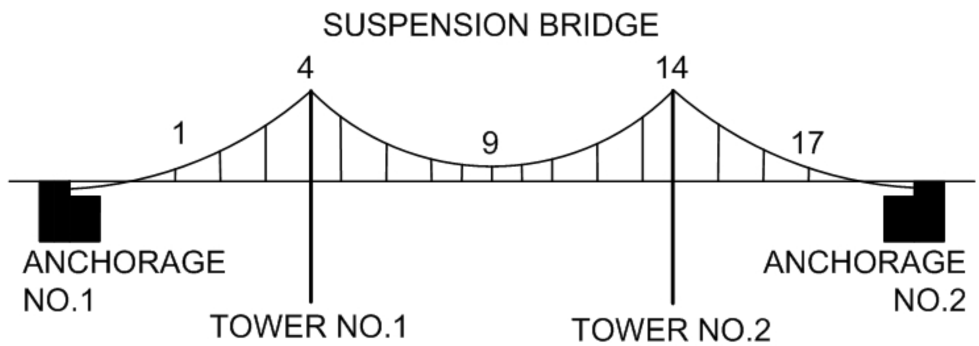
- Corrosion, cracks, or deficiencies
- Abrasion at connection to bridge superstructure
- Possible unanticipated movement

Cable Bands

Inspect the cable bands for missing or loose bolts, or broken suspender saddles. Signs of possible slippage are caulking that has pulled away from the casting or "bunching up" of the soft wire wrapping adjacent to the band. Check for the presence of cracks in the band itself, corrosion or deficiencies of the band, and loose wrapping wires at the band.

Recordkeeping and Documentation

Prepare a set of customized, preprinted forms for documenting all deficiencies encountered in the cable system of a suspension bridge. A suggested sample form is presented in Figure 16.1.39. Separate forms are to be used for each main suspension cable. Designations used to identify the suspender ropes and the panels provide a methodology for locating the problems in the structure. Note and describe vibrations whether local or global, while performing inspections of cable-supported structures.



SUSPENDER CABLE CONDITION

1.	7.	13.
2.	8.	14.
3.	9.	15.
4.	10.	16.
5.	11.	17.
6.	12.	

Figure 16.1.39 Form for Recording Deficiencies in the Cable System of a Suspension Bridge

16.1.7 Inspection Locations and Methods for Cable-Stayed Bridge Cable System Elements

A cable-stayed bridge is a bridge in which the superstructure is supported by cables, or stays, passing over or attached directly to towers located at the main piers (see Figure 16.1.40 and 16.1.41). There are several special elements that are unique to cable-stayed bridges.

See the National Cooperative Highway Research Program (NCHRP) Synthesis 353 "Inspection and Maintenance of Bridge Cable Systems", 2005 for a detailed description of inspection locations and procedures for cable-stayed bridge cable element systems.

These bridges are considered to be complex according to the NBIS regulations. The NBIS requires identification of specialized inspection procedures, and additional inspector training and experience required to inspect these complex bridges. The bridges are then to be inspected according to these procedures.

Figure 16.1.40 Cable-Stayed Bridge

Figure 16.1.41 Cable-Stayed Bridge Cables

Inspection Elements

Cable element inspection includes:

- Cable wrappings and wrap ends near the tower and deck
- Cable sheathing assembly
- Dampers
- Anchorages

Cable Wrapping

Common wrapping methods for corrosion protection of finished cables include spirally wound soft galvanized wire, neoprene, or plastic wrap type tape (see Figure 16.1.42). Inspect the wrappings for corrosion and cracking of soft galvanized wire, staining and dark spots indicating possible corrosion of the cables, and loose wrapping wires or tape. Bulging or deforming of wrapping material may indicate possible corrosion or broken wires (see Figure 16.1.43). Check for evidence of water seepage at the cable bands, saddles, and castings.

Figure 16.1.42 Cable Wrapping Placement

Figure 16.1.43 Deformed Cable Wrapping

Cable Sheathing Assembly

The most common types of cable sheathing assemblies are steel sheathing and polyethylene sheathing.

Steel Sheathing

If steel sheathing is used, inspect the system for corrosion (see Figure 16.1.44), condition of protective coatings, and weld fusion. Bulging may indicate corrosion or broken wires (see Figure 16.1.45). Splitting may be caused by water infiltration and corrosive action. Cracking is sometimes caused by fatigue (see Figure 16.1.46).

Polyethylene Sheathing

If polyethylene sheathing is used, inspect the system for nicks, cuts, and abrasions. Check for cracks and separations in caulking and in fusion welds. Bulging may indicate broken wires (see Figure 16.1.45). Splitting is sometimes caused by temperature fluctuations (see Figure 16.1.47). Coefficient of the thermal expansion for polyethylene is three times higher than the value for steel or concrete. Cracking is sometimes caused by fatigue.

Figure 16.1.44 Corrosion of Steel Sheathing

Figure 16.1.45 Bulging of Cable Sheathing

Figure 16.1.46 Cracking of Cable Sheathing

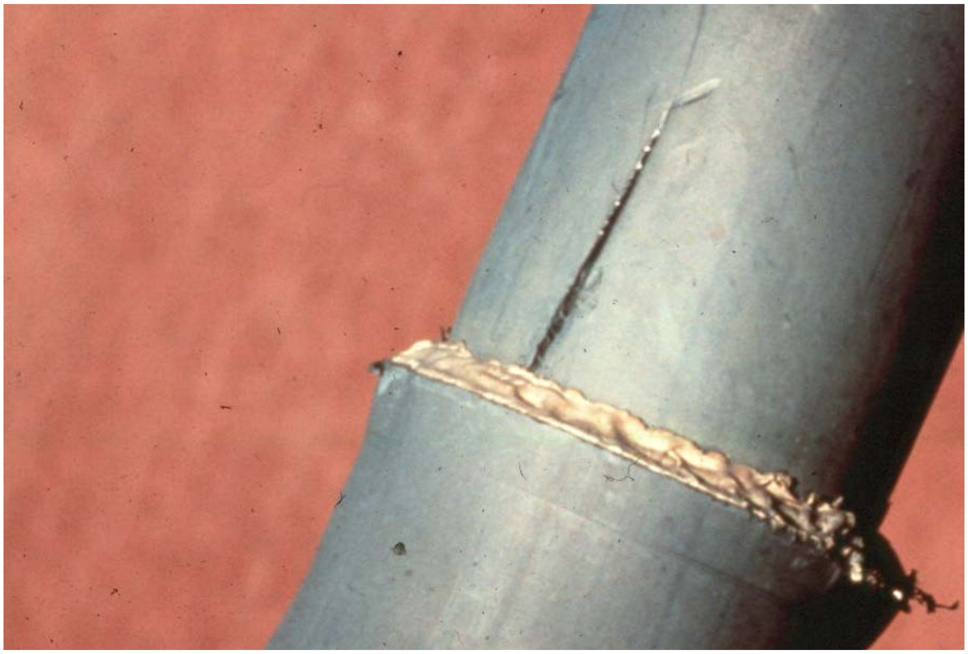


Figure 16.1.47 Splitting of Cable Sheathing

Dampers

Shock Absorber Type

A variety of damper types may have been installed (see Figure 16.1.48 and 16.1.49). If shock absorber type dampers are used, inspect the system for corrosion, oil leakage in the shock absorbers, and deformations in the bushings. Check for tightness in the connection to the cable pipe, and torque in the bolts.

Figure 16.1.48 Shock Absorber Damper System

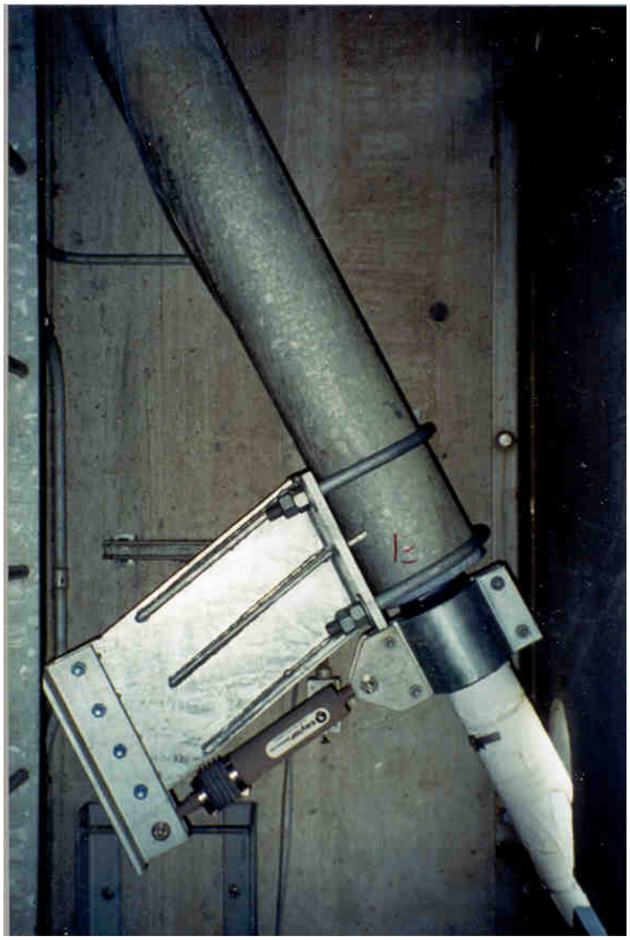


Figure 16.1.49 Shock Absorber Damper System

Tie Type

Inspect the tie type dampers (see Figure 16.1.50) for corrosion, and deformations in the bushings. Check for tightness in the connection to the cable pipe, and torque in the bolts.

Figure 16.1.50 Cable Tie Type Damper System

Tuned Mass Type

Inspect the tuned mass dampers (see Figure 16.1.51) for corrosion, and deformations in the bushings. Check for tightness in the connection to the cable pipe, and torque in the bolts.

Figure 16.1.51 Tuned Mass Damper System

Anchorages

End Anchorage

Inspect the transition area between the steel anchor pipe and cable for water tightness of neoprene boots at the upper ends of the steel guide pipes (see Figure 16.1.52). Check for drainage between the guide pipe and transition pipe, and deteriorations, such as splits and tears, in the neoprene boots (see Figure 16.1.53). Check for sufficient clearance between the anchor pipe and cable, noting rub marks and kinks.

Figure 16.1.52 Neoprene Boot at Steel Anchor Pipe Near Anchor

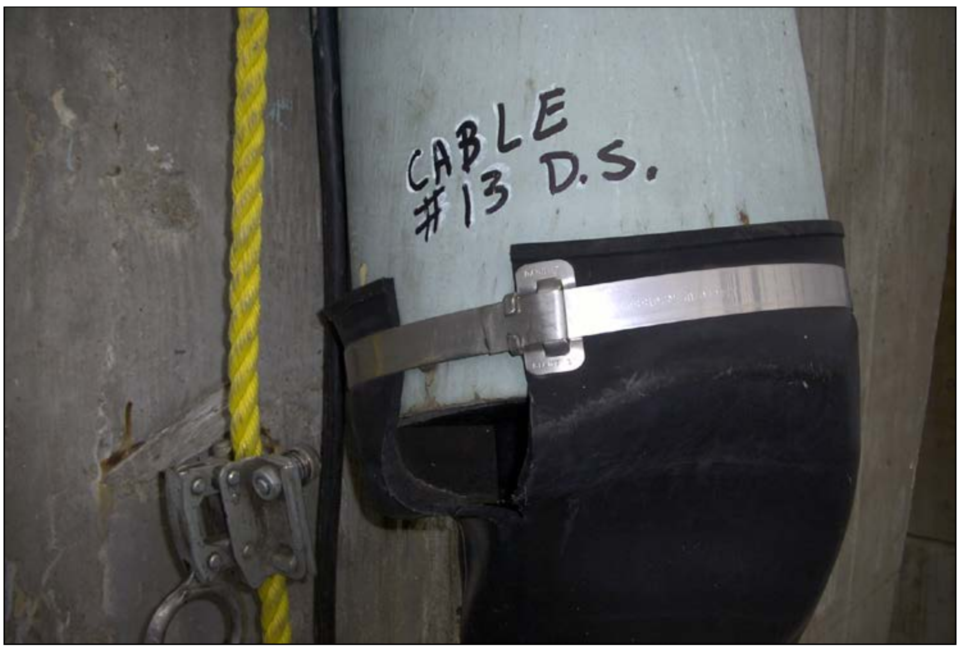


Figure 16.1.53 Split Neoprene Boot

Tower Anchorage

Inspect the cable anchorages for corrosion of the anchor system (see Figure 16.1.54). Check for cracks and nut rotation at the socket and bearing plate, and seepage of grease from the protective hood.

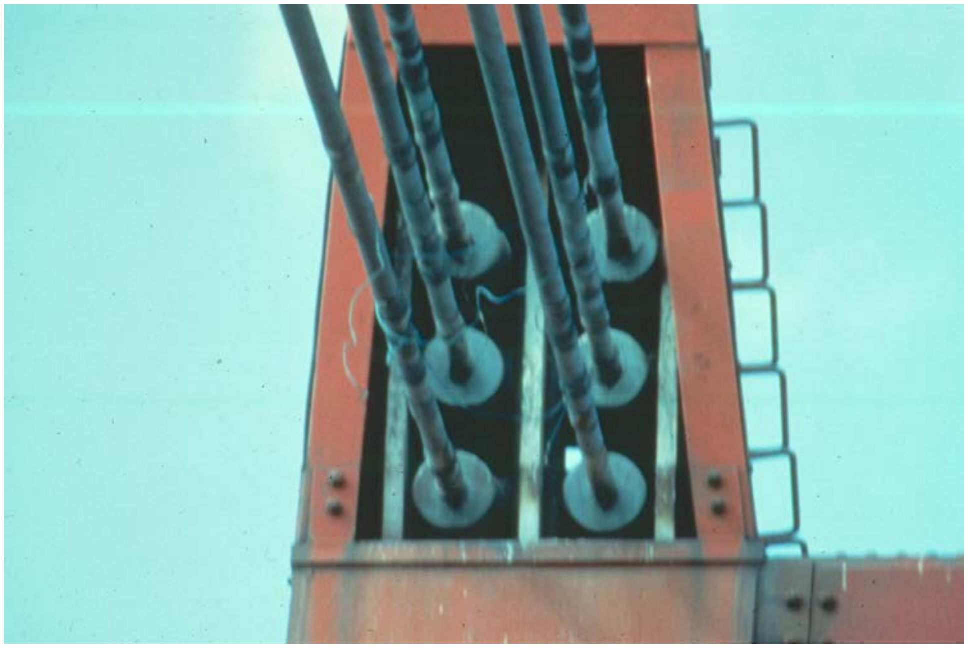


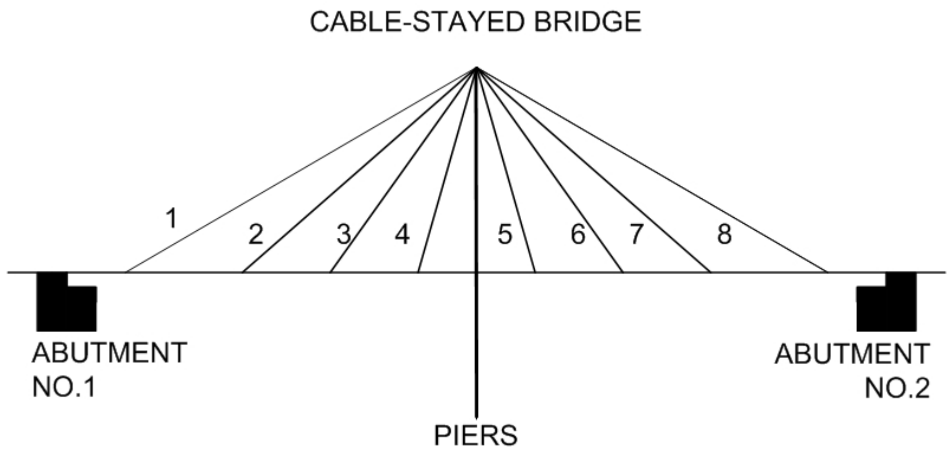
Figure 16.1.54 Corrosion of the Anchor System

Other Inspection Items

Include anchor pipe clearances, flange joints, and polyethylene expansion joints within the inspection of the cable system. Read the load cells and record the forces in the cables. Note and record all excessive vibrations including amplitude and type of vibration along with wind speed and direction, or other forces including vibrations such as traffic. Also evaluate cable and tower lighting systems.

Recordkeeping and Documentation

Prepare a set of customized, preprinted forms for documenting all deficiencies encountered in the cable system of a cable-stayed bridge. A suggested sample form is presented in Figure 16.1.55. Use a separate form for each plane or set of cables. Designations used to identify the cables and the panels provide a methodology for locating the deficiencies in the structure. Note and describe vibrations whether local or global, while performing inspections of cable-supported structures.



CABLE CONDITION

1.	5.
2.	6.
3.	7.
4.	8.

Figure 16.1.55 Form for Recording Deficiencies in Cable System of a Cable-Stayed Bridge

16.1.8 Advanced Inspection Methods

In bridge cables, whether a suspension bridge or cable-stayed bridge, the greatest problems generally occur due to corrosion and fracture of individual wires. Visual inspection of unwrapped cables is limited to the outer wires, while visual inspection of wrapped cables is limited to the protective sheathing. Therefore, advanced inspection methods are used to achieve a more rigorous and thorough inspection of steel bridges, including:

- Acoustic Emissions Testing
- Corrosion Sensors
- Smart Coatings
- Dye Penetrant
- Magnetic Particle
- Radiography Testing
- Computed Tomography
- Robotic Inspection

- Ultrasonic Testing
- Eddy Current
- Electrical fatigue sensor (EFS)
- Magnetic flux leakage
- Laser vibrometer

See Topic 15.3 for Advanced Inspection Procedures for steel.

Other methods specific to cables include:

- Cable force measurements using the precursor transformation matrix - this method uses a linearly elastic finite-element analysis (FEA) model of the cable-supported bridge. Through the model, the temperature each of cable is raised to simulate loss of stiffness one cable, noting the resultant changes in force of the other cables. Field measurements are then taken and compared with the matrix to identify cables that have suffered a loss of stiffness. Alternatively, a matrix may be formed using resultant deck elevations instead of resultant cable stiffnesses from the single cable temperature change.
- Accelerometer - this method operates on the vibrating string theory and is an alternative to the laser vibrometer for vibration-based cable force measurements. The accelerometer measures the natural frequency of the cable, which is then used to calculate the tension in the cable from the known length and mass per unit length. This method is generally taken as an estimate, since the vibrating string theory does not take into consideration cable bending stiffness, cable sag, neoprene rings, viscous dampers, and variable stiffness.
- Vibration decay - this method measures the cable damping by inducing high-vibration amplitudes. The cable is then allowed to slow down, decaying the signal of the accelerometer and providing the damping ratio.

16.1.9 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of steel superstructures. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using the NBIS component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBIS component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating.

Element Level Condition State Assessment

In an element level condition state assessment of a cable-supported bridge, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	**Description**
Superstructure	
147	Steel Main Cables (not embedded in concrete)
148	Secondary Steel Cables (not embedded in concrete)

BME No.	**Description**
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating

The unit quantity for cables is feet. The total length cable is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coating is square feet, and the total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of cable-supported bridges:

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
362	Superstructure Traffic Impact (load capacity)

See Chapters 9 and 10 for the inspection and evaluation of concrete and steel girders, floorbeams and stringers.

See Chapter 7 for the inspection and evaluation of decks.

See Chapter 12 for the inspection and evaluation of abutments, piers and bents.

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Chapter 16
Complex Bridges

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Topic 16.2 Movable Bridges

16.2.1 Introduction

This topic serves as an introduction to the highly specialized area of movable bridge inspection. It focuses on the types of movable bridges and special elements associated with the various bridge types.

Each of these specialized bridge types has their unique features and unique mechanisms for movement. The following sections cover common types of movable bridges, including information on the operation, maintenance and inspection of the structure and the movement mechanisms. The inspection of these specialized bridges requires a diverse team collectively capable of inspecting the structure as well as the mechanical, electrical, pneumatic or other movement mechanisms. The duties of the bridge inspector are defined by the bridge owner for the inspection of these type structures and should be complemented, as necessary, by duties of other inspectors for the inspection of the movement mechanisms and by the duties for maintenance and operation personnel. The bridge inspector should confirm with the owner their role in the inspection of these specialized structures (see Figure 16.2.1).

Figure 16.2.1 Movable Bridge

Movable bridges are normally constructed only when fixed bridges are either too expensive or impractical. Movable bridges are constructed across designated “Navigable Waters of the United States”, in accordance with “Permit Drawings” approved by the U.S. Coast Guard. When a movable bridge is fully open, it must provide the channel width and the underclearance shown on the Permit Drawings (see Figure 16.2.2). If the bridge cannot be opened to provide these clearances, notify the U.S. Coast Guard immediately and take action to restore the clearances. If that is impossible, an application must be submitted to revise the Permit Drawings.

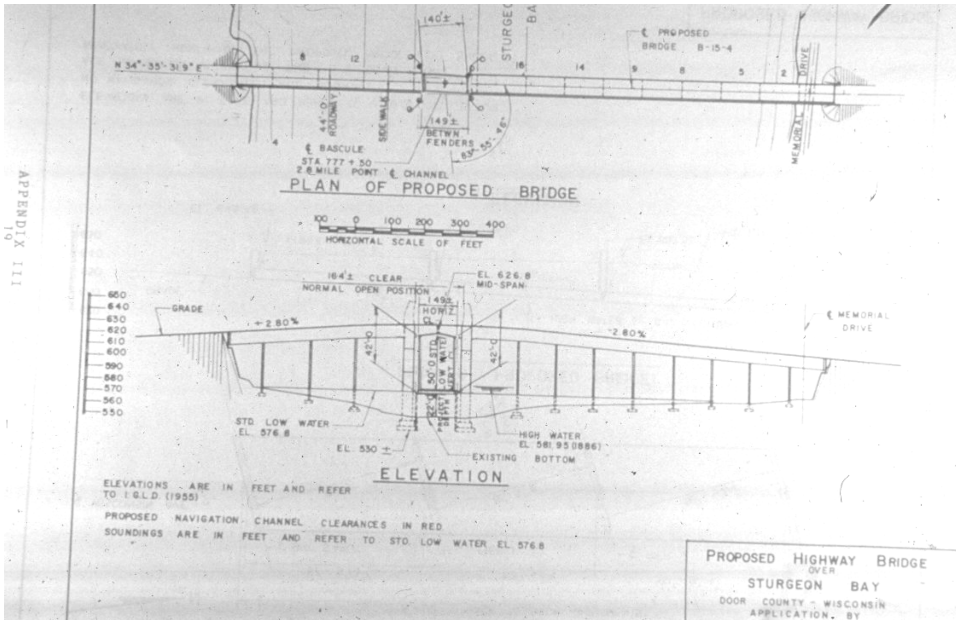

Figure 16.2.2 Typical "Permit Drawing" Showing Channel Width and Underclearance in Closed and Open Position

If any work is to be done in the channel or on the movable span to reduce the clearances from those shown on the Permit Drawing, obtain an additional permit, from the U.S. Coast Guard District covering the scheduled time for the work.

The U.S. Coast Guard publishes Local Notices to Mariners to keep waterway users informed of work in progress that may affect navigation. The permittee keeps the U.S. Coast Guard informed of all stages of construction.

Verify that the bridge conforms to the Permit Drawing and that the operator is instructed to open the bridge to the fully open position every time the bridge is operated. Failure to do this would establish a precedent that a vessel is expected to proceed before the green navigation lights have turned "on". Any accident caused as a result of this practice could be ruled the fault of the bridge owner.

Early America's engineering literature did not establish where the first iron drawbridge was built. The first all-iron movable bridge in the Midwest was completed in 1859 carrying Rush Street over the Chicago River (see Figure 16.2.3). The bridge was a rim bearing swing span and was probably operated by steam. It was destroyed on November 3, 1863 when it was opened while a drove of cattle was on one end. It was rebuilt but destroyed by the great Chicago fire of 1871.

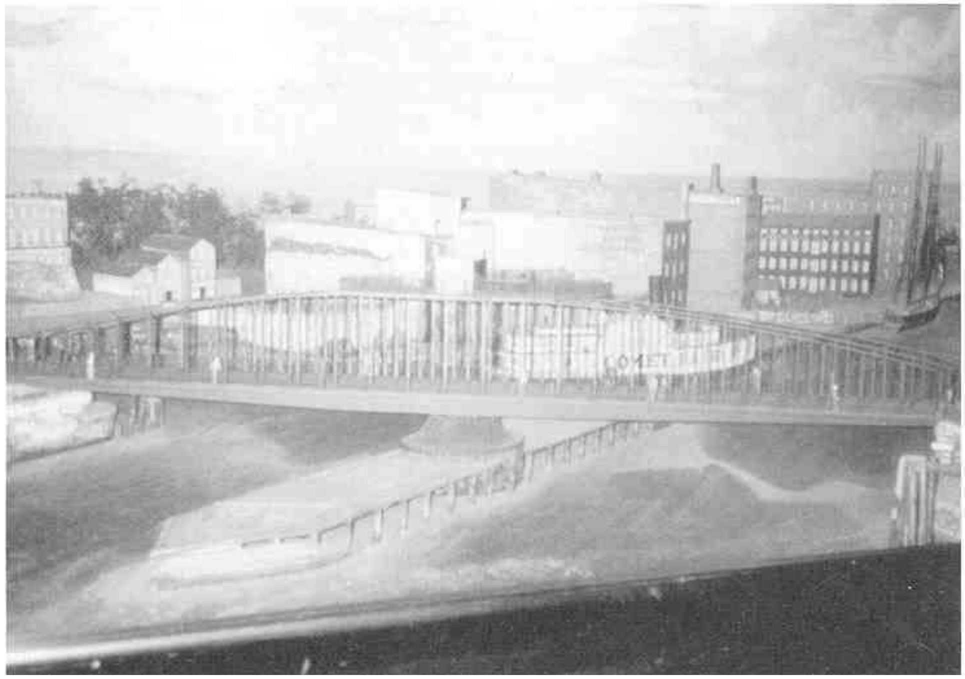

Figure 16.2.3 The First All-Iron Movable Bridge in the Midwest was Completed in 1859 (Photo on File at the Chicago Historical Society)

All categories of movable bridges are powered by electric-mechanical or hydraulic-mechanical drives with power driven pinions operating against racks, or by hydraulic cylinders. A small number are hand powered for normal operation. A few bridges use hand power for standby operation. Three categories of movable bridges comprise over 95 percent of the total number of movable bridges within the United States. These categories include:

- Swing bridges
- Bascule bridges
- Vertical lift bridges

16.2.2 Swing Bridges Design Characteristics

Swing bridges consist of two-span trusses or continuous girders, which rotate horizontally about the center (pivot) pier (see Figure 16.2.4). The spans are usually, but not necessarily, equal. When open, the swing spans are cantilevered from the pivot (center) pier and must be balanced longitudinally and transversely about the center. When closed, the spans are supported at the pivot pier and at two rest (outer) piers or abutments. In the closed condition, wedges are usually driven under the outer ends of the bridge to lift them, thereby providing a positive reaction sufficient to offset any possible negative reaction from live load and impact in the other span. This design feature prevents uplift and hammering of the bridge ends under transient live load conditions.

Figure 16.2.4 Center-Bearing Swing Bridge

Swing spans are subdivided into two types:

- Center-bearing
- Rim-bearing

Center-Bearing

Center-bearing swing spans carry the entire load of the bridge on a central pivot (usually metal discs). Balance wheels are placed on a circular track around the outer edges of the pivot pier to prevent tipping (see Figures 16.2.5 and 16.2.6). When the span is closed, wedges similar to those at the rest piers are driven under each truss or girder at the center pier. This relieves the center bearing from carrying any live load. However, these wedges do not raise the span at the pivot pier, but are merely driven tight.

The latest swing spans built are nearly all of the center-bearing design. Center-bearing swing spans are less complex and less expensive to build than rim-bearing swing spans.

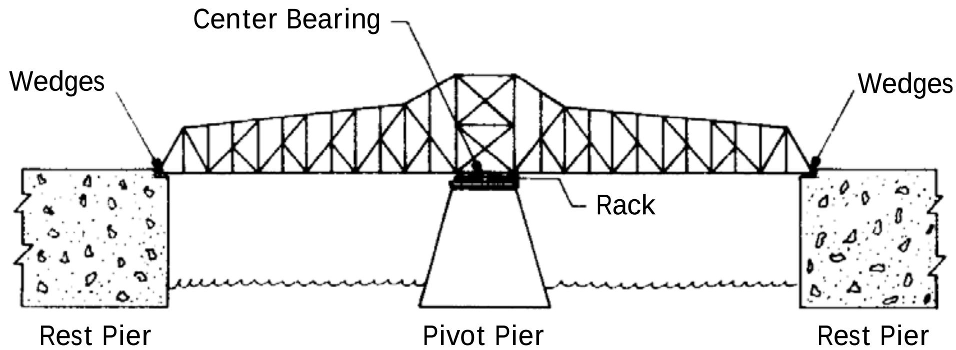

Figure 16.2.5 Center-Bearing Swing Span in Closed Position

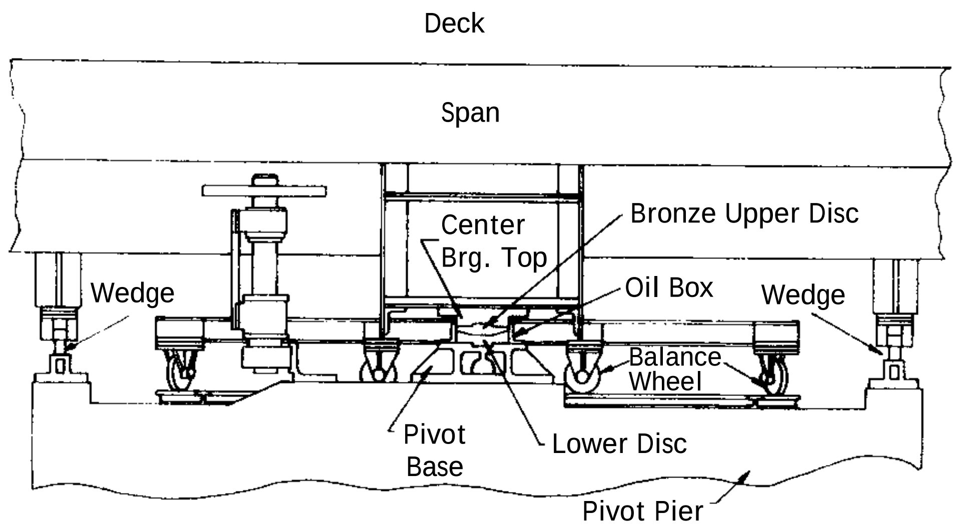

Figure 16.2.6 Layout of Center-Bearing Type Swing Span with Machinery on the Span

Rim-Bearing

Rim-bearing swing spans transmit all loads, both dead and live, to the pivot pier through a circular girder or drum to beveled rollers. The rollers move on a circular track situated inside the periphery of the pier. The rollers are aligned and spaced on the track by concentric spacer rings. This type of swing span bridge also has a central pivot bearing which carries part of the load. This pivot bearing is connected to the rollers by radial roller shafts and keeps the span centered on the circular track.

On both types of swing bridges, the motive power is usually supplied by electric motor(s), hydraulic motor(s), or hydraulic cylinder(s), although gasoline engines or manual power may also be used. The bridge is rotated horizontally by a circular rack and pinion arrangement, or cylinders.

16.2.3 Bascule Bridges Design Characteristics

Bascule bridges open by rotating a leaf or leaves (movable portion of the span) from the normal horizontal position to a point that is nearly vertical, providing an open channel of unlimited height for marine traffic (see Figure 16.2.7).

Figure 16.2.7 Bascule Bridge in the Open Position

If the channel is narrow, a single span may be sufficient. This is called a single-leaf bascule bridge. For wider channels, two leaves are used, one on each side of the channel. When the leaves are in the lowered position, they meet at the center of the channel. This is known as a double-leaf bascule bridge.

A counterweight is necessary to hold the raised leaf in position. In older bridges, the counterweight is usually overhead, while in more modern bascule bridges, the counterweight is placed below the deck and lowers into a pit as the bridge is opened.

The leaf lifts up by rotating vertically about a horizontal axis. The weight of the counterweight is adjusted by removing or adding balance blocks in pockets to position the center of gravity of the moving leaf at the center of rotation. When the bridge is closed, a forward bearing support located in front of the axis is engaged and takes the live load reaction. On double-leaf bascule bridges, a tail-lock behind the axis and a shear lock at the junction of the two leaves are also engaged to stiffen the deck.

There are many types of bascule bridges, but the most common are the following three types:

- Rolling lift (Scherzer) bridge
- Simple trunnion (Chicago) bridge
- Multi-trunnion (Strauss) bridge

Rolling Lift (Scherzer) Bridge

The first rolling lift bridge was completed in 1895 in Chicago, and was designed by William Scherzer. The entire moving leaf, including the front arm with the roadway over the channel and the rear arm with the counterweight, rolls away from the channel while the moving leaf rotates open (see Figures 16.2.8 and 16.2.9). On this type of bridge, curved tracks are attached to each side of the tail end of the leaf. The curved tracks roll on flat, horizontal tracks mounted on the pier. Square or oblong holes are machined into the curved tracks. The horizontal tracks have lugs (or teeth) to mesh with the holes preventing slippage as the leaf rolls back on circular castings whose centerline of roll is also the center of gravity of the moving leaf.

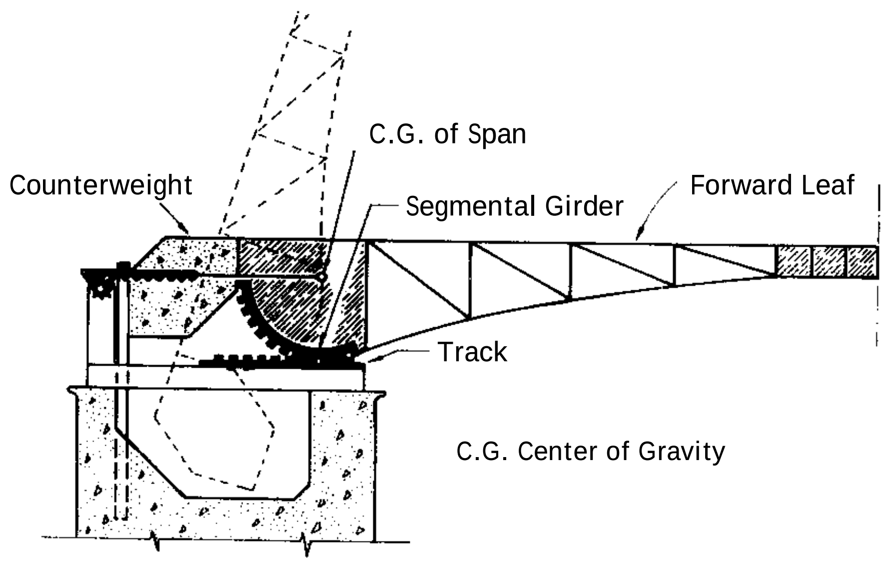

Figure 16.2.8 Rolling Lift Bascule Bridge Schematic

Figure 16.2.9 Double-Leaf Rolling Lift Bascule

The simple principal of this type of bridge can be seen easiest with a railroad bridge. The dead load of the bridge is balanced about the centerline of the drive pinion (center of roll). The pinion teeth are engaged with the teeth on the rack casting. When the pinion turns it moves along on the fixed rack and causes the span to rotate on the circular tread casting as it rolls back on the track casting.

The weight of the leaf, including the superstructure and counterweight, is supported by the curved tracks resting on the horizontal tracks. The counterweight is positioned to balance the weight of the leaf.

On one variation of this type, the trusses on the two leaves acted as three-hinged arches when closed. Locks are engaged in the closed position, allowing the bridge to function as a simple span. In the open position, the leaves operate as a cantilever span. There is a 310 feet span between the centerline of bearings. This bridge was built across the Tennessee River at Chattanooga in 1915, and it is believed to be the third longest double-leaf bascule in the world. It provides an 295 foot channel, which is the widest channel spanned by a bascule bridge.

Simple Trunnion (Chicago) Bridge

The Chicago Bridge Department staff of Engineers built the first Chicago type simple trunnion bascule bridge in 1902. This type of bascule bridge consists of a forward cantilever arm out over the channel and a rear counterweight arm (see Figure 16.2.10). The leaf rotates about the trunnions. Each trunnion is supported on two bearings, which in turn, are supported on the fixed portion of the bridge such as trunnion cross-girder, steel columns, or on the pier itself (see Figures 16.2.11, 16.2.12 and 16.2.13). Forward bearing supports located in front of the trunnions are engaged when the leaf reaches the fully closed position. They are intended to support only live load reaction. Uplift supports are located behind the trunnions to take uplift until the forward supports are in contact (if misadjusted) and to take the live load uplift that exceeds the dead load reaction at the trunnions. If no forward live load supports are provided or if they are grossly misadjusted, the live load and the reaction at the uplift supports are added to the load on the trunnions. A double-leaf bascule bridge of this type in Lorain, Ohio has 333 feet between trunnions. Of the three types of movable bridges, the simple trunnion is by far the most popular.

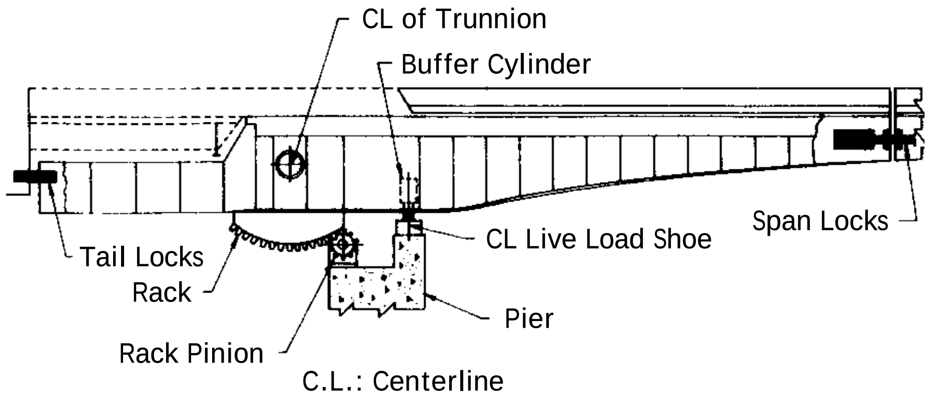

Figure 16.2.10 Trunnion Bascule Bridge Schematic

Figure 16.2.11 Double-Leaf Trunnion Bascule Bridge

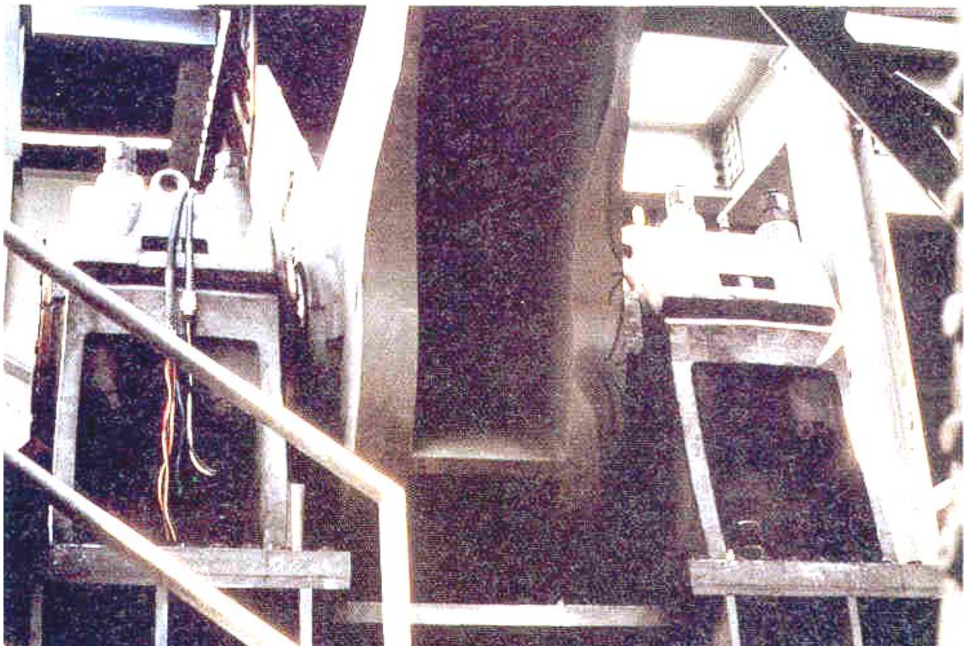

Figure 16.2.12 Each Trunnion is Supported on Two Bearings

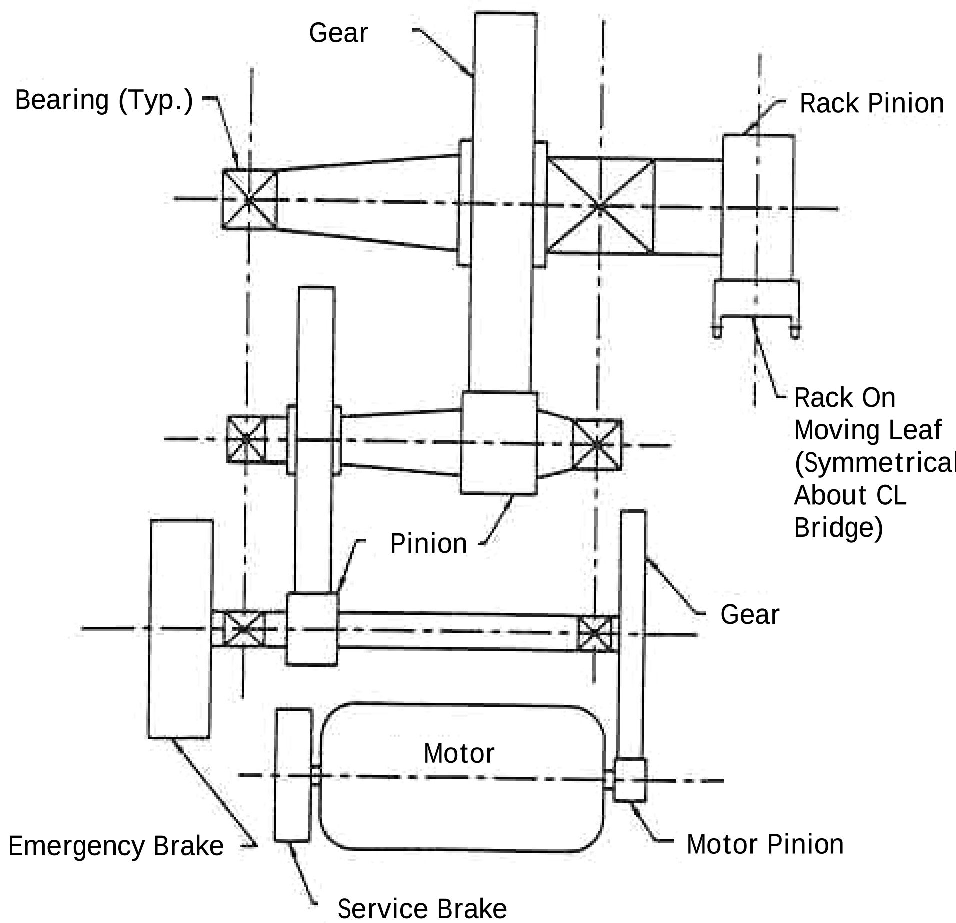

Figure 16.2.13 Trunnion Bascule Bridge Machinery (One Quarter Shown) is Located Outside of the Bascule Trusses on the Pier

Multi-Trunnion (Strauss Bridge)

The first multi-trunnion (Strauss) bascule bridge was designed by J.B. Strauss and completed during 1905 in Cleveland, Ohio. There are many variations of multi-trunnion bascule bridges, but basically one trunnion supports the moving span, one trunnion supports the counterweight, and two link pins are used to form the four corners of a parallelogram-shaped frame that changes angles as the bridge is operated. The counterweight link keeps the counterweight hanging vertically from the counterweight trunnions while the moving leaf rotates about the main trunnions (see Figure 16.2.14). One variation of this parallelogram layout is the heel trunnion. A double-leaf bascule bridge of this type in Sault St. Marie, Michigan has 336 feet between the span trunnions. It was built across the approach to a lock in 1914, and it is believed to be the longest double-leaf bascule in the world.

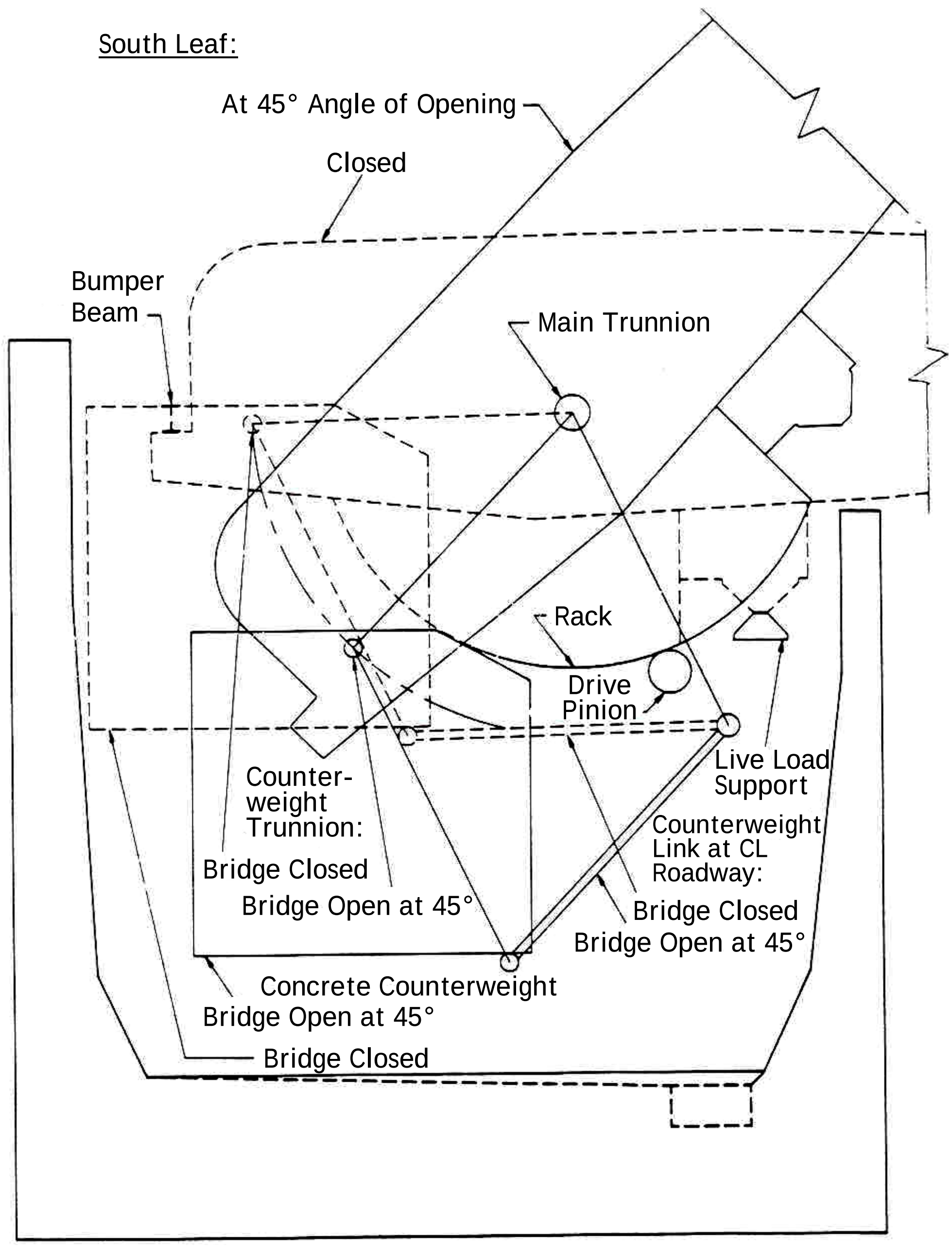

Figure 16.2.14 Multi-Trunnion, Strauss Type Bascule Bridge

16.2.4 Vertical Lift Bridges Design Characteristics

Vertical lift movable bridges have a movable span with a fixed tower at each end. The span is supported by steel wire ropes at its four corners. The ropes pass over sheaves (pulleys) atop the towers and connect to counterweights on the other side. The counterweights descend as the span ascends (see Figure 16.2.15).

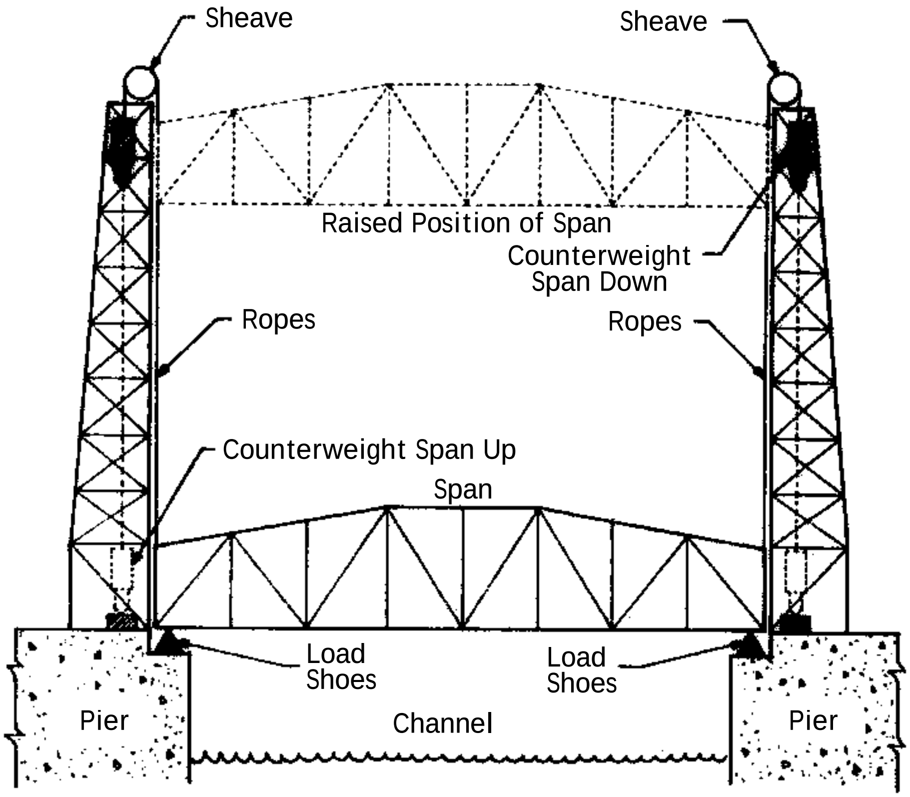

Figure 16.2.15 Vertical Lift Bridge Schematic

There are two basic types of vertical lift bridges:

- Power and drive system on lift span
- Power and drive system on towers

Power and Drive System on Lift Span

The first vertical lift bridge completed during 1894 in Chicago was designed by J.A.L. Waddell. This bridge type locates the power on top of the lift truss span. The actual lifting is accomplished using "up-haul and down-haul ropes" where turning drums wind the up-haul (lifting) ropes as they simultaneously unwind the down-haul ropes. Vertical lift bridge machinery is located on top of the lift truss span, and the operating drums rotate to wind the up-haul (lifting) ropes as they simultaneously unwind the down-haul ropes (see Figure 16.2.16). A variation of this type provides drive pinions at both ends of the lift span which engage racks on the towers.

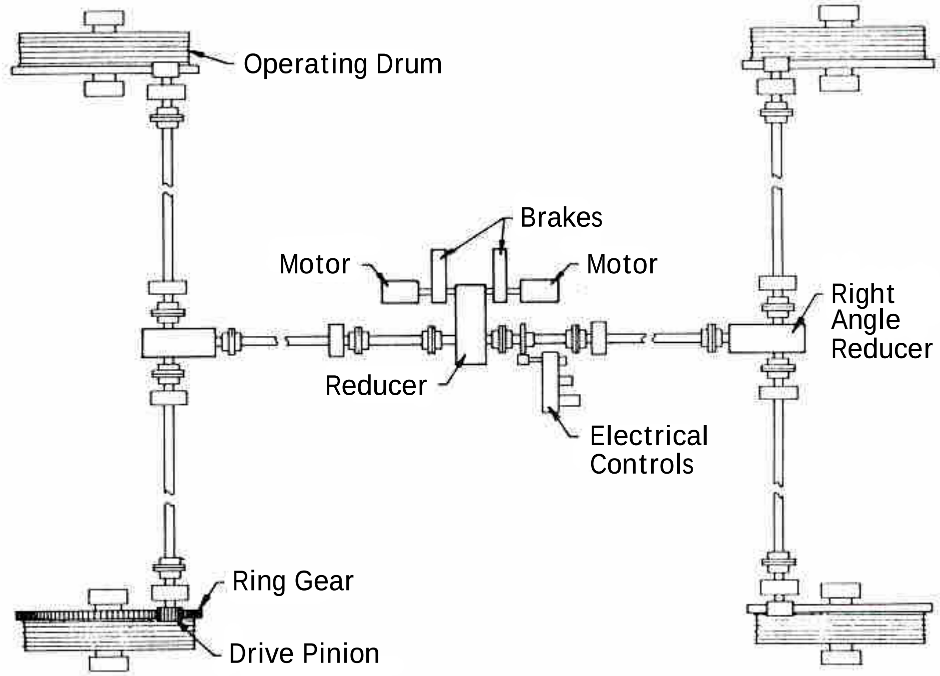

Figure 16.2.16 Vertical Lift Bridge Machinery is Located on Top of the Lift Truss Span, and the Operating Drums Rotate to Wind the Up-Haul (Lifting) Ropes as They Simultaneously Unwind the Down-Haul Ropes

Power and Drive System on Towers

The other basic type of vertical lift bridge locates the power on top of both towers, where drive pinions operate against circular racks on the sheaves. The lifting speed at both towers must be synchronized to keep the span horizontal as it is lifted (see Figures 16.2.17 and 16.2.18).

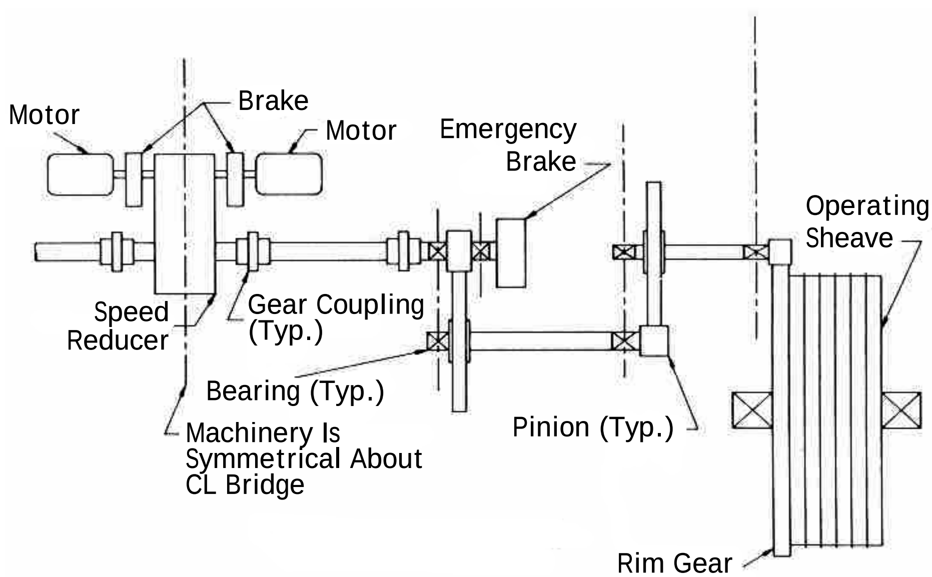

Figure 16.2.17 Vertical Lift Bridge Machinery is Located on the Towers, and the Rim Gears (and Operating Sheaves) are Rotated to Raise and Lower the Bridge

Figure 16.2.18 Vertical Lift Bridge with Power and Drive System on Towers

16.2.5 Special Elements Common to All Movable Bridges

Give particular attention to the special elements found in swing bridges, bascule bridges, and vertical lift bridges during inspection. These elements are commonly found on all types of movable bridges:

- Open Gearing
- Speed Reducers Including Differentials
- Shafts and Couplings
- Bearings
- Brakes
- Drives
- Air Buffers and Shock Absorbers
- Span Locks
- Counterweights
- Live Load shoes and Strike Plates
- Traffic Barriers

Open Gearing

Open gearing is used to transmit power from one shaft to another and to alter the speed and torque output of the machinery. Beveled gears are also used to change direction (see Figure 16.2.19).

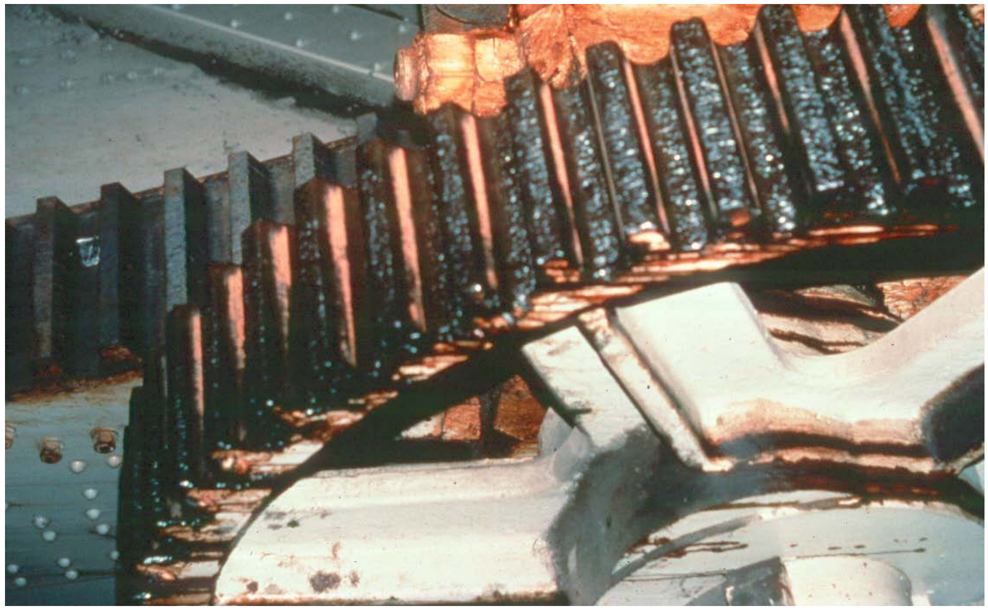

Figure 16.2.19 Open Gearing

Speed Reducers Including Differentials

Speed reducers including differentials serve the same function as open gearing (see Figure 16.2.20). However, they may contain several gear sets, bearings, and shafts to provide a compact packaged unit, which protects its own mechanical elements and lubrication system with an enclosed housing. Differential speed reducers also function to equalize torque and speed from one side of the mechanical operating system to the other.

Figure 16.2.20 Speed Reducer

Shafts and Couplings

Shafts transmit mechanical power from one part of the machinery system to another. Couplings transmit power between the ends of shafts in line with one another, and several types can be used to compensate for slight imperfections in alignment between the shafts (see Figure 16.2.21).

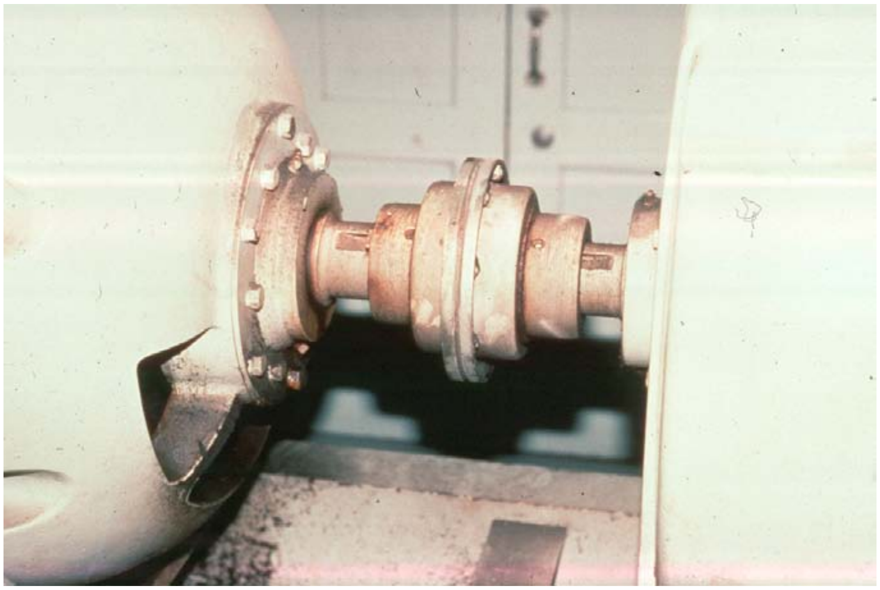

Figure 16.2.21 Coupling

Bearings

Bearings provide support and prevent misalignment of rotating shafts, trunnions, and pins (see Figure 16.2.22).

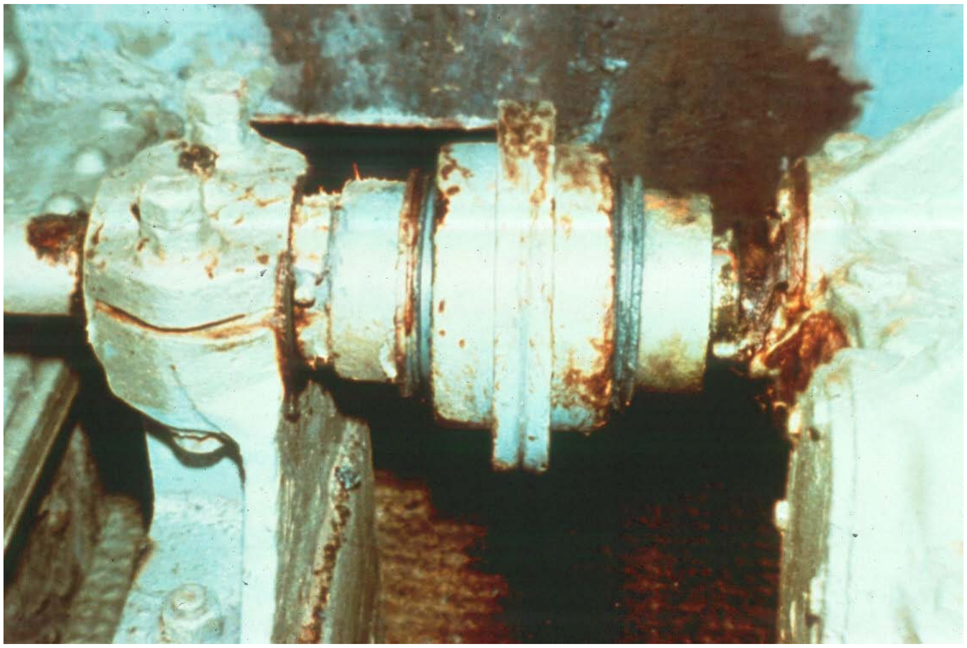

Figure 16.2.22 Bearing

Brakes

Brakes can be of either the shoe type or disc type, and can be released manually, electrically, or hydraulically (see Figures 16.2.23 and 16.2.24). They are generally spring applied for fail safe operation. Motor brakes are located close to the drive to provide dynamic braking capacity, except that some types of drives can provide their own braking capability, thereby eliminating the need for separate motor brakes. Machinery brakes are located closer to the operating interface between movable and fixed parts of the bridge and are used to hold the span statically, in addition to serving as emergency brakes in many cases. Supplemental emergency brakes are sometimes also provided.

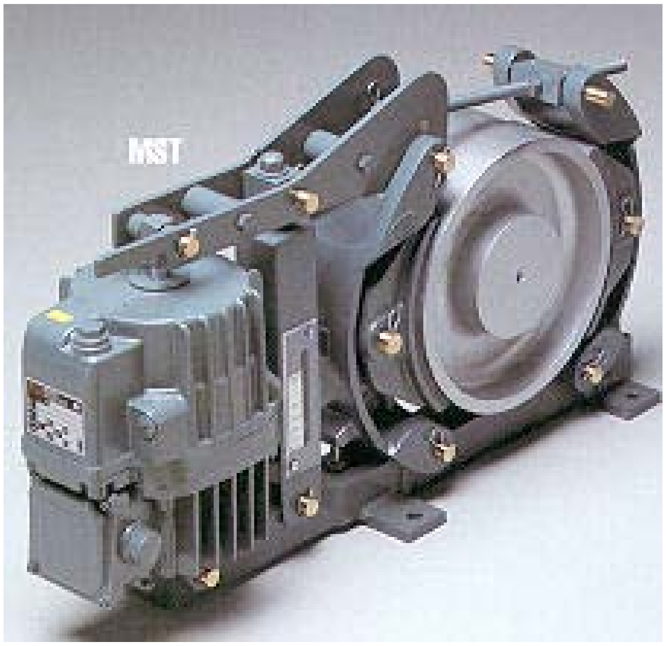

Figure 16.2.23 Shoe Type Break

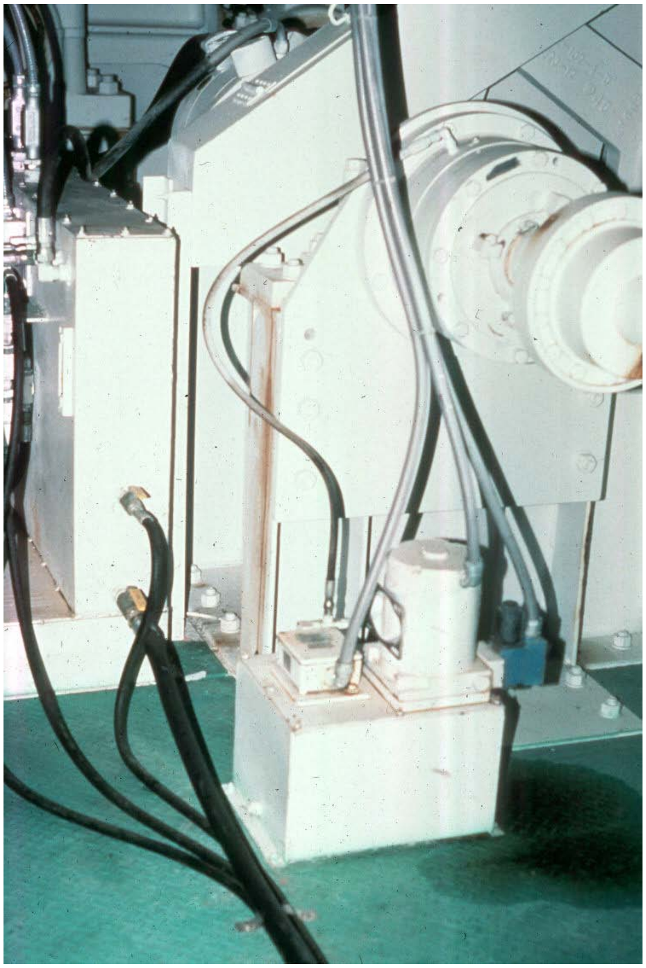

Figure 16.2.24 Spring Set Hydraulically Released Disc Break

Drives

Drives can consist of electric motors, hydraulic equipment, or auxiliary drives.

For electric motors, either AC or DC power may be used. AC power is often used to power wound rotor motors with torque controllers on older bridges, while new bridges may utilize squirrel cage induction motors with adjustable frequency speed control. DC motors can also provide speed control.

For hydraulic equipment, prime movers may include either large actuating cylinders or hydraulic motors (see Figure 16.2.25). Either type of drive must be supplied with pressure to provide force and fluid flow to provide speed to the operating system. Electrically operated hydraulic power units consisting of a reservoir and pump, with controls, provide power to the operating systems.

For auxiliary drives, emergency generators are provided to serve in the event of power failure. Auxiliary motors and hand operators, with their clutches and other mechanical power transmission components, are provided to serve in the event the main drive fails (see Figure 16.2.26). In some cases, to prevent the need for larger auxiliary generators, the auxiliary motors are required for use any time the auxiliary generators are used, requiring increased time of operation.

Figure 16.2.25 Low Speed High Torque Hydraulic Motor

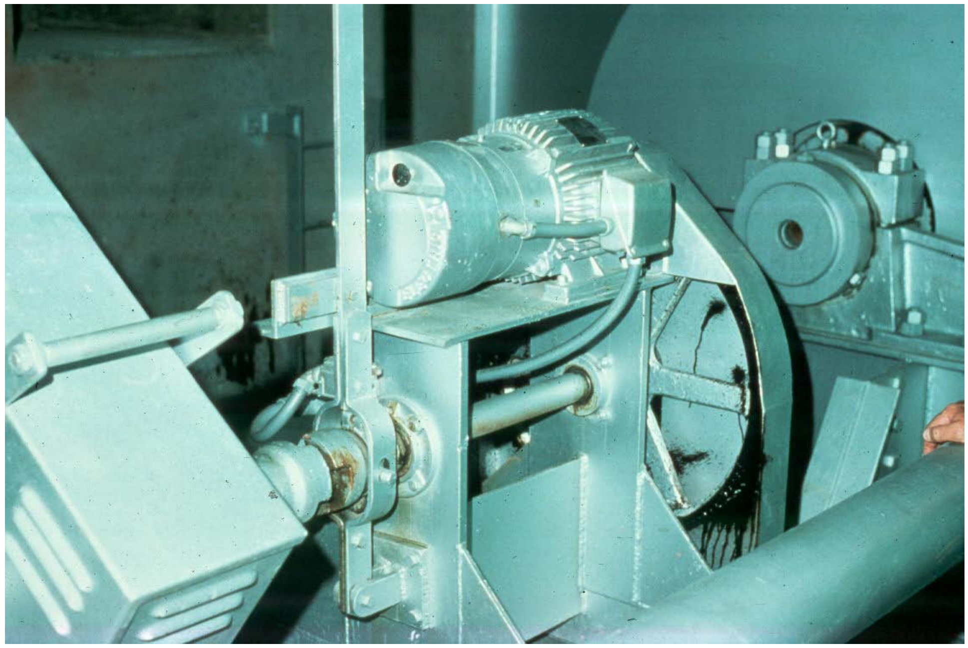

Figure 16.2.26 AC Emergency Motor

Air Buffers and Shock Absorbers

Air buffers and shock absorbers are located between the span and the pier at points where impact may occur between the two (see Figures 16.2.27 and 16.2.28). A cross section of the buffer shows the air chamber and seals on the piston. As the span lowers, the rod is pushed in, causing the air inside to be compressed (see Figure 16.2.29). A pressure relief valve allows the air to escape beyond the pressure setting. Forces are required to build-up and keep the pressure of the air at the movement of the span for a "soft" touchdown on the bearings. Shock absorbers provide the same purpose as the air buffers. However, they are completely self-contained and, therefore, require very little maintenance.

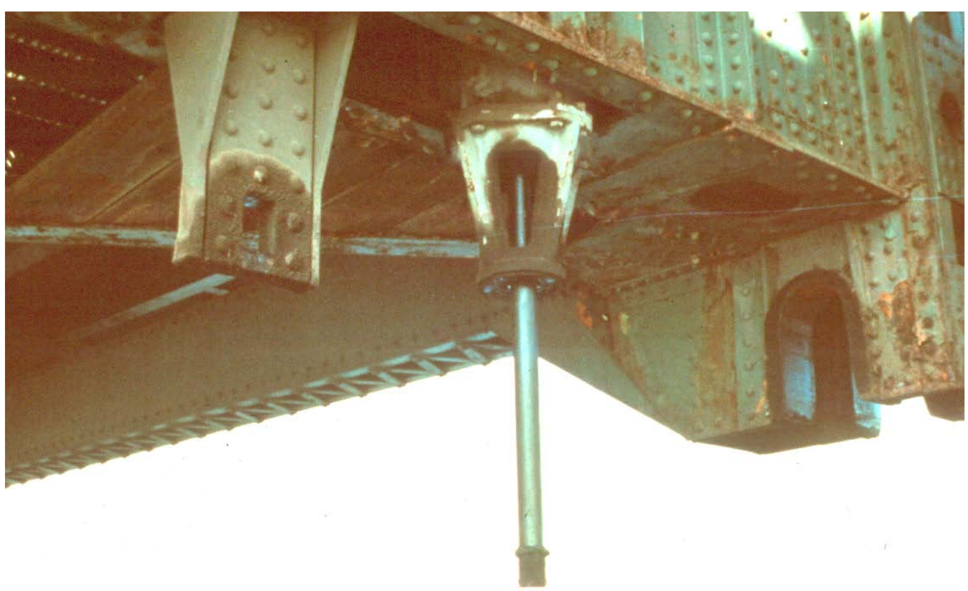

Figure 16.2.27 Air Buffer

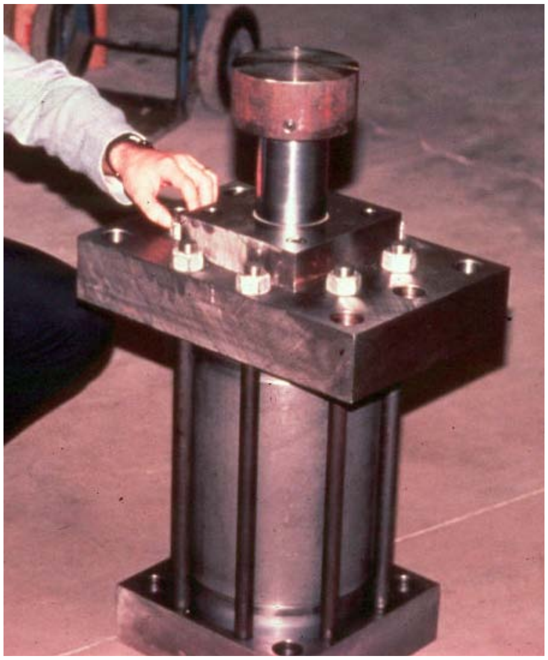

Figure 16.2.28 Shock Absorber

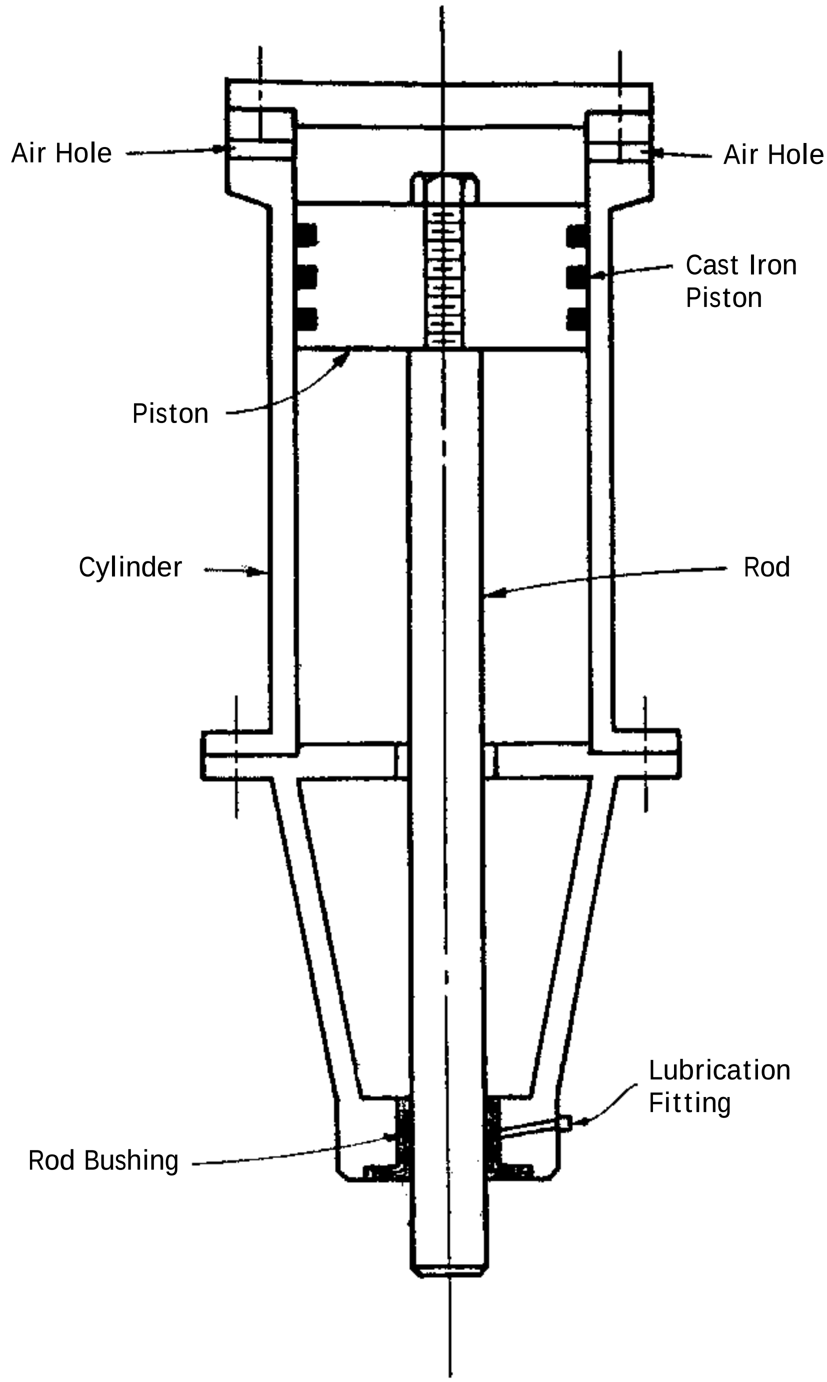

Figure 16.2.29 Typical Air Buffer Schematic

Span Locks

Span lock bars at the end of the span are driven when the span is fully closed to prevent movement under live load. Span locks may also be provided at other locations on the span to hold the span in an open position against strong winds or to prevent movement from an intermediate position. They can be driven either mechanically or hydraulically (see Figures 16.2.30 and 16.2.31).

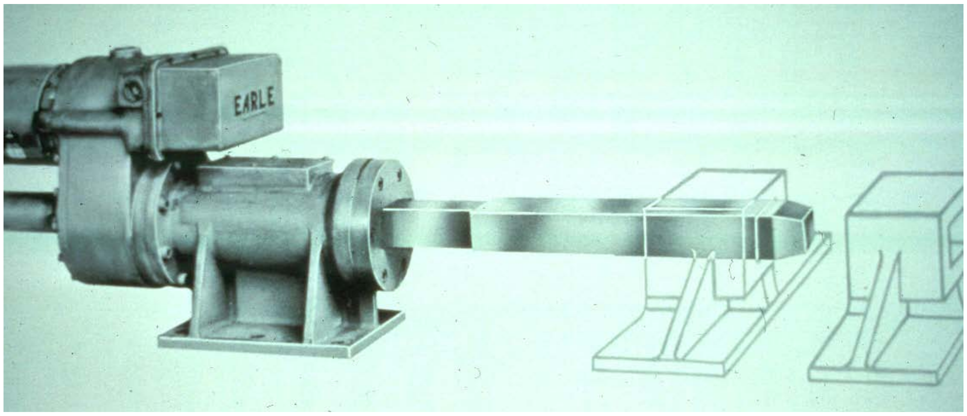

Figure 16.2.30 Typical Mechanically Operated Span Lock

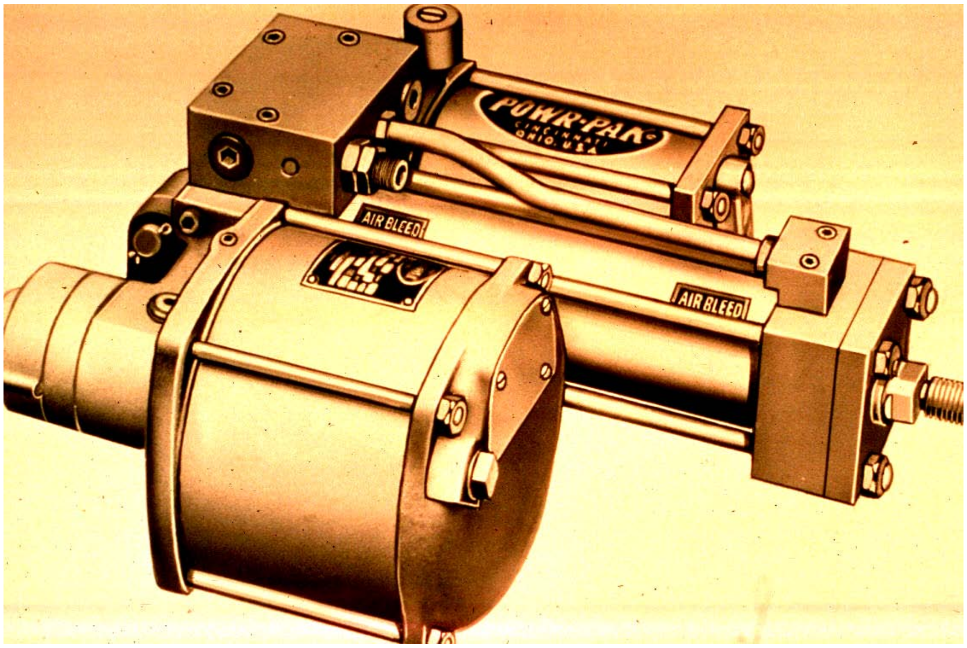

Figure 16.2.31 Hydraulic Cylinder that Drives Lock Bars

Counterweights

Adjustable quantities of counterweight blocks are provided in addition to the permanent counterweight, which is part of the structure so that adjustments may be made from time to time due to changes in conditions (see Figures 16.2.32 and 16.2.33). A movable span is designed to function in a balanced condition, and serious unbalanced conditions will cause overstress or even failure of the mechanical or structural elements.

Figure 16.2.32 Concrete Counterweight on a Single-Leaf Bascule Bridge

Figure 16.2.33 Concrete Counterweight on a Vertical Lift Bridge

Live Load Shoes and Strike Plates

Live load shoes and strike plates between the movable and fixed portions of the bridge are designed to bear most or all of the live load when the bridge is carrying traffic (see Figure 16.2.34).

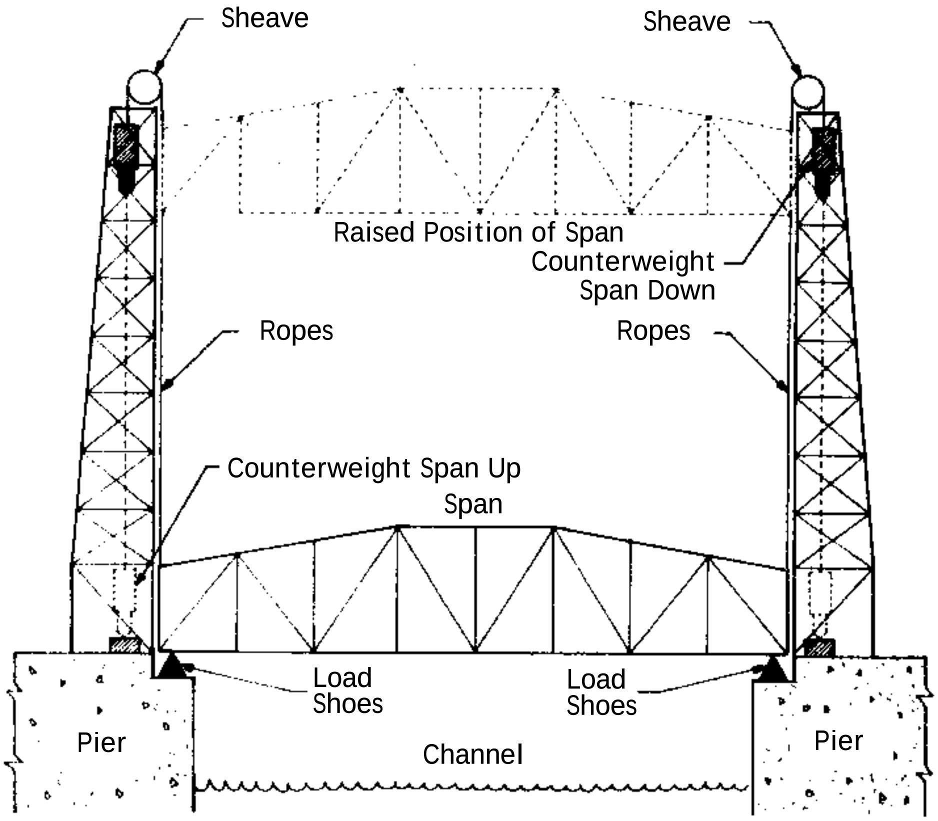

Figure 16.2.34 Closed Span Resting on Live Load Shoes

Traffic Barriers

Traffic barriers are heavy-duty movable gates or posts that are designed to prevent a vehicle from plunging from the roadway into the draw or into the pit below the bridge (see Figure 16.2.35. Their operation is important for public safety. They are used mainly in situations where a large opening exists between the approach span and the movable span when it is open.

Figure 16.2.35 Traffic Barrier

16.2.6 Swing Bridge Special Elements

Swing bridges are designed utilizing the following special elements:

- Pivot Bearings
- Balance Wheels
- Rim-Bearing Rollers
- Wedges
- End Latches

Pivot Bearings

In center-bearing types (with balance wheels), the axially loaded thrust bearing is usually composed of spherical discs, attached to top and bottom bases, enclosed in an oil box to provide lubrication and prevent contamination (see Figure 16.2.36). In rim-bearing types, the pivot bearing is also enclosed but will be radial loaded, maintaining the position of the pivot shaft or king pin.

Figure 16.2.36 Center Pivot Bearing

Balance Wheels

On center-bearing types only, non-tapered balance wheels bear on the circular rail concentric to the pivot bearing only when the span is subjected to unbalanced loading conditions (see Figure 16.2.37). At other times, when the span is not subjected to unbalanced loads, a gap will be present between each wheel and the rail.

Figure 16.2.37 Balance Wheel in-place over Circular Rack

Rim-Bearing Rollers

Usually tapered to allow for the differential rolling distance between the inside and outside circumferences of the rail circle, rim-bearing rollers usually bear at all times.

Wedges

End wedges are used to raise the ends of the span and support live load under traffic (see Figure 16.2.38). The end wedge bearings are under all four corners of the span. Center wedges are used to stabilize the center of the span and to prevent the center bearing from supporting live load. Wedges may be actuated by machinery and linkage, which connects wedges to actuate together or each wedge may have its own actuator (see Figure 16.2.39).

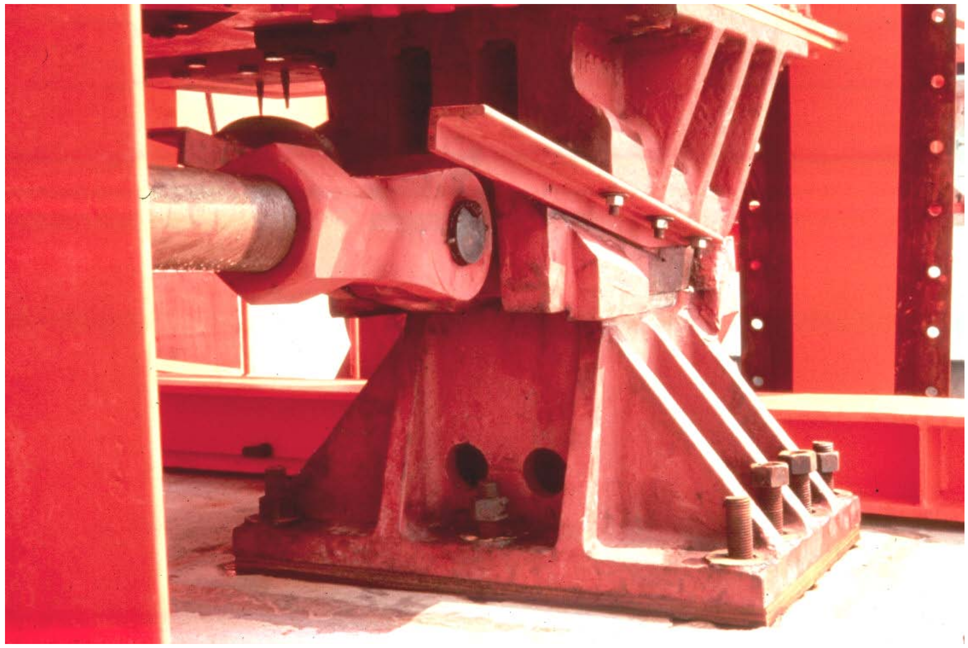

Figure 16.2.38 End Wedge

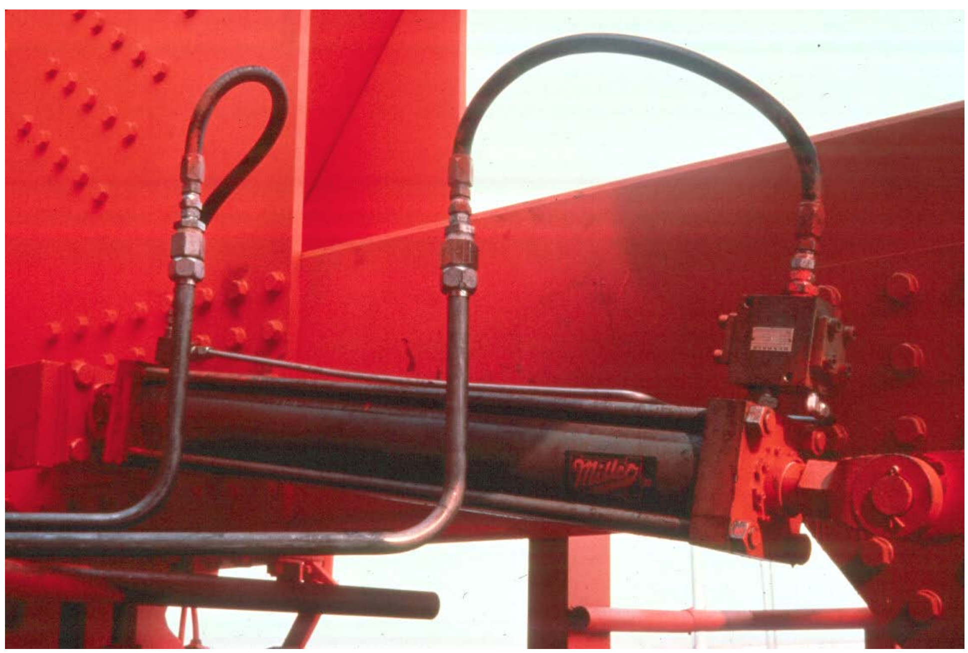

Figure 16.2.39 Hydraulic Cylinder Actuator

End Latches

Located at the center of one or both rest piers, end latches generally consist of a guided tongue with roller mounted on the movable span that occupies a pocket mounted on the rest pier when the span is in the closed position. To open the span, the tongue is lifted until it clears the pocket at the time the wedges are withdrawn (see Figure 16.2.40). As the span is swung open, the latch tongue is allowed to lower or fall into a position in which the roller may follow along a rail or track mounted on the pier. When closing, the tongue rolls along the rail or track and up a ramp which leads to the end latch pocket where the tongue is allowed to drop to center the span.

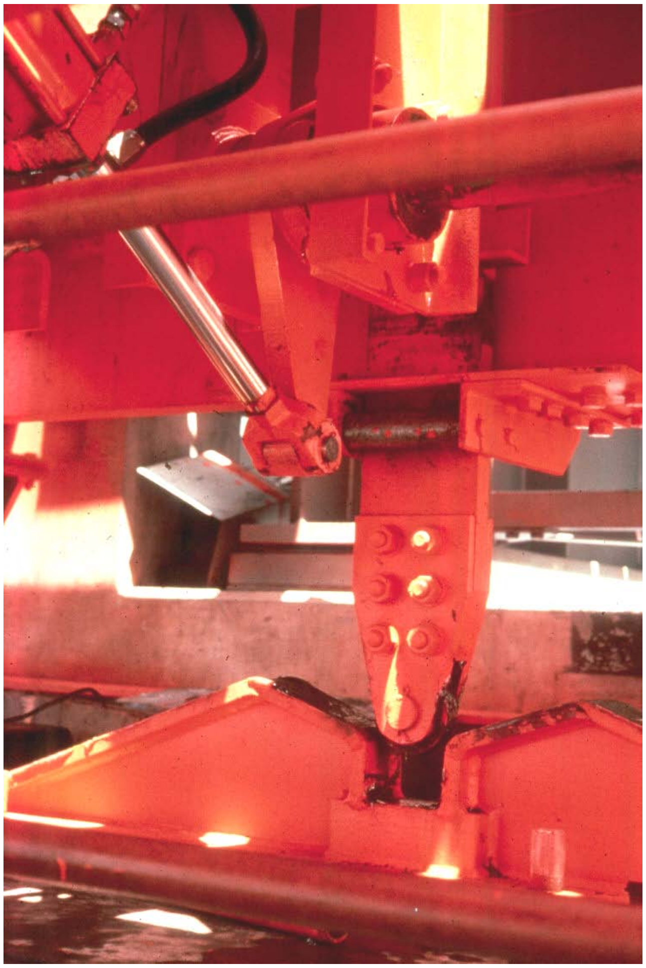

Figure 16.2.40 End Wedges Withdrawn and End Latch Lifted

16.2.7 Bascule Bridge Special Elements

Bascule bridges utilize the following elements specific to their design:

- Rolling Lift Tread and Track Castings
- Racks and Pinions
- Trunnions and Trunnion Bearings
- Hopkins Frame
- Tail (Rear) Locks
- Center Locks
- Transverse Locks

Rolling Lift Tread and Track Castings

Rolling lift tread and track castings are rolling surfaces which support the bascule leaves as they roll open or closed (see Figure 16.2.41). Tread sockets and track teeth prevent transverse and lateral movement of the span due to unbalanced conditions, such as wind, during operation and especially when held in the open position.

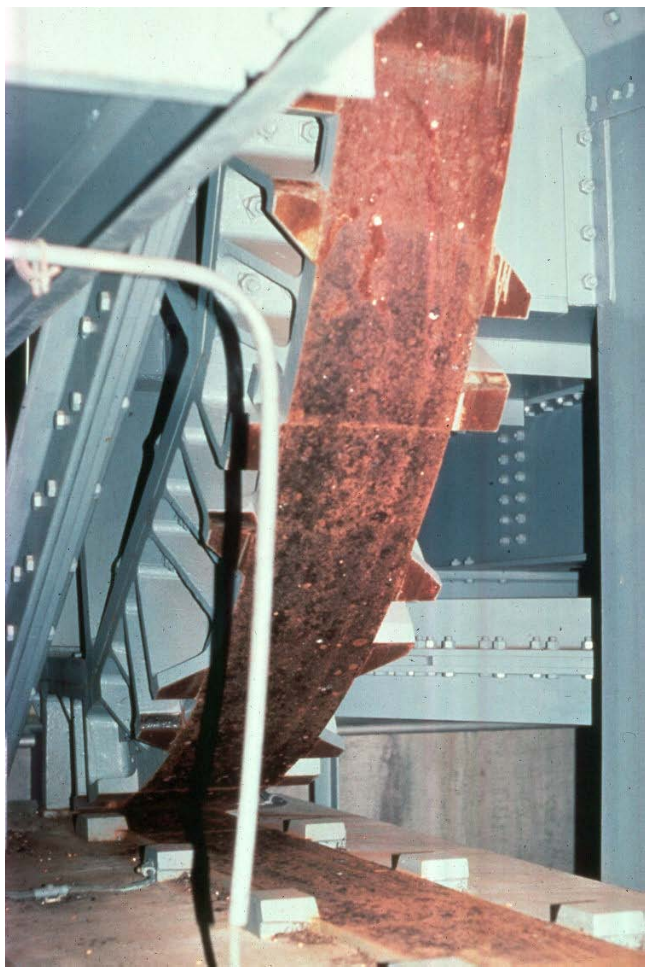

Figure 16.2.41 Circular Lift Tread and Track Castings

Racks and Pinions

In the rolling lift rack and pinion, the driving pinion engages the rack teeth at the centerline of the roll (see Figure 16.2.42).

In the trunnion rack and pinion, the circular rack castings are attached in the plane of the truss (or girder) in front of the counterweight (see Figures 16.2.43 and 16.2.44).

The drive pinions are overhung in order to engage the rack teeth. A cover is placed over the pinions for safety and to keep debris from falling on it.

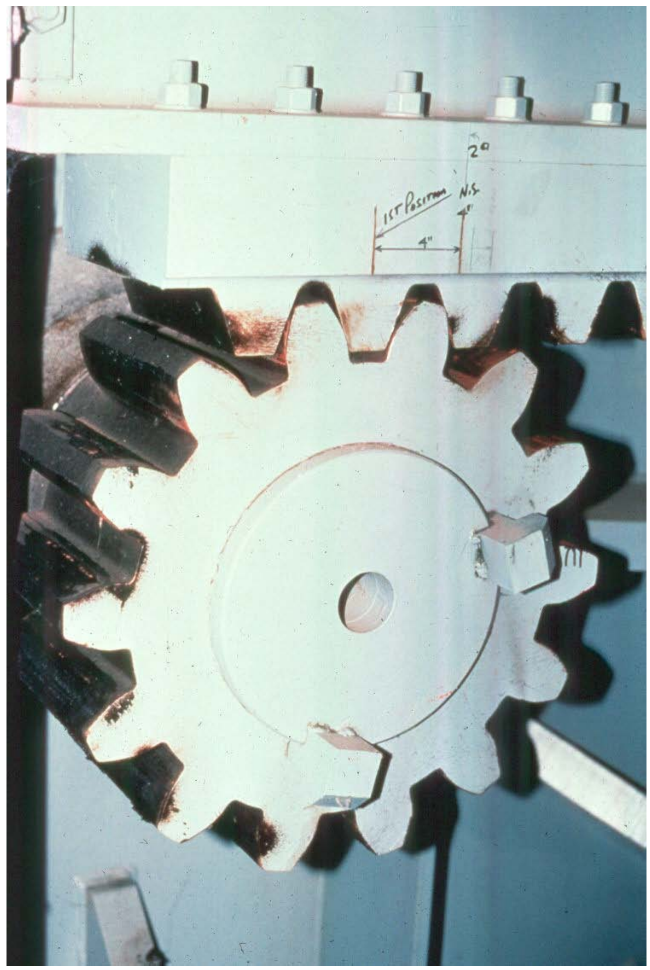

Figure 16.2.42 Rack Casting and Pinion

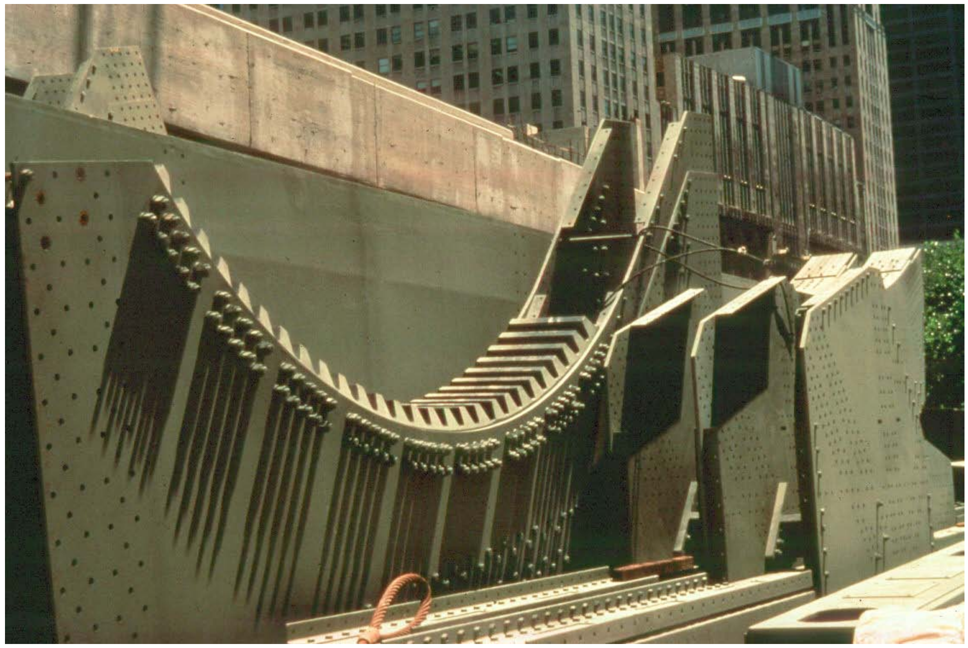

Figure 16.2.43 Rack Casting Ready for Fabrication

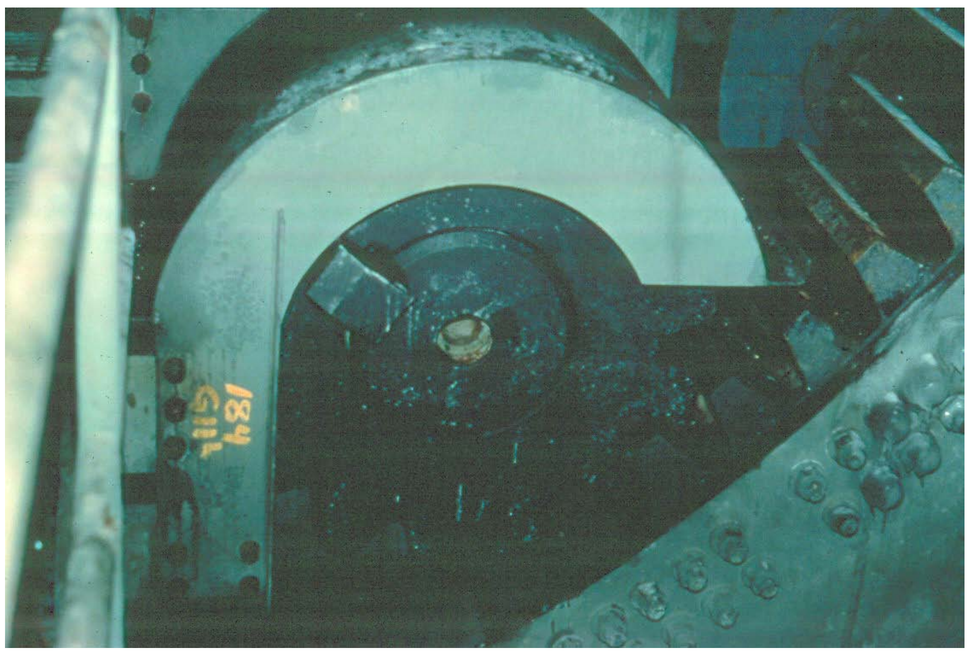

Figure 16.2.44 Drive Pinion

Trunnions and Trunnion Bearings

Trunnions and trunnion bearings (see Figure 16.2.45) are large pivot pins or shafts. Their bearings support the leaf as it rotates during operation as well as supporting dead load when the bridge is closed. Some designs require the trunnions to carry live load in addition to dead load (see Figure 16.2.46).

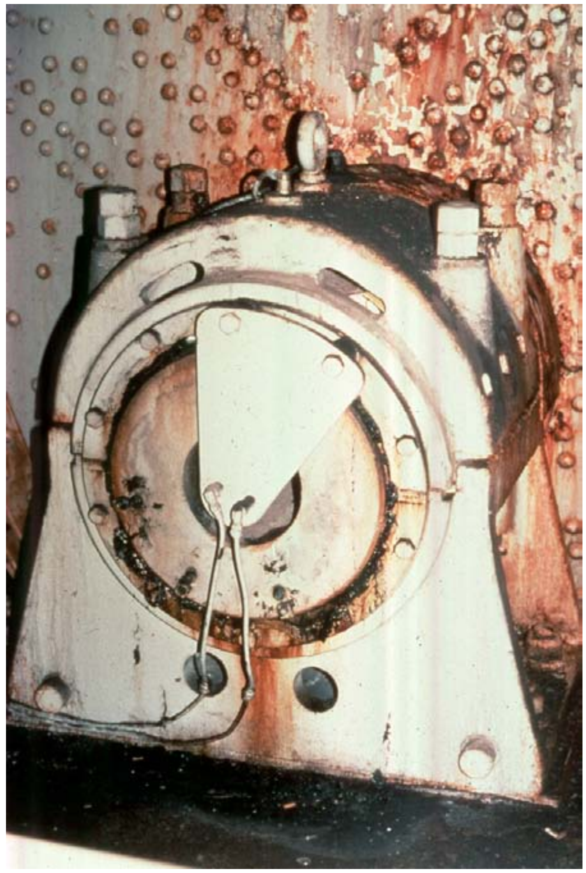

Figure 16.2.45 Trunnion Bearing

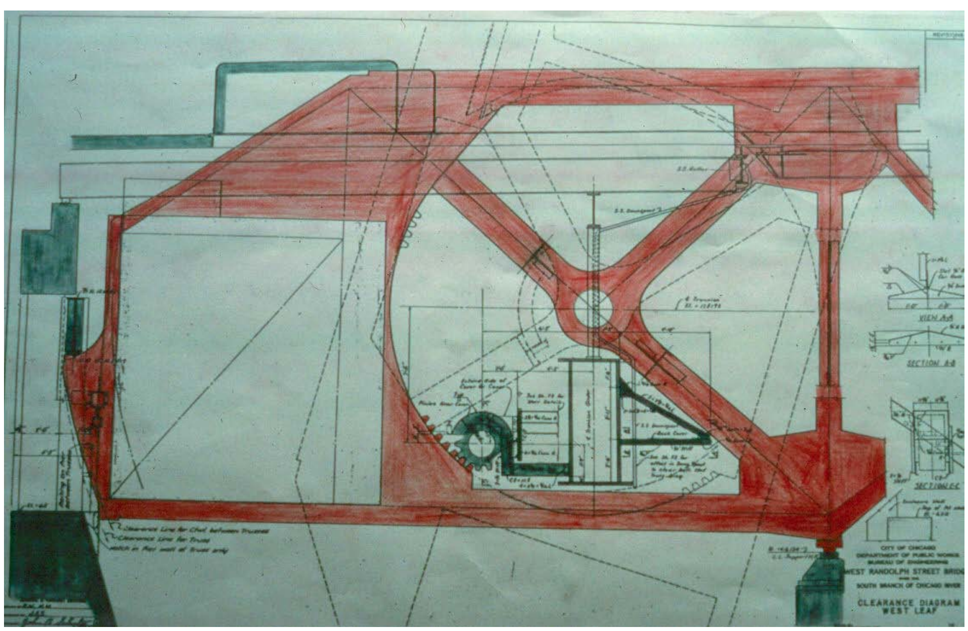

Figure 16.2.46 Trunnion Design Drawing

Hopkins Frame

A Hopkins frame machinery arrangement is provided on some trunnion bascule bridges. The main drive pinion locations are established in relationship to their circular racks by a pivot point on the pier and pinned links attached to the trunnions.

Tail (Rear) Locks

Located at the rear of the bascule girder on the pier, tail locks prevent inadvertent opening of the span under traffic or under a counterweight-heavy condition if the brakes fail or are released (see Figure 16.2.47).

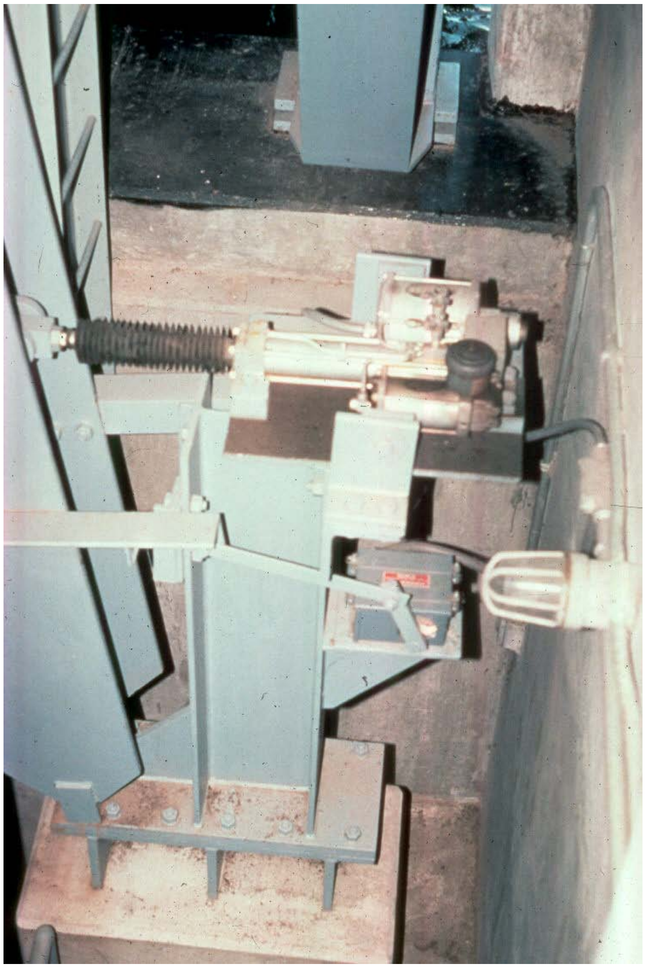

Figure 16.2.47 Rear Lock Assembly

Center Locks

Center locks are provided to transfer shear load from one leaf to the other when the bridge is under traffic. Center locks may consist of a driven bar or jaw from one leaf engaging a socket on the other leaf, or may be a meshing fixed jaw and diaphragm arrangement with no moving parts (see Figure 16.2.48).

The superstructure acts as a cantilever when opening and closing the bridge with the maximum negative moment near the supporting piers and zero moment at the ends of the cantilever. Once the bridge is lowered into position, center locks are engaged. These locking mechanisms are designed to transmit shear necessary to produce equal deflections at mid point under unbalanced transient loads. These center locks are not normally designed to carry superstructure moment.

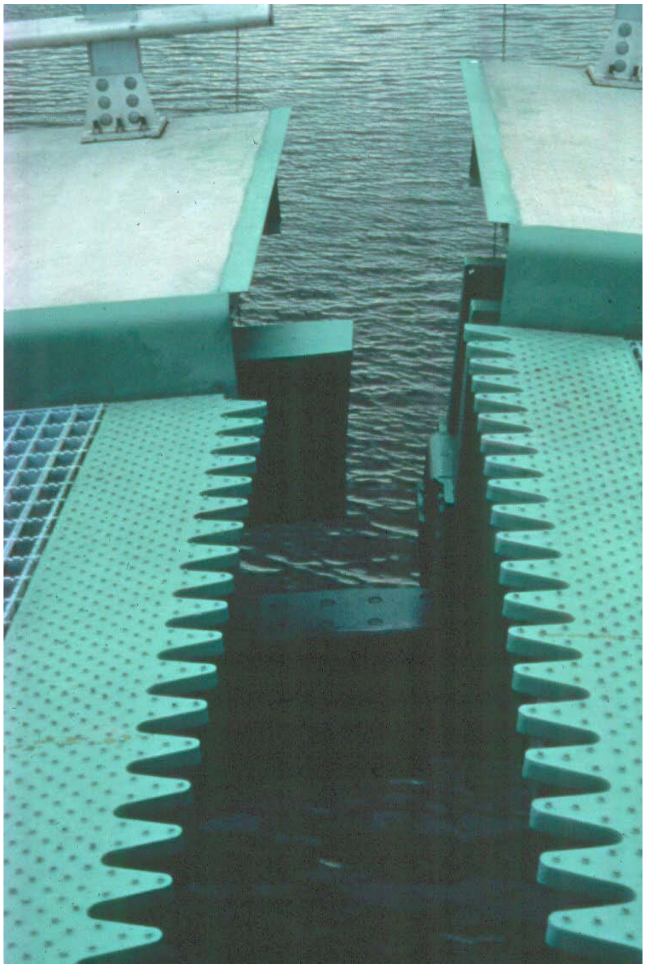

Figure 16.2.48 Center Lock Jaws

Transverse Locks

In twin bascule bridges that are split longitudinally to allow flexibility during construction, repair, or rehabilitation; transverse locks between the inside girders are used to keep the pairs together during operation (see Figure 16.2.49). These are usually operated manually, as they are not normally released for long periods of time.

Figure 16.2.49 Transverse Locks on Underside can be Disengaged

16.2.8

Vertical Lift Bridge Special Elements

Vertical lift bridges may utilize the following elements peculiar to their design:

- Wire Ropes and Sockets
- Drums, Pulleys, and Sheaves
- Span and Counterweight Guides
- Balance Chains
- Span Leveling Devices

Wire Ropes and Sockets

Wire ropes and sockets include up-haul and down-haul operating ropes and counterweight ropes (see Figures 16.2.50 and 16.2.51). Ropes consist of individual wires twisted into several strands that are wound about a steel core. Fittings secure the ends of the rope and allow adjustments to be made.

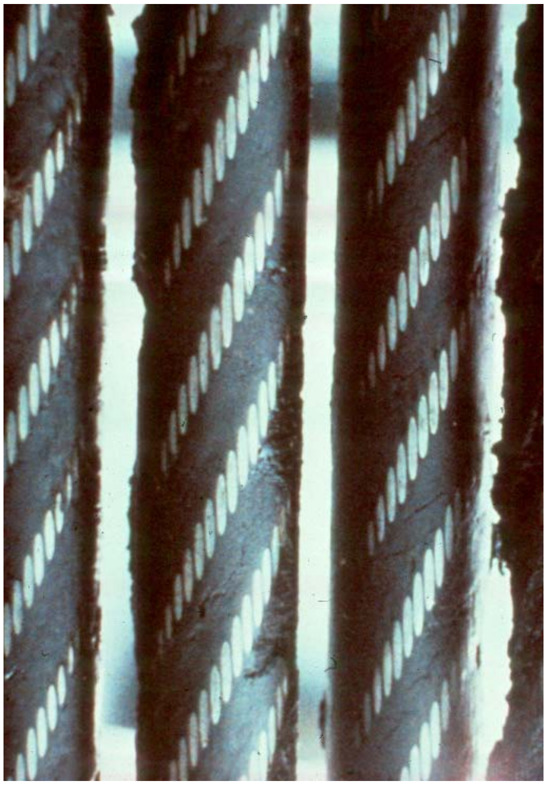

Figure 16.2.50 Wire Rope

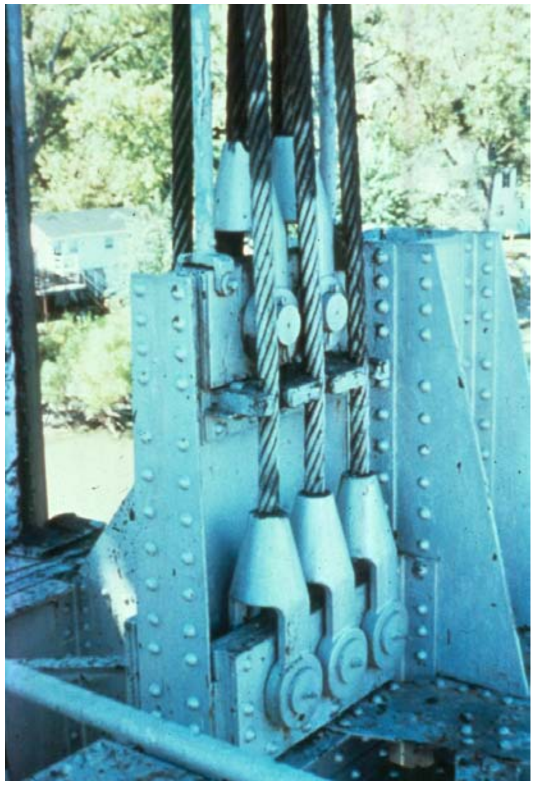

Figure 16.2.51 Wire Rope Sockets and Fittings

Drums, Pulleys, and Sheaves

Drums are used to wind a rope several times around to extend or retract portions of the bridge (see Figure 16.2.52). Pulleys and sheaves change the direction of the rope or guide it at intermediate points between ends of the rope.

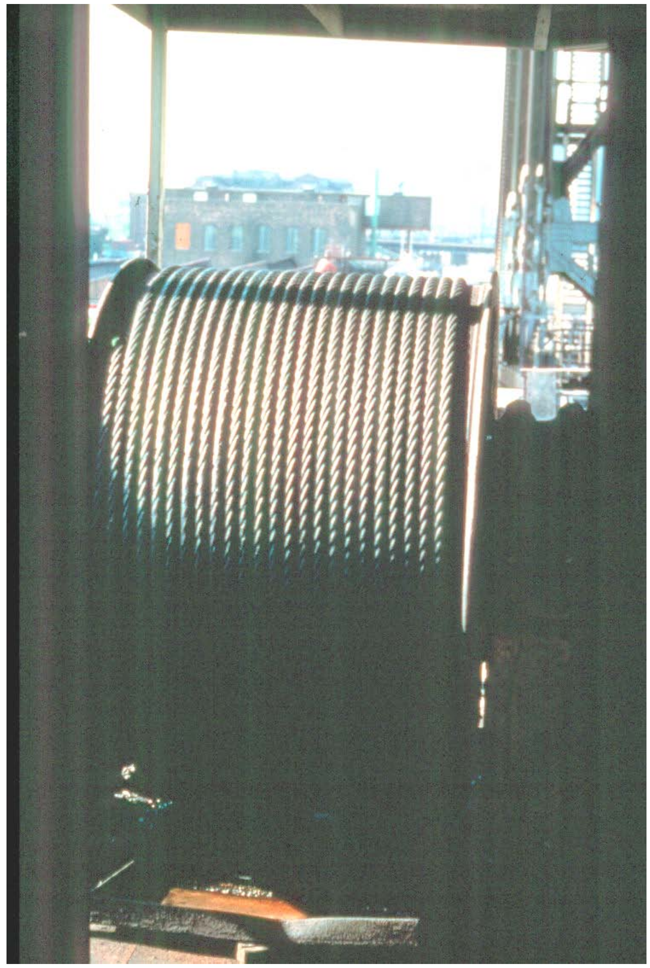

Figure 16.2.52 Drums Wind Up the Up-Haul (Lifting) Ropes as they Simultaneously Unwind the Down-Haul Ropes

Span and Counterweight Guides

Span and counterweight guides are located between tower and span or counterweight to prevent misalignment.

Balance Chains

Balance chains are provided to compensate for the weight of counterweight rope that travels from the span side to the counterweight side of the sheaves at the top of the tower as the span is raised. Weight of chain is removed from the counterweight and is supported by the tower as rope weight is increased on the counterweight side of the sheaves on the tower.

Span Leveling Devices

Mechanical or electrical, span leveling devices compensate and adjust the movement of the two ends of the span during operation to prevent unsynchronized movement.

16.2.9 Overview of Common Deficiencies

Steel

Common deficiencies that can occur to steel members of movable bridges include:

- Corrosion
- Fatigue cracking
- Overloads
- Collision damage
- Heat damage
- Paint failures

See to Topics 6.3.4 – 6.3.7 for a detailed presentation of the properties of steel, types and causes of steel deficiencies, and the examination of steel. Refer to Topic 6.4 for Fatigue and Fracture in Steel Bridges.

Concrete

Common deficiencies that occur to concrete members of movable bridges include:

- Cracking (structural, flexure, shear, crack size, nonstructural, crack orientation)
- Scaling
- Delamination
- Spalling
- Chloride contamination
- Freeze-thaw
- Efflorescence
- Alkali silica reactivity (ASR)
- Ettringite formation
- Honeycombs
- Pop-outs
- Wear
- Collision damage
- Abrasion
- Overload damage
- Internal steel corrosion
- Loss of prestress
- Carbonation

Refer to Topics 6.2.3 – 6.3.8 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies, and the examination of concrete.

16.2.10 Inspection Locations and Methods - Safety

Movable Bridge Inspector Safety

It is imperative that a movable bridge inspectors coordinate their work with the Bridge Operator and emphasize the need for advance warning of a bridge opening. The Bridge Operator cannot operate the bridge until being notified by all inspectors that they are ready for an opening. There are many ways that this can be accomplished, such as placing a warning note on the control console or opening the circuit breakers and locking the compartment to the equipment that they will be inspecting.

Inspection Considerations

Important considerations for a movable bridge inspector include observing and making comments in the inspection report on the following safety considerations.

Public Safety

Public safety considerations include good visibility of roadway and sidewalk for the Bridge Operator (see Figure 16.2.53), adequate time delay on traffic signals for driver reaction and before lowering gates, all "gates down" before raising bridge (bypass available if traffic signals are on), the bridge is closed before gates can be raised (bypass available if locks are driven), and traffic signals do not turn off until all gates are fully raised (bypass available).

Observe the location of the bridge opening in relation to the gates, traffic lights and bells, and determine whether approaching motorists can easily see them. Check their operation and physical condition to determine if they are functioning and well maintained. Recommend replacement when conditions warrant.

Unprotected approaches, such as both ends of a swing bridge and vertical lift bridge and the open end of a single-leaf bascule bridge, preferably have positive resistance barriers across the roadway, with flashing red lights as provided on the gate arms (see Figure 16.2.54). High-speed roadways and curved approaches to a movable bridge preferably have advanced warning lights (flashing yellow).

Figure 16.2.53 Operator's House with Clear View of Traffic Signals and Lane Gates

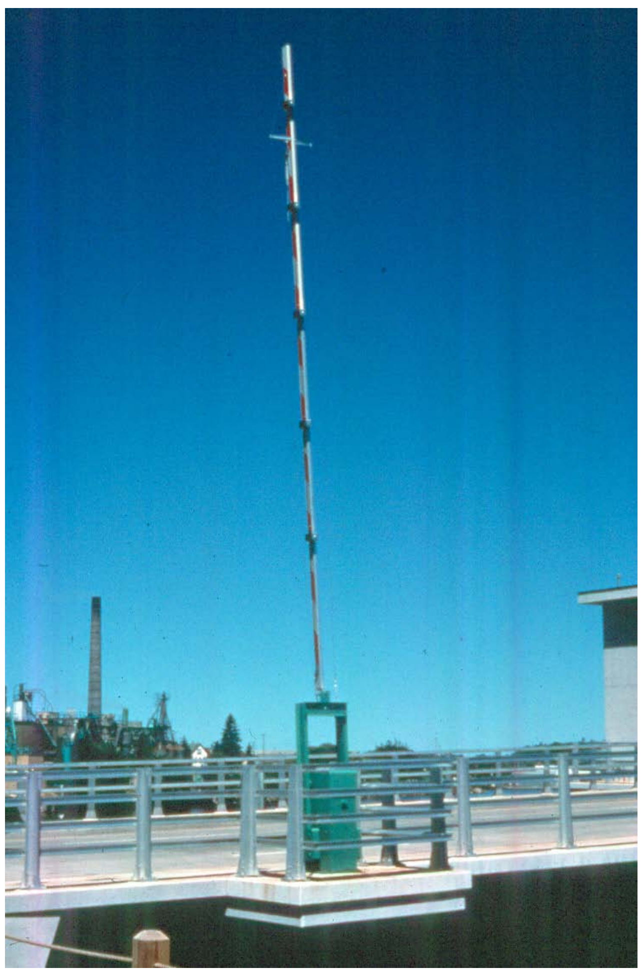

Figure 16.2.54 Traffic Control Gate

Navigational Safety

Navigational safety considerations include compliance with minimum channel width with any restriction on vertical clearance when span is open for navigation. Minimum underclearance designated on the permit drawing are to be provided. Inspect underclearance gauges for closed bridges for accuracy, visibility, and legibility.

See that all navigation lights have a relay for backup light, and red span lights do not change to green until both leaves are fully open (see Figure 16.2.55). Check navigation lights for broken lenses, deteriorated insulation of wiring and cable, and dry and clean interior, as these lights are very important to navigational safety.

Check that the marine radio communication equipment is functional (see Figure 16.2.56). Verify that the Operator can automatically sound the emergency signal to navigation vessels if bridge cannot be opened.

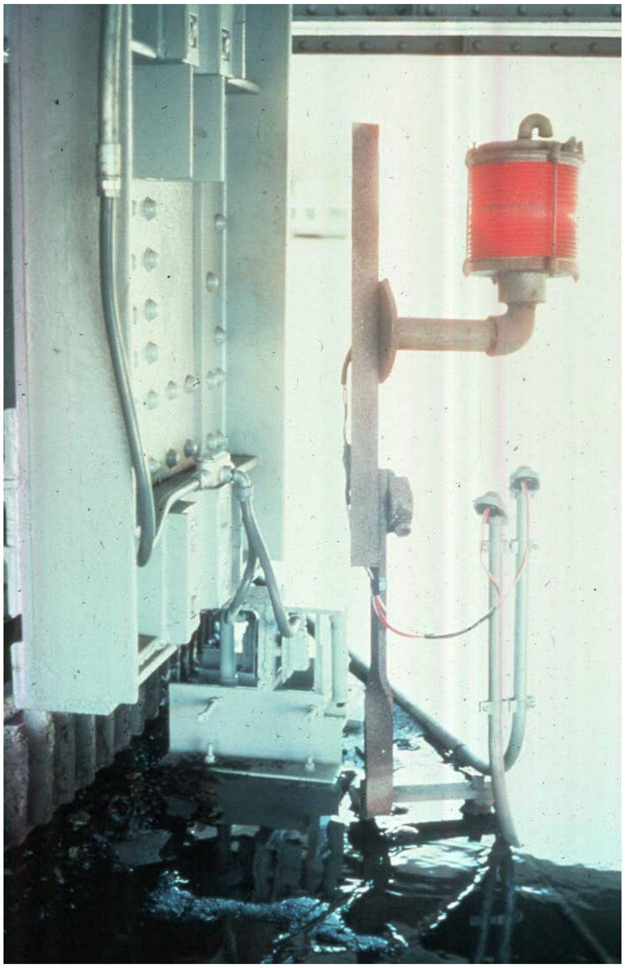

Figure 16.2.55 Navigational Light

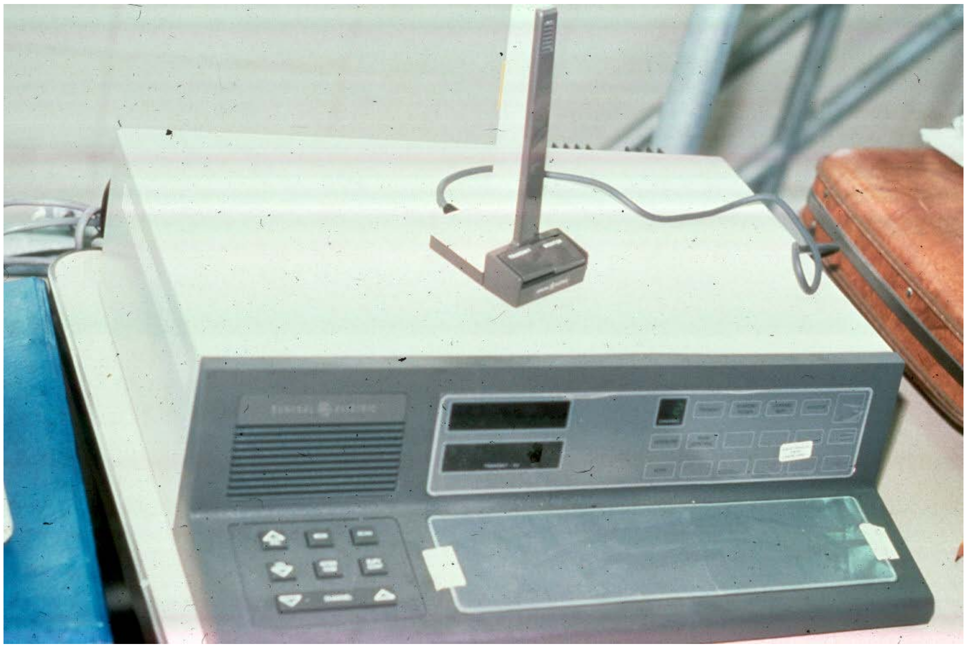

Figure 16.2.56 Marine Two-Way Radio Console

Structure Safety

Structure safety considerations include the structural ability to carry the anticipated loads. Pressure relief valves on hydraulic power units are used to limit hydraulic forces applied to machinery and structure. Horsepower applied to machinery and structure are to be kept within design limits by limiting speed.

Dependable Operation

Operate the movable bridge in both normal and emergency modes to check all interrelated interlocks and to verify every component is operating correctly.

16.2.11 Inspection Locations and Methods of Movable Bridge Opening and Closing Sequences

Movable bridges are considered to be complex according to the NBIS regulations. The NBIS requires identification of specialized inspection methods, and additional inspector training and experience required to inspect these complex bridges. The bridges are then to be inspected according to these methods.

Interlocking for Normal Operation

During normal operation, verify that each interlock functions properly and can be bypassed (when provided). Verify the controls for the traffic signals, traffic gates, center or rear locks, emergency brakes, and the bridge operation are interlocked so that they can only be operated in the following sequences.

Opening Sequence

The bridge opening sequence:

1. Activate traffic signals.
2. Lower oncoming gates and, when traffic has cleared, lower off-going gates. “All gates down” interlocked for withdrawing locks (bypass provided).
3. Press "raise" button if automatic operation is provided or, if manual operation is provided, proceed as follows:

a. Withdraw locks – "Locks Withdrawn." Interlocked for bridge operation (no bypass).

b. Release emergency brakes - no interlock provided. Warning buzzer sounds if brakes are not released when power is applied to motors to move bridge.

c. Accelerate leaves to full speed.

d. When advanced to nearly open position, decelerate leaves to slow speed and stop at nearly open position.

e. At nearly open position, with reduced power, lower leaves to stop at fully open position.

f. Set emergency brakes.

Closing Sequence

The bridge closing sequence:

1. Press "lower" button if automatic operation is provided or, if manual operation is provided, proceed as follows:

   a. Release emergency brakes.

   b. Accelerate leaves to full speed.

   c. For all types of bridges with lock bars:

      (1) At advanced nearly closed position, decelerate leaves to slow speed. Leaves stop at nearly closed position by action of the bridge limit switch.

      (2) At nearly closed position with reduced power, lower leaves to stop at fully closed position.

      (3) With machinery wound up (bascule bridges and counterweight heavy vertical lift bridges) or when span is fully closed (swing bridges and span heavy vertical lift bridges), set the brakes and drive lock bars.

   d. For rolling lift bridges having jaw and diagram shear locks with no moving parts:

      (1) At advanced nearly closed position, decelerate to slow speed. The jaw leaf stops at the "locking position" (within the "window" to receive the diaphragms) by action of the bridge limit switch.

      (2) At advance nearly closed position, decelerate to slow speed. The diaphragm leaf stops in the "clear position" (where the lower jaw will clear the diaphragm) by action of the bridge limit switch.

      (3) Depress foot switch to provide reduced power from this point until both leaves are closed.

      (4) Lower the diaphragm leaf to make "soft" contact with lower jaw.

      (5) Close both leaves together with diaphragm castings against lower jaws.

      (6) When leaves are fully closed, drive the rear locks. "Fully closed" interlock provided for rear lock operation (no

bypass).

(7) Set emergency brakes with reduced power applied to motors to hold machinery wound up.

2. Deactivate automatic traffic control, or manually raise gates:

a. All gates raise, off-going gates start up before oncoming gates raise.

b. Warning signals and red lights do not turn off until all gates are raised, even if the power switch is turned “off” (bypass is provided), after which the green traffic lights are turned “on”.

Bypass Note: All bypass switches have handles that are spring returned to “off”. When the switch is turned to bypass momentarily, a holding relay holds the bypass activated until power is removed from the controls or the switch is turned to cancel bypass. Verify these circuits are provided in order to prevent inadvertent use of any bypass. Until a malfunction is corrected, the operator is required to initiate the use of any bypass switch that is needed every time the bridge is operated.

16.2.12 Inspection Locations and Methods for the Control House

Inspection of the control house is necessary to assure the safety of a movable bridge. The operator is responsible for public and navigational safety during operation and, together with maintenance personnel, is usually the most familiar with any known structural or operational issues. Operational and maintenance log books are to be kept in the control house for reference. The resources within the control house can therefore provide a great deal of general information, through the knowledge of its personnel and the records stored there. The position of the control house provides the best general view of the bridge itself.

Consult with the bridge operators to ascertain whether there are any changes from the normal operation of the bridge. Note whether all Coast Guard, Corps of Engineers, and local instructional bulletins are posted. Check for obvious hazardous operating conditions involving the safety of the operator and maintenance personnel.

Note where the control panel is located in relation to roadway and waterway, and also whether the bridge operator has a good view of approaching boats, vehicles, and pedestrians (see Figure 16.2.57). Check operation of all closed circuit TV equipment, and evaluate its position for safe operation. If controls are in more than one location, note description of the other locations and include their condition as well as the information about the control house. Note whether alternate warning devices such as bullhorns, lanterns, flasher lights, or flags are available.

Note whether the structure shows cracks, and determine whether it is windproof and insulated. Check for any accumulations of debris, which may be readily combustible. Check controllers while bridge is opening and closing. Look for excess play and for sparking during operation. Note whether the submarine cables are kinked, hooked, or deteriorated, especially at the exposed area above or below the water. In tidal areas, check for marine and plant growth. Note if the ends of the cable have been protected from moisture.

Figure 16.2.57 Control Panel

16.2.13 Inspection Locations and Methods for Structural Members

Deficiencies

During the inspection of any type of movable structures, be sure to note any deficiencies that are detrimental to all steel and concrete structures. Most of the bridge structure deficiencies are listed in Chapter 6: Materials, as potential problems apply to movable spans also.

Fatigue

Fatigue can be a problem with movable bridges due to the reversal or the fluctuation of stresses as the spans open and close (see Figure 16.2.58). Carefully inspect any member or connection subject to such stress variations for signs of fatigue.

Figure 16.2.58 Stress Reversals in Members

Counterweights and Attachments

Inspect the counterweights to determine if they are sound and are properly affixed to the structure. Also check temporary supports for the counterweights that are to be used during bridge repair and determine their availability in the event such an occasion arises. Determine whether the counterweight pockets are properly drained. On vertical lift bridges, be sure that the sheaves and their supports are well drained. Examine every portion of the bridge where water can collect. All pockets that are exposed to rain and snow are to have a removable cover. Check for debris, birds, animals, and insect nests in the counterweight pockets.

Where steel members pass through or are embedded in the concrete, check for any corrosion of the steel member and for rust stains on the concrete. Look for cracks and spalls in the concrete.

Where lift span counterweight ropes are balanced by chains (or other means), make sure the links hang freely, and check these devices along with slides, housings, and storage devices for deficiencies and for adequacy of lubrication, where applicable.

Determine whether the bridge is balanced and whether extra balance blocks are available. A variation in the power demands on the motor, according to the span's position, is an indication of an unbalanced leaf or span. If the controls provide a “drift” position, use this to test the balance. Several coats of paint can increase the structure dead load. Otherwise, the counterweights will eventually be inadequate due to excess paint dead load.

Piers

Take notice of any rocking of the piers when the leaf is lifted. This is an indicator of a serious deficiency or critical finding and is to be reported at once. Survey the spans including towers to check both horizontal and vertical displacements. This will help to identify any foundation movements that have occurred.

Check the braces, bearings, and all housings for cracks, especially where stress risers would tend to occur. Inspect the concrete for cracks in areas where machinery bearing plates or braces are attached (see Figure 16.2.59). Note the tightness of bolts and the tightness of other fastening devices used.

Check the pier protection system (see Figure 16.2.60).

Figure 16.2.59 Concrete Bearing Areas

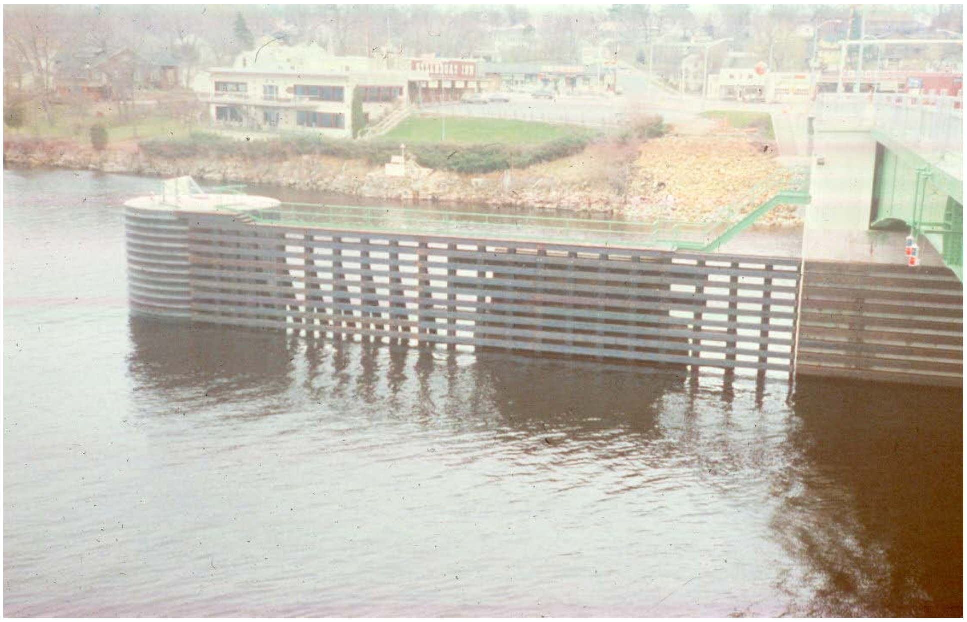

Figure 16.2.60 Pier Protection Systems – Dolphins and Fenders

Steel Grid Decks

Verify that structural welds are sound and the grid decks have adequate skid resistance. Check the roadway surface for evenness of grade and for adequate clearance at the joints where the movable span meets the fixed span. For more information on steel grid decks, see Topic 7.4.

Concrete Decks

A solid concrete deck is used over the pier areas (pivot or bascule pier) to keep water and debris from falling through onto the piers and mechanical devices. Since the machinery room is usually under the concrete deck, check the ceiling for leaks or areas that allow debris and rust to fall on the machinery. For more information of concrete decks, see Topic 7.2.

Other Structural Considerations

Other structural considerations include:

- Examine the live load bearings and wedges located under the trusses or girders at the pivot pier for proper fit alignment and amount of lift.
- Inspect the fully open bumper blocks and the attaching bolts for cracks in the concrete bases.
- Examine the counterweight pit for water. Check the condition of the sump pump, the concrete for cracks, and the entire area for debris.
- See if the shear locks are worn. Measure the exterior dimensions of the lock bars or diaphragm casting and the interior dimensions of sockets or space between jaws to determine the amount of clearance (wear). Report excessive movement and investigate further.
- On swing bridges, check the wedges and the outer bearings at the rest piers for alignment and amount of lift. This can be recognized by excessive vibration of span or uplift when load comes upon the other span.
- On double-leafed bascule bridges, measure the differential vertical movement at the joint between the two leaves under heavy loads. On other types, check for this type of movement at deck joints (breaks in floor) between movable and fixed portions of the structure. This can indicate excessive wear on lock bars or shear lock members.
- Inspect the joint between the two leaves on double-leaf bascule bridges, or the joints between fixed and movable portions of the structure for adequate longitudinal clearance for change in temperature (thermal expansion).
- On bascule bridges, see if the front live load bearings fit snugly. Also observe the fit of tail locks at rear arm and of supports at outer end of single-leaf bridges.
- On rolling lift bascule bridges, check the segmental and track castings and their respective supporting track girders (if used) for wear on sides of track teeth due to movement of sockets on segmental castings. Compare all wear patterns for indications of movement of the leaves. Check for cracking at the fillet of the angles forming the flanges of the segmental and track girders, cracking in the flanges opposite joints in the castings, and cracking of the concrete under the track. Inspect rack support for lateral movement when bridge is in motion.
- On multi-trunnion (Strauss) bascule bridges, check the strut connecting the counterweight trunnion to the counterweight for fatigue cracks. On several bridges, cracking has been noted in the web and lower flanges near the

gusset connection at the end nearer the counterweights. The crack would be most noticeable when the span is opened.

16.2.14 Inspection Locations and Methods for Machinery Members

Mechanical, electrical, and hydraulic equipment includes specialized areas, which are beyond the scope of this reference manual. Since operating equipment is the heart of the movable bridge, it is recommended that expert assistance be obtained when conducting an inspection of movable spans. In many cases, the owners of these movable bridges follow excellent programs of inspection, maintenance, and repair. However, there is always the possibility that some important feature may have been overlooked. Any problems noted during the inspection are reported to the owner.

Trial Openings

Conduct trial openings as necessary to insure proper operational functioning and that the movable span is properly balanced. Trial openings are specifically for inspection. During the trial openings, the safety of the inspection personnel, traveling public and boat operators is a primary concern.

Machinery Inspection Considerations

On all movable structures, the machinery is so important that considerable time is to be devoted to its inspection. The items covered and termed as machinery include all motors, brakes, gears, tracks, shafts, couplings, bearings, locks, linkages, over-speed controls, and any other integral part that transmits the necessary mechanical power to operate the movable portion of the bridge. Inspect machinery not only for its current condition, but also for operational and maintenance methods and analysis of the characteristics of operation. The items listed below and items similar to them are to be inspected and analyzed by a machinery or movable bridge specialist. Refer to FHWA-IP-77-10, *Bridge Inspector's Manual for Movable Bridges* and the *AASHTO Movable Bridge Inspection, Evaluation and Maintenance Manual, Manual for Bridge Evaluation* for further information on inspecting these items. The FHWA-IP-77-10 manual is published by the Federal Highway Administration (FHWA), but is currently out of print.

Operation and General System Condition

Observe the general condition of the machinery as a whole, and its performance during operation. Check for smoothness of operation, and note any abnormal performance of components. Note any noise or vibration and the source determined. Document any unsafe or detrimental methods followed by the operator to prevent injury to the public or to personnel, or deficiencies to the equipment. Also note the condition of the paint system.

Maintenance Methods

Perform an evaluation of maintenance methods in light of design details for the equipment. Check application methods and frequency of lubrication in the maintenance logbook, if available. Note general appearance of existing applied lubricant.

Open Gearing

Check open gearing for tooth condition and alignment including over- and under-engagement. Verify that the pitch lines match. Note excessive or abnormal wear. Inspect the teeth, spokes, and hub for cracks. Observe and note the general appearance of the applied lubricants on open gearing. If the lubricant has been contaminated, especially with sand or other gritty material, remove the old lubricant and have new lubricant applied. If there is a way to prevent future contamination, recommend this appropriate procedure as part of the inspector's comments in the report. Check the teeth of all gears for wear, cleanliness, corrosion, and for proper alignment.

Speed Reducers Including Differentials

Examine the exterior of the housing and mountings for cracks and deficiencies (see Figure 16.2.61 and 16.2.62). Check bolts for tightness and note any corrosion. Inspect the interior of the housing for condensation and corrosion. Check the condition of gears. Watch for abnormal shaft movement during operation, indicating bearing and seal wear. Periodically check oil levels and condition of lubricant. Check that circulating pumps and lubricating lines are properly operating. Any abnormal noise is to be documented. Leaking oil may indicate the presence of a crack.

Figure 16.2.61 Cracked Speed Reducer Housing

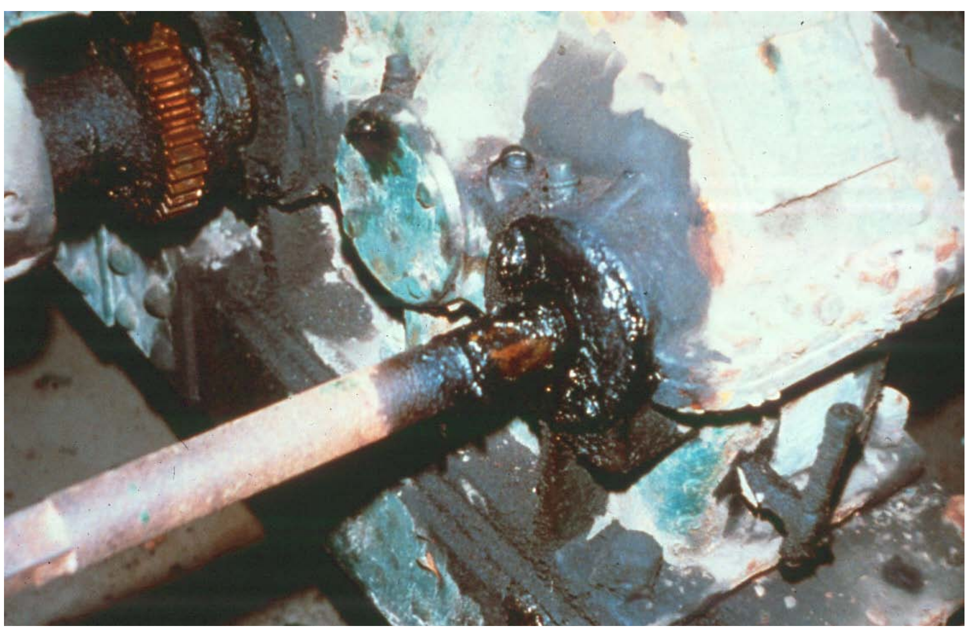

Figure 16.2.62 Leaking Speed Reducer

Shafts and Couplings

Examine shafts damage, twisting, and strain. Cracks, if suspected, may be detected using non-destructive evaluations (NDE) such as magnetic particle or dye penetrant (see Figure 16.2.63). Various advanced inspection methods for steel members are presented in Topic 15.3. Cracks in mechanical components may be determined to be a critical finding. Note misalignment with other parts of the machinery system. Document cracks in shafts and record the exact location. Examine other shafts in the same locations as they were probably made from the same material and fabricated to the same details. They have also been exposed to the same magnitude and frequency of loading. Check coupling hubs, housings, and bolts for condition. Inspect seals and gaskets for leaks. Internal inspection of couplings is warranted if problems are suspected and can be used to determine tooth wear in gear couplings.

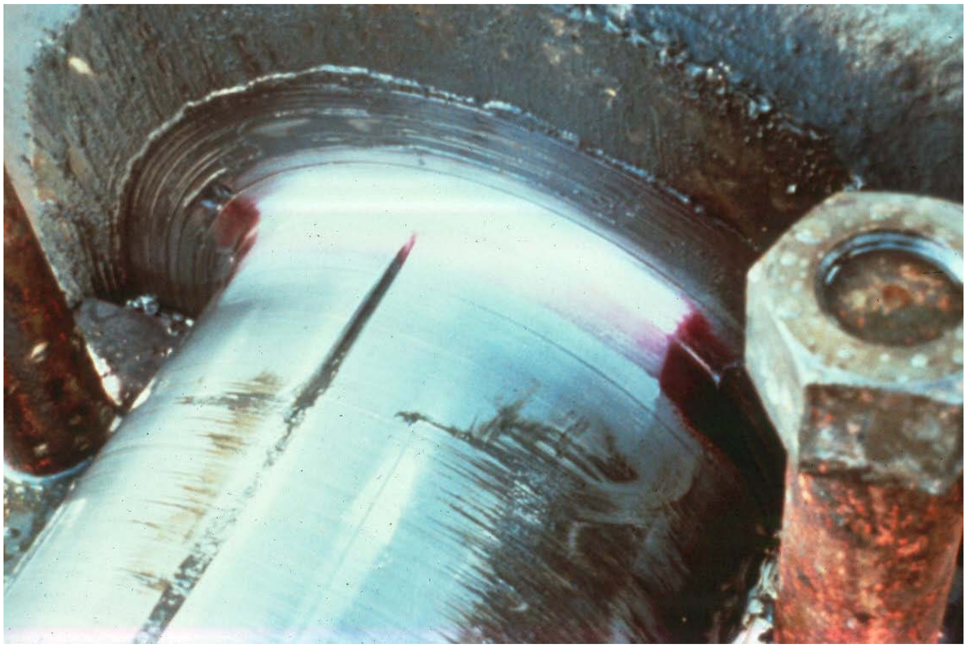

Figure 16.2.63 Hairline Crack Revealed on Shaft from Dye Penetrant Test

Bearings

Examine bearing housings, pedestals, and supports for external condition, noting any cracks. Check bolts in housings and those used for anchors for tightness, damage, and corrosion, noting apparent lubrication characteristics. Grinding noises can be caused the lack of lubricant (see Figure 16.2.64). In sleeve bearings, inspect the bushings for damage and excessive wear. Note evidence of seal damage in anti-friction bearings. Investigate any unusual noise. Check the trunnion bearings for excessive wear, lateral slip, and loose bolts.

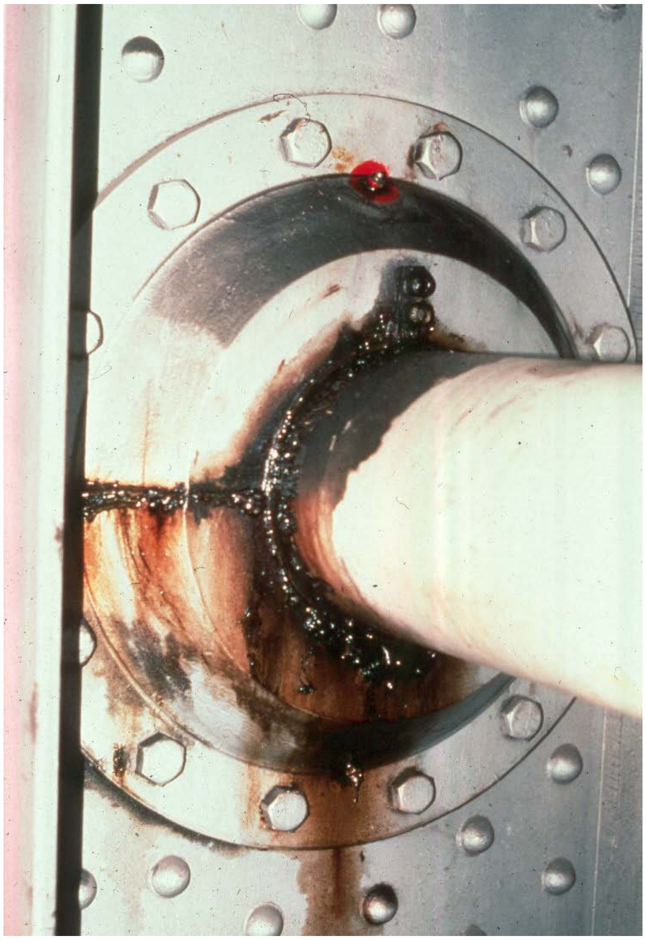

Figure 16.2.64 Leaking Bearing

Brakes

Inspect all braking devices for proper setting of braking torque and for complete release of the brakes when actuated. On shoe brakes, check drums and shoes for wear, damage, and corrosion, for misalignment of shoes with drums, and for clearance when released. Determine if worn linings need replaced. Check for proper actuation without leakage by actuators. Verify that linkages and hand releases are free but not sloppy. On enclosed hydraulic disc brakes, make certain there is proper actuation without leakage at connections or seals. Check the brakes, limit switches, and stops (cylinders and others) for excessive wear and slip movement. Note whether the cushion cylinder ram sticks or inserts too easily. Inspect the brake limit switches for proper setting. Observe the surface of the brake drum for indications of contact with the brake shoes. Check the pressure developed by each disc brake power unit to be sure the brakes are releasing. Also check the manual release on all of the brakes.

Drives - Electric Motors

Check the housing and mountings for damage, corrosion, and fastener condition. Inspect bearings for lubrication and note indications of wear (movement) and seal leakage at shaft extensions.

Drives - Hydraulic Equipment

Look for any leakage at connections and seals. Note any corrosion on the cylinder rods. Listen to motors and pumps, and note any unusual noise. Check power units to make sure all components are functioning and that pressures are properly adjusted. Sample fluid periodically and examine for contamination and wear metal. Check all main hydraulic power units for charge pressure setting and maximum pressure that can be developed by the unit. Check all filters routinely and replace as needed. Also check the level of fluid in the vertical reservoir.

Auxiliary Drives

Check emergency generators for operation and readiness, verifying that there are no oil leaks or abnormal noises. Mechanical service specialists and electrical inspectors are required for more thorough inspections. Auxiliary motors and hand operators, with their clutches and other transmission components, are to be checked for adjustment and readiness to perform when called upon.

Drives - Internal Combustion Engines

Detailed inspections of internal combustion engines are made by mechanical engine specialists. The inspection may include but is not limited to checking of the following conditions:

- If a belt drive is used, look for any wear or slippage. Note the condition of all belts and the need for replacement, if any.
- If a friction drive is used, check that all bracing and bearings are tight.
- If a liquid coupling is used, make sure that the proper quantity of fluid is used. Look for leaks.

Locks

Examine the center locks and tail locks (if used) on double-leafed bascule spans, and the end locks on single-leaf bascule bridges, swing bridges, and vertical lift bridges. Note whether there is excessive deflection at these joints or vibration on the bridge. Inspect the locks for fit and for movement of the span or leaf (or leaves). Check lubrication and for loose bolts. Verify that the lock housing and its braces have no noticeable movement or misalignment. The paint adjacent to the locks will have signs of paint loss or wear if there is movement. Check lock bars, movable posts, linkages, sockets, bushings, and supports for damage, cracks, wear, and corrosion.

Check all rear locks in the withdrawn position for clearance from the path of the moving leaf as it opens and for full engagement when the leaf is closed. Measure the gap, if any, between the lock plate and the moving leaf bearing plate. Check each rear lock hydraulic drive unit for leakage of oil and operation for correct length of movement of the lock.

On bascule bridges, see if the front live load bearings fit snugly. Also observe the fit of tail locks at the rear arm and of supports at the outer end of single-leaf bridges.

Examine actuators for operational characteristics, including leakage if hydraulic. Note both the quantity and quality of the lubricant. Check for alignment, and analyze the type of wear that is occurring. Note condition of movable operators.

Live Load Shoes and Strike Plates Inspect the fasteners and structure for deficiencies and corrosion. Note contact surface conditions. Check for alignment and movement under load.

Air Buffer Cylinders and Shock Absorbers Note indications of lack of pressure or stickiness during operation. Check piston rod alignment with strike plate. Note the condition of the rod and housing, and verify if hydraulic leakage is present. Check the air filter and function of any pressure reading or adjusting devices and the operating pressure, if possible. Verify that the air buffers have freedom of movement and development of pressure when closing. Inspect the fully open bumper blocks and the attaching bolts for cracks in the concrete bases.

Machinery Frames, Supports, and Foundations Check that there is no cracking in the steel or concrete. Note corrosion and damage. Check for deflection and movement under load. Ensure that the linkages and pin connections have the proper adjustment and are in functional condition. Check motor mounting brackets to ensure secure mounting.

Fasteners Inspect the fasteners for corrosion, loss of section, and tightness.

Wedges Check the wedges and the outer bearings at the rest piers for alignment and amount of lift. This can be recognized by excessive vibration of span or uplift when load comes upon the other span.

Examine the live load bearings and wedges located under the trusses or girders at the pivot pier for proper fit alignment and amount of lift.

Special Machinery for Swing Bridges Check center bearings for proper and adequate lubrication, oil leaks, and noise. Examine the housing for cracking, pitting, fit of joints, and note indications of span translation (irregular rotation) at racks and track. Measure for proper clearance of balance wheels above track. Verify that the tracks and balance wheels are free of wear, pitting, and cracking. Check for proper and adequate lubrication at all lubrication points.

Note balance characteristics as indicated by loads taken by balance wheels, and by drag on the rest pier rail.

Check the rim bearing for wear on tracks and rollers, particularly at rest positions where the bridge is carrying traffic. Examine the center pivots and guide rings for proper fit, and for wear, pitting, and cracking. Check for proper and adequate lubrication at all lubrication points.

Examine the center (live load) wedges located under the trusses or girders at the pivot pier for proper fit (no lifting) and alignment. Check end wedges and bearings at the rest piers for alignment and amount of lift. This can be recognized by excessive vibration of the span or uplift when live load crosses the other span. Inspect the end lift jacks, shoes, and all linkages for wear, proper bearing under load, and proper adjustment.

Note the condition of end latches, including any modification that adversely affects their functional design.

Special Machinery for Bascule Bridges

On rolling lift bascule bridges, check the segmental and track castings and their respective supporting track girders (if used) for wear on the sides of track teeth due to movement of sockets on segmental castings. Inspect the trunnion assemblies for deflection, buckling, lateral slip, and loose bolts. Examine the trunnions for any signs of corrosion, pitting, or cracking, particularly at stress risers. Laser leveling may be used during the inspection of trunnions. Check the balance of each leaf. Compare all wear patterns for indications of movement of the leaves. Check for cracking at the fillet of the angles forming the flanges of the segmental and track girders, cracking in the flanges opposite joints in the castings, and cracking of the concrete under the track. Inspect rack support for lateral movement when bridge is in motion.

Check trunnion bearings for lubrication of the full width of the bearing. Verify that extreme pressure (EP) lubrication oil of the proper grade is used.

Special Machinery for Vertical Lift Bridges

The condition of wire ropes and sockets, including wire rope lubrication, is important. Look for flattening or fraying of the strands and deficiencies between them. This is reason for replacement. Similarly, check the up-haul and down-haul ropes to see if they are winding and unwinding properly on the drums. Note any need for tension adjustments in up-haul and down-haul ropes. Determine whether ropes have freedom of movement and are running properly in sheave grooves. Look for any obstructions to prevent movement of the ropes through the pulley system, and check the supports on span drive type bridges. Check rope guides for alignment, proper fit, free movement, wear, and structural integrity of the longitudinal and transverse grooved guide castings. Inspect the grooved guide castings closely for wear in the grooves. Examine the cable hold-downs, turnbuckles, cleats, guides, clamps, splay castings, and the travel rollers and their guides.

Check that balance chains hang freely, that span leveling devices are functioning, and that span and counterweight balance closely. Observe if span becomes "out of level" during lifting operation. Inspect spring tension, brackets, braces, and connectors of power cable reels.

Check for damage, including cracking, at drums and sheaves. Note the condition and alignment of span guides.

16.2.15 Electrical Inspection Considerations

An available electrical specialist is required for the inspection of the electrical equipment. For this inspection, use current AASHTO guidance on inspection of movable bridges. Observe the functional operation of the bridge and look for abnormal performance of the equipment. Check the operational methods and safety features provided. Evaluate the maintenance methods being followed and check the frequency of services performed.

Power Supplies

Examine the normal power supply, standby power supply, and standby generator set (for emergency operation of bridge and service lighting) and note the following:

- Take megger readings on the cable insulation values, noting the weather conditions, namely temperature and humidity.
- Make sure all cable connections are properly tightened.
- Measure the voltage and the current to the motors at regular intervals during the operation of the bridge.
- Check the collector rings and windings on the generator set.
- Test starting circuitry for automatic starting and manual starting.
- See if the unit is vibrating while running under load.

If the power cable has been repaired with a splice, note the condition of the splice box seal.

If no standby power supply has been provided, determine whether a portable generator could be used. A manual transfer switch would be a convenient way of connecting it.

Motors

Examine span drive motors, lock motors, brake thrustor motors, and brake solenoids for the same items as given for power supplies.

Transformers

Check dry transformer coil housings, terminals, and insulators, including their temperature under load. Observe the frames and supports for rigidity to prevent vibration. Check the liquid filled transformer in the same way, along with checking the oil level while looking for leakage. Examine oil insulation test records.

Circuit Breakers

Check circuit breakers (e.g., air, molded case, and oil) and fuses, including the arc chute, contact surfaces, overload trip settings, insulation, and terminal connections. Examine oil insulation test records, and observe the closing and tripping operation. Record all fuse types and sizes being used.

Wires and Cables

Examine the wiring and cables for both power and control. Note whether the submarine cables are kinked, hooked, or deteriorated, especially at the exposed area above and below the water. In tidal areas, look for marine and plant growth. Note if the ends of the cable have been protected from moisture. Record the insulation value of each wire as measured by megger. Look for cracking, overheating, and deterioration of the insulation. Check for wear against surfaces and especially sharp edges. Check the adequacy of supports and that dirt and debris do not accumulate against the conduit and supports. Check terminal connections, clamps, and securing clips for tightness, corrosion, and verify that there are wire numbers on the end of each wire. The weight of the wires or cables will be carried by the clamps and not by the wire connections at the terminal strips.

Cabinets

Examine the programmable logic controller (PLC) cabinets, control consoles and stations, switchboards (see Figure 16.2.65), relay cabinets, motor control centers (MCC), and all enclosures for deficiencies, debris inside, drainage, operations of heater to prevent condensation, and their ability to protect the equipment inside. Check the operation of all traffic signals, traffic gates, traffic barriers, and navigation lights. Verify that the bridge is open to provide the clearance shown on the permit drawing before the green span light turns on. Check the traffic warning equipment and control circuits, including the advanced warning signals (if used), traffic lights/signals, gates, barriers, and the public address and communication equipment.

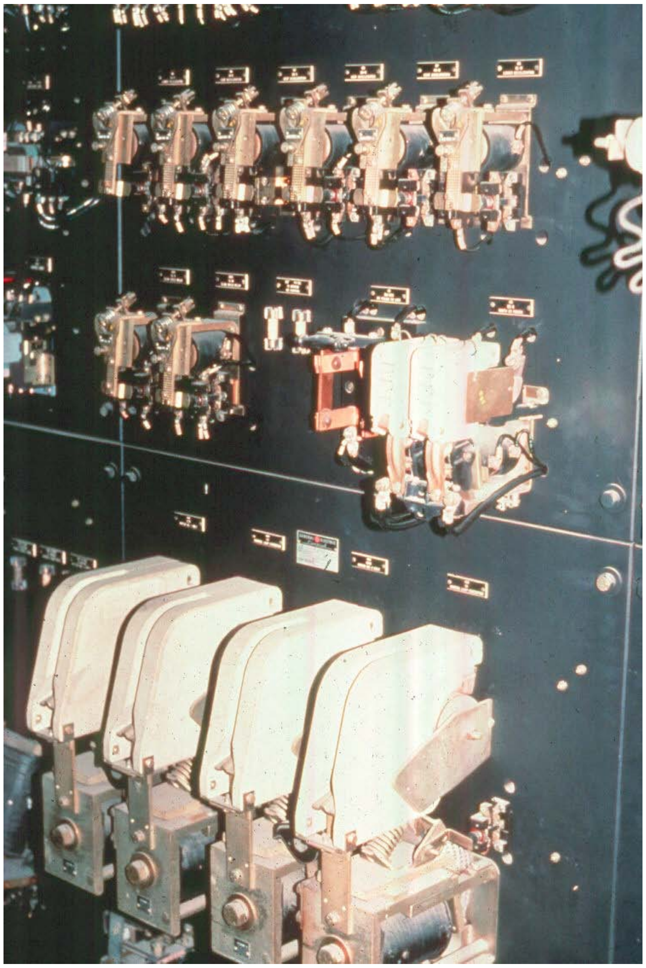

Figure 16.2.65 Open Switchboard

Conduit

See if conduit is far enough away from all surfaces to avoid debris from collecting against it. Note if it is adequately supported and pitched to drain away from junction boxes and pull boxes, so that water is not trapped within. Also, note if all conduits have covers with seals. Report deteriorated conduit so that it can be replaced with new conduit. Seal and re-coat the connectors at the ends of all PVC coated conduit after all fittings are installed.

Junction Boxes	Examine the covers on all junction boxes (JBs) for an effective seal, dry interior, functioning breather-drains, heaters having enough power to prevent condensation inside, and terminal strips all secured to the bottom of horizontal JBs or to the back of vertical JBs.
Meters	Observe if all voltmeters, ammeters, and watt meters are freely fluctuating with a change in load. Check that all switches and meters are operable.
Control Starters and Contactors/Relays	Check the operation of this equipment under load, and watch for arcing between contacts, snap action of contacts, deterioration of any surfaces, and drainage of any moisture. Look for signs of corrosion and overheating.
Limit Switches	Set all limit switches so they do not operate until they are intended to stop the equipment or complete an interlock. Verify that the interior is clean and dry, with all springs active.
Selsyn Transmitters and Receivers	Check for power to the field and signal being sent from the transmitter to the receiver. Observe the receiver tracking the rotation of the bridge as it operates. Observe the mechanical coupling between the driving shaft and the transmitter, checking for damage and misalignment.
Service Light and Outlet	Check to see if power is going to each light and outlet. Note if there is a shield or bar for protecting each bulb and socket. It is desirable to have service lights available when power is removed from all movable bridge controls and equipment.

16.2.16 Hydraulic Inspection Considerations

A hydraulic power specialist is required for the inspection of the hydraulic equipment (see Figure 16.2.66). Observe the functional operation of the bridge and look for abnormal performance of the equipment. Check the safety features provided and evaluate the maintenance methods being followed, checking the frequency of services performed. Due to the inter-related function of components, the requirements for fluid cleanliness, and the need for personnel safety, do not open the reservoir or hydraulic lines. In addition, do not shut off or adjust any component or part of the power circuit without complete understanding of their function and knowledge of the effect such action will have upon the system. Items which are checked during a hydraulic inspection include the following:

- Note leakage anywhere in the system. Significant leakage is immediately brought to the attention of the bridge authority.
- Check for corrosion of reservoir, piping, and connections.
- Inspect sight gauges for proper fluid level in reservoir. Note gauges with low fluid levels or gauges which cannot be read.
- Note unusual noises from any part of the system.
- Check filter indicators to make sure filters are clean.
- Collect a sample of the hydraulic fluid for analysis by a testing laboratory during periodic inspections.

Figure 16.2.66 Hydraulic Power Specialists

16.2.17 Recordkeeping and Documentation

General

The owner of a movable bridge keeps a complete file available for the engineer who is responsible for the operation and maintenance of the bridge. See Topic 4.4 for general record keeping and documentation. The file includes (if applicable), but not be limited to, the following:

- Copy of the latest approved permit drawing
- Complete set of design plans and special provisions
- "As-built" shop plans for the structural steel, architectural, mechanical, electrical, and hydraulic
- Machinery Maintenance Manual
- Electrical Maintenance Manual
- Hydraulic Maintenance Manual
- Copy of maintenance methods being followed
- Copy of the latest Operator's Instruction being followed
- Copies of all inspection reports
- Copy of all maintenance reports
- Copy of all repair plans
- Up-to-date running log on all spare parts that are available, on order, or out of stock

Review inspection and maintenance reports with preventative maintenance measures in mind. An example would be the "megger" readings on wiring insulation; especially those taken on damp rainy days when moisture could influence (reduce) the values. An acceptable minimum reading is usually 1 megaohm. If the value on a wire is decreasing on progressive reports, preventative maintenance may save a "short" that could burn out equipment and put the bridge out of operation.

Inspection and Maintenance Data

Examples of inspection and maintenance records are shown in Figures 16.2.67 through 16.2.73.

South Tower Differential Assembly GEARS - General 1.

Gear	General Condition	Lubrication	Keys	Alignment: Center Distance	Axial	Parallel
Pinion P5	Very Good. Tooth profiles show normal wear	Very Good	Good	Good. Pitch Lines Tangent	Good	Good
Gear I5	Very Good. Tooth profiles normal.	Very Good	Good	No Pitch Line on G5. Looks good. Measured backlash.		
Gear G5	Very Good. Tooth profiles normal	Very Good	Good			
Pinion P4	Very Good. Tooth profiles normal.	Very Good	Integral with shaft	No pitch line on P4. Center distance looks. Looks good. Measured backlash.	Good	Good
Gear G4	Very Good. Tooth profiles normal.	Very Good	Not keyed to shaft. Clutch locks G4 to shaft.			
Bevel Gears BG3 (2)	Very Good. Tooth profiles normal.	Very Good	Integral with sleeves.	Good. Pitch Lines 1/16" to 1/8" apart.	Good	Good
Bevel Pinions BP3 (2)	Very Good. Tooth profiles normal.	Very Good	Integral with shafts.			

Figure 16.2.67 Example of Notes on Operating Machinery (Gears-General)

South Tower Differential Assembly GEARS - Teeth 2.

Gear	Chordal Thickness		Backlash		Condition of Teeth					
					Normal	Pitting	Rolling-Peening	Abnormal		
	Original	Measured	Original	Measured				Scoring	Interference	Rust & Corr.
Pinion P5	.625"	Did not measure	.011" min to .020" max.	Did not measure. Pitch lines indicate good backlash.	✓					
Gear I5	.625"		.011" min		✓					
Gear G5	.625"		to .020" max.	.0135" Good.	✓					
Pinion P4	.625"		.011" min to	.020" Good	✓					
Gear G4	.625"		.020" max		✓					
Bevel Gears BG3 (2)	.875" at large end of teeth		.015" min to	Did not	✓					
Bevel Pinions BP3 (2)	.875" at large end of teeth		.029" max	measure. Pitch lines indicate good backlash.	✓					

Figure 16.2.68 Example of Notes on Operating Machinery (Gears-Teeth)

South Tower Differential Assembly BEARINGS 3

Bearing	General Condition	Clearance		Bolts	Lubrication
		Original	Measured		
West end Emer. Motor Shaft	Good. Fairly clean, paint good. Bearing has 45° angle lube fitting w/dust cap.	.0025" min. to .0073" max.	.006" Good	Good. Nuts tight. Clean, paint good.	Good.
East end Emer. Motor Shaft		.0025" min. to .0073" max.	.006" Good		
West end Intermediate Shaft		.0025" min. to .0073" max.	.007" Good		
East end Intermediate Shaft		.0025" min. to .0073" max.	.005" Good		
West end Normal Motor Shaft		.0025" min. to .0073" max.	.007" Good		
East end Normal Motor shaft		.0025" min to .0073" max	.009" Fair		

Figure 16.2.69 Example of Notes on Operating Machinery (Bearings)

South Tower Differential Assembly MECHANICAL COMPONENTS 4.

Item	General Condition
Housing Cover	Very good condition. Cover has four hinged maintenance panels, secured with studs and wingnuts. Cover bolted to lower supports with 20 bolts.
Normal (Main) Drive Clutch Cone	Very good condition. No slippage during span operation, starting or stopping. Clutch cone is inside differential assembly and impossible to inspect without disassembly of differential.
Emergency Drive Clutch Cone Assembly	Very good condition. Design plans show cone type clutch. Actually have jaw type clutch.
Differential Clutch Operating Linkage	Very good condition. Well lubricated. Linkage operates smooth and quiet.
Emergency Drive Clutch Operating Linkage	Very good condition. Well lubricated. Linkage operates smooth and quiet.
Gear Motor for operation of Differential Clutch	Good condition. Operates smoothly. Operated with hand crank, turned fairly easy. GE AC Gearmotor, Model KY3AC2345, Motor 1800 rpm, 1/8 HP, 250:1 ratio
Support for above Gear Motor	Good. Some debris and oil on support.
Gear Motor for operation of Emer. Drive Clutch	Good. Operates smoothly. Turned easily with hand crank. Same gearmotor as at differential clutch
Support for above Gear Motor	Good. Some debris and oil on support.
Housing Support	Good condition. Some debris and oil on support and floor. Paint good. 2 lights attached to supports inside

Figure 16.2.70 Example of Notes on Operating Machinery (Mechanical Components)

Electrical Equipment 125HP, 600RPM, 3φ, 60H

Motor A (Normal-Traction) Tower South - Side W

General Items	General Condition
Stiffness of Supports	Good
Connection to "	Bolts tight.
Condition of Frame	Dirty & Dusty Inside & Out
Inspection Covers	Wire Mesh, 2 on Top (2 on Bott. missing)
Gaskets on "	None
Bolts on "	Tight
Ventilation	Open Ends
Operation-Noise	Normal
" -Vibration	Minimal
" -Bearings	Normal wear
Lubrication	Needs normal application
Oil-Dirt Build-Up	None (Except at couplings)
Insulation	See Megger test
Cable Connections	Good

Wound Rotor Motors	Wire No.	Raising Span Amps.	Lowering Span Amps.	
Motor Current - φ A	T1A	10-25-80 122	91	
B	T3A	124	93	
C	T2A	124	92	
Motor Voltage - A-B				460V
A-C				
B-C				

Rings - Surface	Normal wear
" - Arcing	None Visible
Brushes - Contact	Good
" - Spring Pressure	Good, Springs Rusty
" - Condition	Good, 2¼" length
Wiring - Connection	Tight, Bolts Rusty
" - Insulation	Good

Rotor Current 3φ A	M1A	10-25-80 50	31	
B	M3A	48	32	
C	M2A	50	32	

Figure 16.2.71 Example of Notes on Electrical Equipment (Motors)

Megger Insulation Test Temp 60's Weather Dry.

Rotating Cam- Normal Height Limit Switch.
contacts shown for Bridge Closed. Tower South Side W

Bottom Connection			Gear	Top Connection		
Remarks	500V MΩ to Ground	Wire No. Taged U.N.	Drive End North Contacts	Wire No.	500V MΩ to Ground	Remarks
	0.2	1084	1	1081	10.	
	0.2	1085	2			
	16.	No Tag 1083	3	1003	8.	
	18.	1105	4	1010.	0.2	
	20.	No Tag 1110	5			
	18.	1117	6			
	18.	1125	7			
	0.2	2051	8	2022	0.2	
	0.2	2052	9			
Spare		No Wires	10			

Remarks: Cover has probably been left OFF for a period of time. No gaskets, clips on some switches not hooked. Connection screws inside all rusty on the bottom. Springs rusty but still springy. Contacts are clean with fair contact alignment.

Figure 16.2.72 Example of Notes on Electrical Equipment (Limit Switch)

Megger Insulation Test of the Submarine Cables					Temp 50°s Weather Dry		
Equipment Being Controlled	Wire	Emergency Cables			Normal Cables		
	No. on Plans	No. in Cable	500V M-Ω	Remarks	No. in Cable	500V M-Ω	Remarks
North Tower Elev.	261	1	6		2	500	
	261	3	6		4	500	
	263	5	1.5		6	<.2	>20k-Ω
	263	7	1.5		8	.1	
	262	9	.9		10	.1	
	262	11	.9		12	.1	
Service Brake C	447	13	2.0		14	1000	
	446	15	40.		16	1000	
	448	17	15.		18	1000	
Service Brake D	467	19	2.		20	1000	
	466	21	25.		22	1000	
	468	23	5.		24	1000	
Drag Brake L	519	25	20.		26	1000	
	516	27	35.		28	1000	
	520	29	5.		30	1000	
Drag Brake M 516	529	31	4.		32	1000	
	526	33	5.		34	1000	
	535	35	1.		36	1000	
North Lock Motor	617	37	0.8		38	1000	
	616	39	10.		40	1000	
	618	41	0.2		42	1000	
North Barrier Gate Motor	647	43	12.		44	1000	
	646	45	.7		46	1000	
	648	47	90		48	∞	
N.W. Traffic Gate Motor	687	49	.2		50	1000	
	686	51	35.		52	∞	
	688	52	100.		54	∞	
N.E. Traffic Gate Motor	697	55	9.		56	1000	
	696	57	6.		58	1000	
	698	59	3.		60	1000	

Figure 16.2.73 Example of Notes on Electrical Equipment (Megger Insulation Test of the Submarine Cables)

16.2.18 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of movable bridges. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using the NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBI component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating.

Element Level Condition State Assessment

In an element level condition state assessment of a movable bridge, possible AASHTO National Bridge Element (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	**Description**
Superstructure	
	Box Girder
102	Steel Closed Web/Box Girder
	Floor System
107	Steel Open Girder/Beam
113	Steel Stringer (Stringer-Floorbeam System)
152	Steel Floorbeam (Stringer-Floorbeam System)
	Steel Truss
120	Steel Truss
162	Steel Gusset Plate
	Steel Arch
141	Steel Arch
	Cable
147	Steel Main Cable (not embedded in concrete)
148	Steel Secondary Cable (not embedded in concrete)

BME No.	**Description**
Wearing Surfaces and Protection Systems	
515	Steel Protective Coating

The unit quantity for the superstructure elements is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit quantity for gusset plates is each, with each gusset plate element placed in one of the four available condition states depending on the extent and severity of the deficiency. The unit quantity for protective coating is square feet, and the total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

For mechanical, electrical, and hydraulic movable bridge members, individual bridge owners may choose to create their own Agency Developed Elements (ADEs).

The following Defect Flags are applicable in the evaluation of movable bridges:

Defect Flag No.	**Description**
356	Steel Cracking/Fatigue
357	Pack Rust
362	Superstructure Traffic Impact (load capacity)
363	Steel Section Loss
364	Steel out-of-plane (compression members)

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

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Chapter 16
Complex Bridges

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Topic 16.3 Floating Bridges

16.3.1 Introduction

Although uncommon, some states have floating bridges that are not supported by a substructure. Instead, they are supported by, or float on the water. The bridge elevation will change as the water level fluctuates (see Figure 16.3.1).

Floating bridges are cost-effective solutions for crossing large bodies of very deep water with a very soft bottom where conventional piers are impractical. For a site with a 100- to 200-foot deep water and a very soft bottom extending another 100 to 200 feet, a floating bridge is estimated to cost three to five time less than a conventional multi-span fixed bridge or a tunnel.

Floating bridges perform well in areas subjected to high winds, moderate currents and moderate waves. They also have low environmental impact and perform well in seismic events.

Washington State is known for its floating bridges with four of the longest and heaviest floating bridges. They are the SR 520 Evergreen Point Bridge, the I-90 Lacey V. Murrow Bridge, the I-90 Homer M. Hadley Bridge, and the SR 104 Hood Canal Bridge.

Figure 16.3.1 Floating Bridge, SR 520 Evergreen Point Bridge, Seattle, WA During Stormy Weather

16.3.2 Design Characteristics

Floating bridges take advantage of the natural law of buoyancy of water to support the loads. This is achieved through the use of giant pontoons secured into place by an anchoring system. Conventional piers and foundations are not used.

Since a floating bridge "sits" on the water, the bridge itself creates an obstacle to vessels attempting to cross the waterway. For this reason, many floating bridges employ a movable bridge section for vessels to pass through, or an elevated span for vessels to pass under (see Figures 16.3.2 and 16.3.3).

Figure 16.3.2 Movable Bridge Section of Evergreen Point Bridge, Seattle, WA

Figure 16.3.3 Elevated Section of Evergreen Point Bridge, Seattle, WA

Pontoons

Floating bridges may be constructed of wood (see Figure 16.3.4), concrete, steel, or a combination of materials depending on the design requirements although concrete pontoons are generally used in the newer bridges.

The pontoons are large water-tight chambers constructed off site and floated into place (see Figures 16.3.5 and 16.3.6). Despite their heavy concrete composition, the weight of the water displaced by the pontoons is equal to the weight of the structure (including all traffic), which allows the bridge to float. They may be prestressed concrete or reinforced concrete and are classified as either continuous pontoon type or separate pontoon type. The pontoons are held into place by huge steel cables anchored deep in the soil below water.

Figure 16.3.4 Brookfield, Vermont, Floating Bridge Constructed from Timber

Figure 16.3.5 Concrete Pontoons Under Construction

Figure 16.3.6 Concrete Pontoons Transported for Hood Canal Project

To control water leaking into the interior of the pontoons and ultimately sinking the bridges, each pontoon contains several water tight cells. This confines any flooding to a small area of the bridge. Access doors to the interior cells are watertight. Each cell may be equipped with water sensors for early detection of any leaks in the pontoons and a bilge pumping system to pump out water.

Bridge pontoons are designed to safely withstand wind and wave forces, major storms and vessel collisions.

Continuous Pontoons

Continuous pontoon bridges are made of individual pontoons, longitudinally connected to each other. The top of the pontoons may be the roadway or a superstructure may be built on top of the pontoons. The size of each pontoon is determined by design requirements as well as constraints imposed by the constructions facilities and the transportation route to the bridge site.

The floating bridges in use today in Washington State are of the continuous type (see Figure 16.3.7).

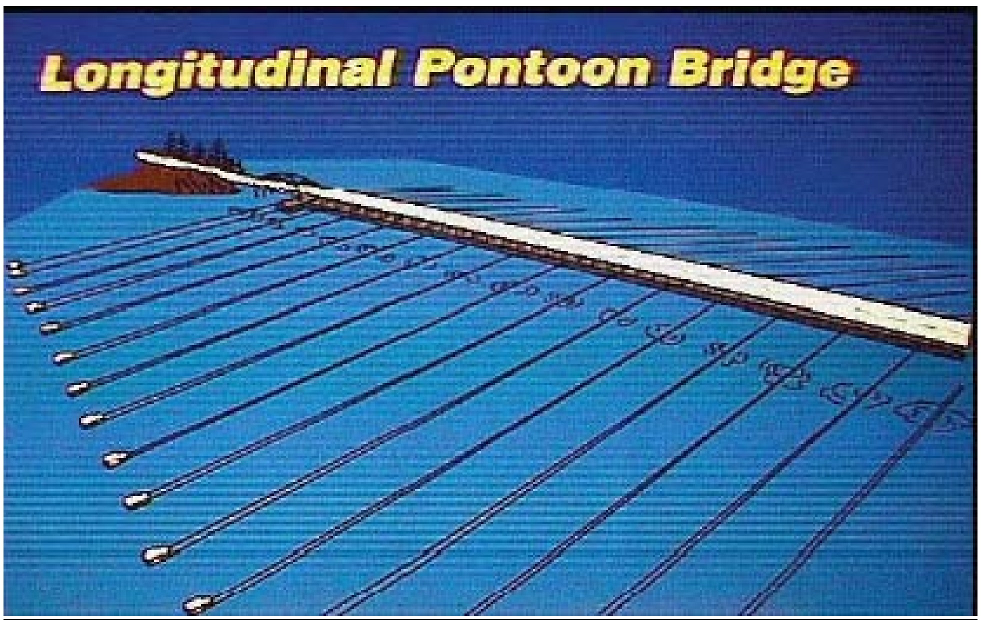

Figure 16.3.7 Continuous Pontoon-Type Structure

Separate Pontoons

A separate pontoon type of floating bridge consists of individual pontoons. These pontoons are placed transversely to the structure and are spanned by a steel or concrete superstructure (see Figures 16.3.8 and 16.3.9). The superstructure needs to be strong enough and rigid enough to maintain the position of the separated pontoons. A series of cables are attached to each pontoon and are anchored deep in the soil below water.

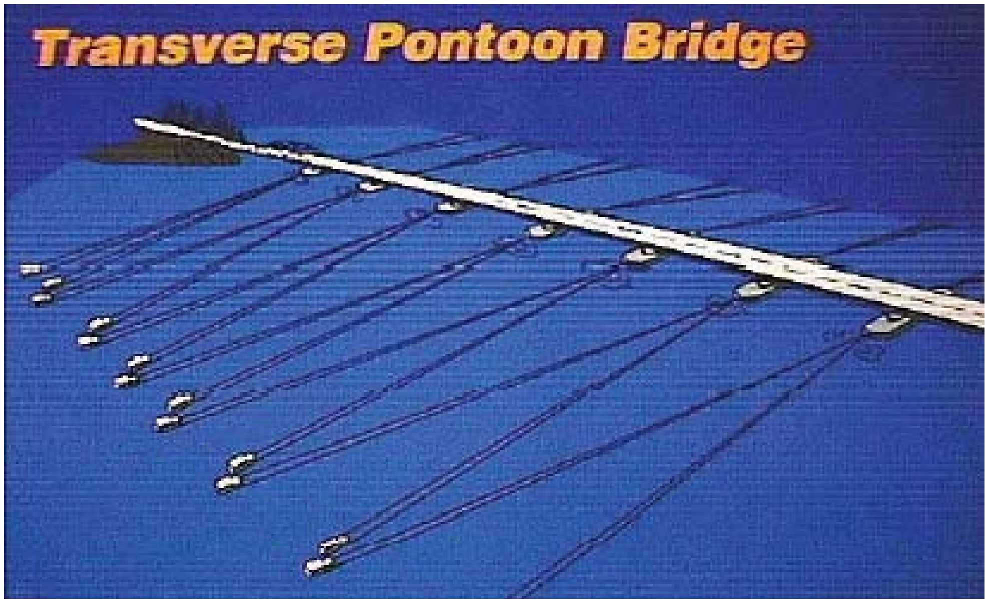

Figure 16.3.8 Separate Pontoon Type Structure

Figure 16.3.9 Bridge Constructed with Separate Pontoons

Anchoring Systems

Floating bridges are held in place in various ways such as a system of piles, caissons, cables, anchors and fixed guide structures. The most common type of system consists of cables and anchors. Anchor cables are normally two and one half inches in diameter and consist of dozens of individual steel strands (see Figure 16.3.10).

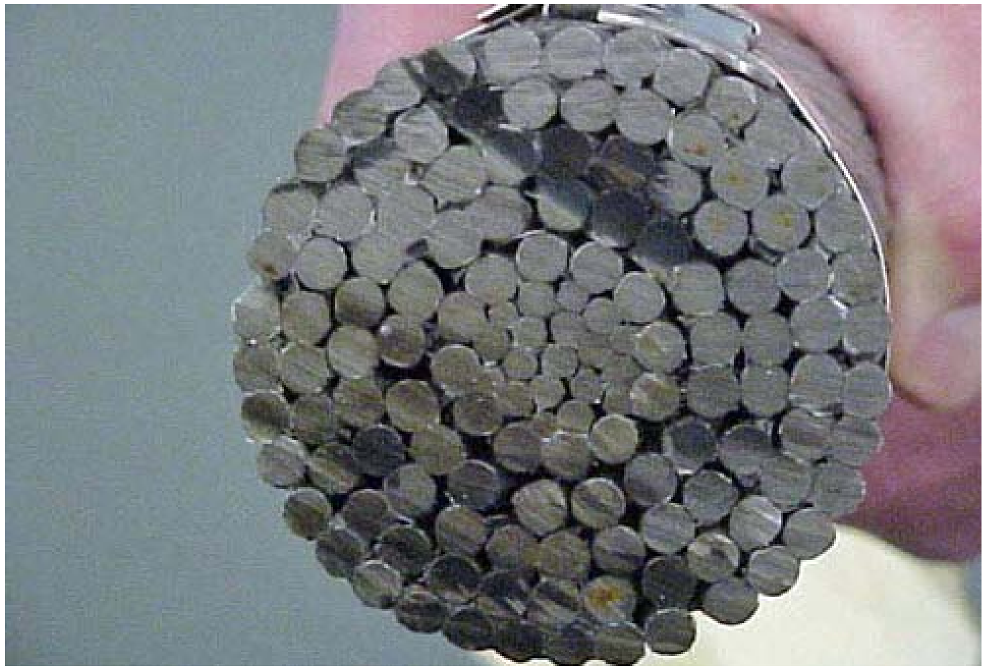

Figure 16.3.10 Cross-Section of Anchor Cable

Anchor cable saddles are used within the pontoon to guide and hold the cable in place (see Figure 16.3.11). Hydraulic jacks inside the pontoon tighten or release the pressure on the cables as the water level fluctuates under the bridge.

Figure 16.3.11 Anchor Cable Saddle

Types of Anchoring Systems

Depending on the depth of the water and the soil conditions, there are four primary types of anchoring systems used on the floating bridges: precast concrete fluke style anchor, pile anchor, open-cell gravity block anchor, and solid gravity slab anchor (stackable).

Precast Concrete Fluke Style Anchor

Precast concrete fluke style anchors are used in deep water with very soft soil conditions. Anchors weighing 60 to 86 tons are lowered to the soil below water. Water jets are turned on allowing the anchors to sink to the proper depth (see Figure 16.3.12).

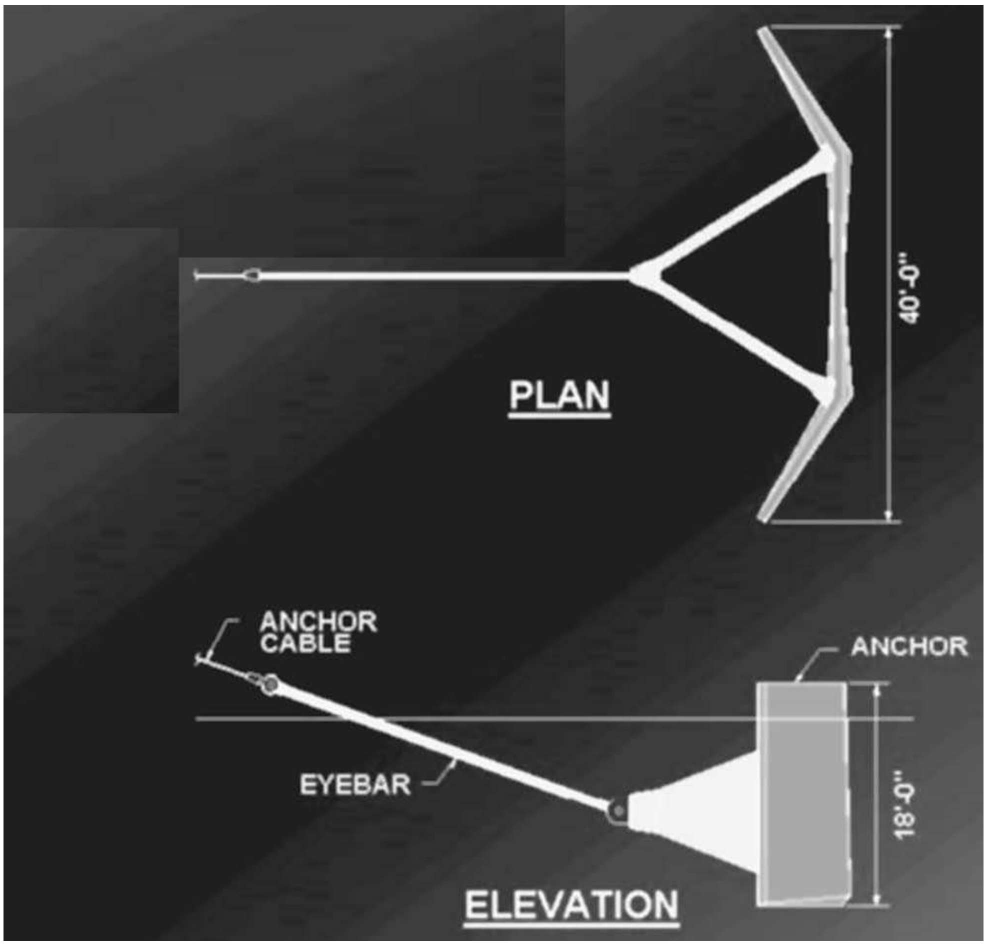

Figure 16.3.12 Precast Concrete Fluke Style Anchor

Pile Anchor

Pile anchors are designed for use in water depths less than 88 feet and with hard soil. Piles are driven into the surface to a specified depth and tied together to increase capacity (see Figure 16.3.13).

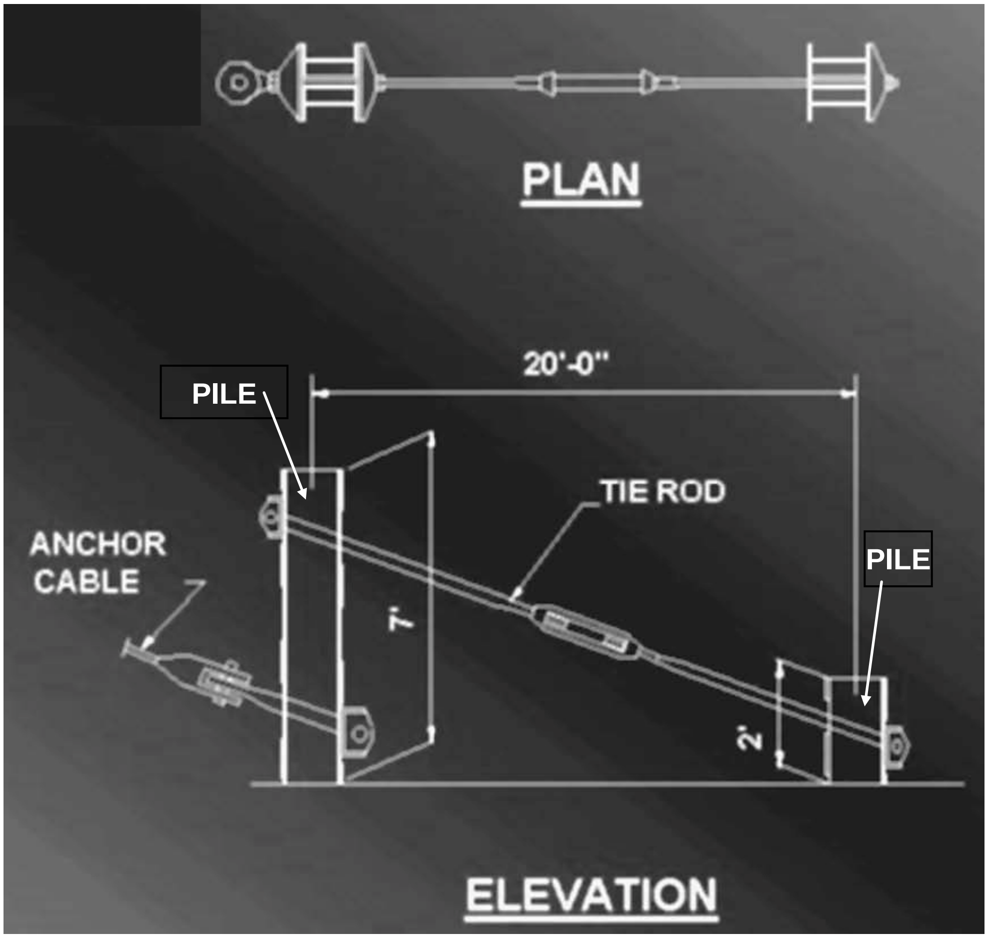

Figure 16.3.13 Pile Anchor

Open-Cell Gravity Block Anchor

Open-cell gravity block anchors are a gravity type of anchor. They are reinforced concrete boxes with an open top that are lowered into position and filled with gravel to a predetermined weight. This type or anchor is used in deep water where the soil is hard (see Figure 16.3.14).

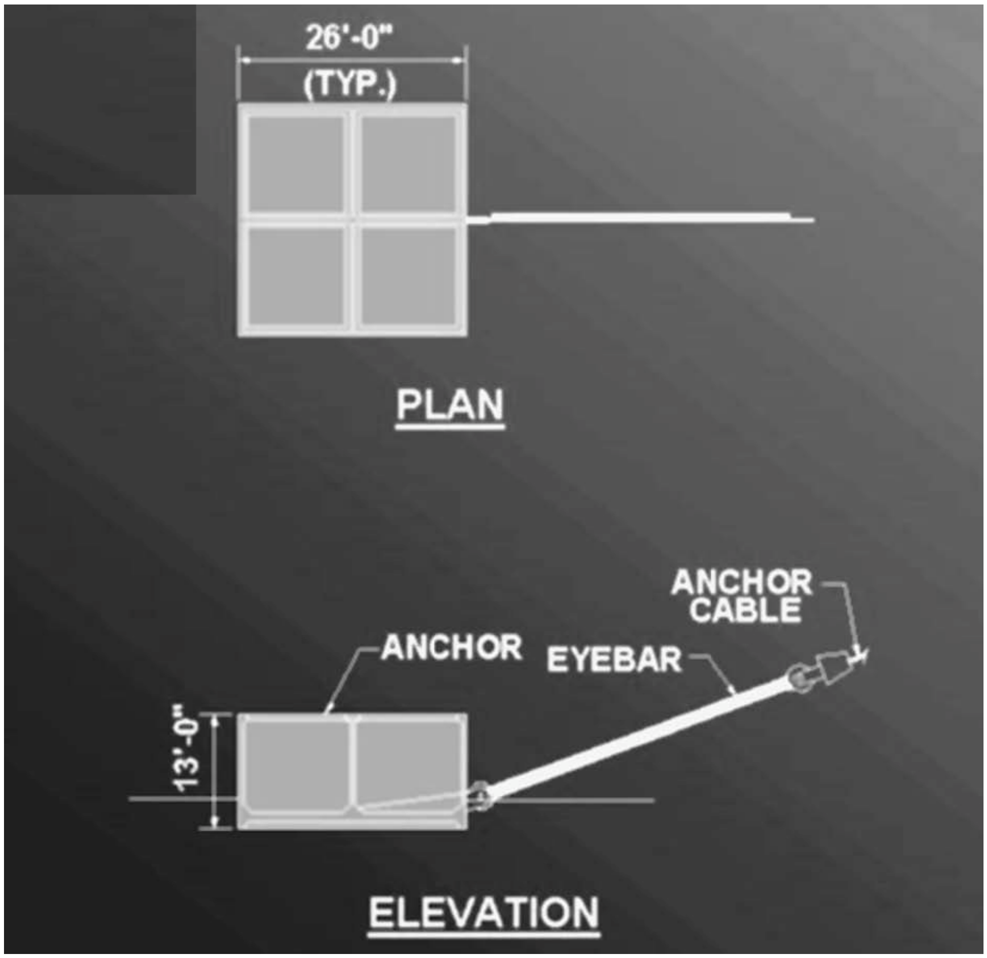

Figure 16.3.14 Open-Cell Gravity Block Anchor

Solid Gravity Slab Anchor (Stackable)

Solid gravity slab anchors are a gravity type of anchor. They can be used in either shallow or deep water where the soil is hard. These anchors are solid reinforced concrete slabs weighing up to 270 tons each. The first slab is lowered into position, and then additional slabs are added until the required anchoring capacity has been reached. Solid gravity slab anchors are the preferred anchor type because they are easy to cast and can be placed quickly (see Figure 16.3.15).

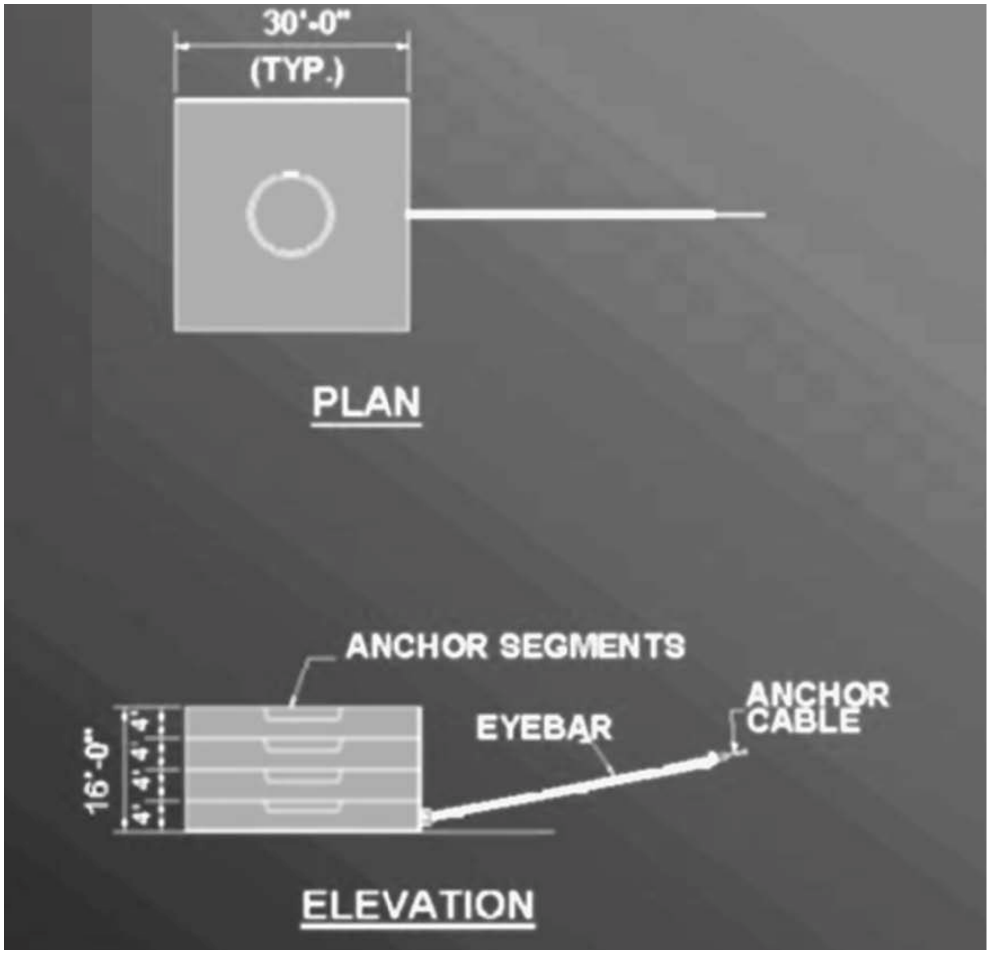

Figure 16.3.15 Solid Gravity Slab Anchor

16.3.3 Overview of Common Deficiencies

Common deficiencies that occur on floating bridges are:

- Corrosion of anchor cables
- Fatigue cracking
- Overloads
- Collision damage
- Water infiltration

Floating bridges may be constructed from steel, concrete or timber. Therefore, deficiencies will depend on the material used to construct the bridge. See Topics 6.1 (Timber), 6.2 (Concrete), and 6.3 (Steel) for specific information regarding deficiencies of each material type.

16.3.4 Inspection Locations and Methods

Because of their uniqueness and depending on the material used, floating bridges can prove challenging to an inspector. Floating bridges can be constructed of steel, concrete or timber, therefore a variety of inspection methods are utilized to thoroughly inspect the bridge. Additionally, since many floating bridges include an elevated conventional bridge structure or a moveable bridge section, those inspection methods and locations are to be considered by the inspection team.

See Chapter 6 for detailed description of anticipated modes of deterioration for common bridge materials. See Chapters 8 through 12 for the inspection and evaluation of timber superstructures, concrete superstructures, steel superstructures, bearings and substructures. See Topic 16.2 for detailed information about movable bridges.

Methods

Visual

Visual inspection of each pontoon cell will reveal any cracks or leaks. Pontoons have access hatches to allow for maintenance and inspection (see Figure 16.3.16).

Visually inspect concrete pontoons for the following deficiencies:

- Cracking
- Spalling
- Delamination
- Overload damage
- Collision damage
- Abrasion
- Loss of watertight seals on access doors and hatches
- Damaged cable connections

Visually inspect steel pontoons for the following deficiencies:

- Cracking
- Overload damage
- Collision damage
- Loss of watertight seals on access doors and hatches
- Coating failure
- Corrosion and section loss
- Damaged cable connections

Figure 16.3.16 Inspector Opening Pontoon Access Hatch

Physical

Measure and record the depth of any water found in each cell. The length, location and width of cracks found are to be accurately measured and recorded (see Figure 16.3.17). For steel pontoons and cables, remove corrosion and rust down to bare metal. With calipers or a D-meter, measure and record remaining section thickness. Use a hammer to check for delaminated areas in concrete pontoons.

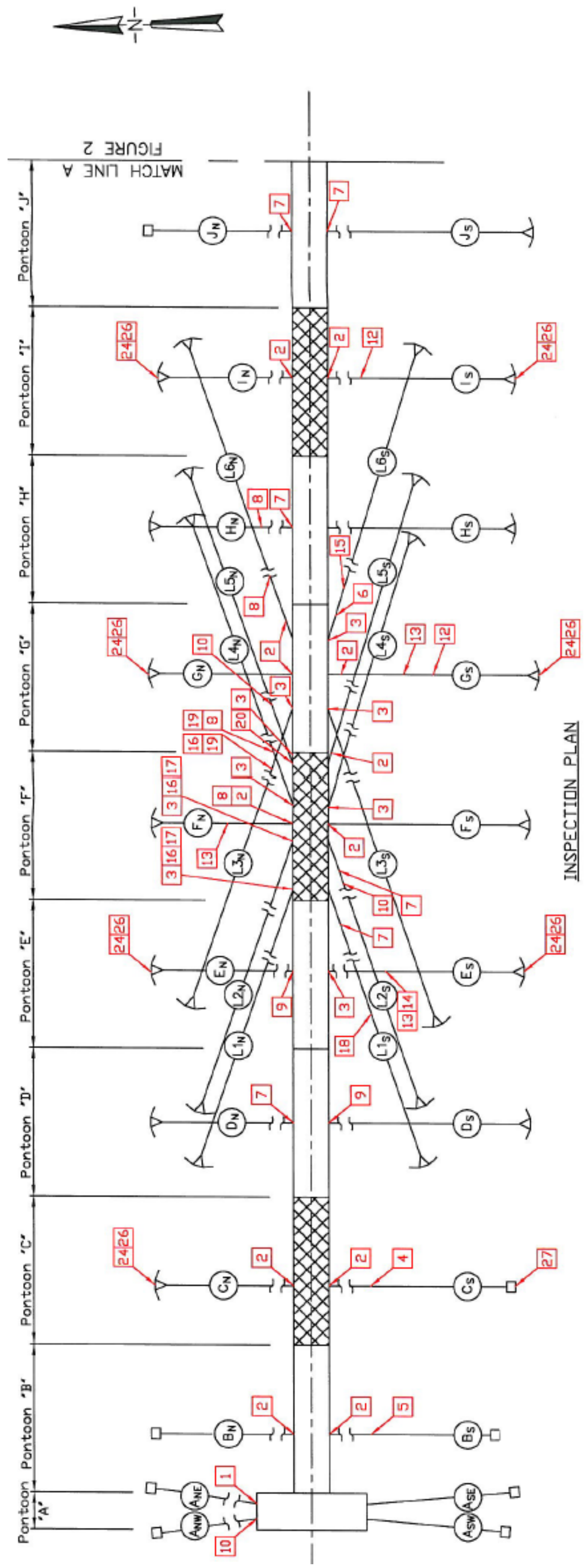

Figure 16.3.17 Sample Pontoon Inspection Plan

Advanced Inspection Methods

Many of the advanced inspection tools used above water have been adopted for underwater use. See Chapter 15 for the advanced inspection methods of timber, steel and concrete.

Anchors may be embedded 100 or more feet below the water surface. Inspection of the anchors will require underwater divers and equipment with the ability to detect any deficiencies present. See Topic 13.3, Underwater Inspection. Underwater cameras, sonar and other specialized equipment can provide access to cables and anchors.

Locations

Pontoons

Examine the floor of each pontoon cell for standing water. Examine pontoon walls and surfaces for cracks. Examine access doors, locks and hatches verifying that they are water tight and in proper working condition. Check the bilge pumping system and verify that it is in working order. Convey any noted problems with the pumping system to specialized maintenance personnel responsible for the system.

Examine the cable ends and anchor cable saddle inside the pontoon. Look at the connections in the pontoons for frayed or broken strands. Verify the presence and functioning of any cathodic protection system on the anchor cables.

Joints

When continuous pontoons are used, inspect the joint between the pontoons. Typically a rubber membrane or grout is used between the pontoons. Examine the alignment of the pontoons across the structure looking for signs of differential movement or distortion. This may indicate water leaking into one of the pontoons or some type of ballast balancing problem within the structure.

Cables

Examine the cable ends at the pontoon portals and check for cable misalignment and fraying. Check for broken wires that may indicate undue stress on the cable securing the pontoon (see Figure 6.3.18). Also check cables for heavy corrosion or section loss (see Figure 6.3.19).

Figure 16.3.18 Frayed Cables Removed from a Floating Bridge

Figure 16.3.19 Typical View of Heavy Corrosion within Pontoon Port

Anchors

Floating bridges are subjected to wind, tides and wave forces that are unpredictable and always changing. This exerts high levels of strain and stress on the cables and the anchors. Inspection of the anchors is not easily accomplished. Underwater remote equipment can provide information on each anchor. Look for any indication of anchor movement, misalignment or undermining of the anchor. Check the ballast on open-cell gravity block anchors to verify if there is enough material to keep the anchors in place.

16.3.5 Evaluation

State and Federal rating guideline systems have been developed to aid in the inspection of floating bridges. The two major rating guideline systems currently in use are the FHWA's *Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges* used for the National Bridge Inventory (NBI) component condition rating method and the AASHTO *Guide Manual for Bridge Element Inspection* for element level condition state assessment.

NBI Component Condition Rating Guidelines

Using the NBI component condition rating guidelines, a one-digit code on the Federal Structure Inventory and Appraisal (SI&A) sheet indicates the condition of the superstructure. Component condition rating codes range from 9 to 0 where 9 is the best rating possible. See Topic 4.2 (Item 59) for additional details about NBI component condition rating guidelines.

Consider previous inspection data along with current inspection findings to determine the correct component condition rating.

Element Level Condition State Assessment

In an element level condition state assessment of a floating bridge, possible AASHTO National Bridge Elements (NBEs) and Bridge Management Elements (BMEs) are:

NBE No.	Description
Superstructure	
107	Steel Girder/Beam
102	Steel Closed Web/Box Girder
113	Steel Stringer
152	Steel Floorbeam
147	Steel Cables
109	Prestressed Concrete Girder/Beam
104	Prestressed Concrete Closed Web/Box Girder
115	Prestressed Concrete Stringer
154	Prestressed Concrete Floorbeam
110	Reinforced Concrete Girder/Beam
105	Reinforced Concrete Closed Web/Box Girder
116	Reinforced Concrete Stringer
155	Reinforced Concrete Floorbeam
111	Timber Girder/Beam
117	Timber Stringer
156	Timber Floorbeam
Substructure	
310	Elastomeric Bearing
311	Moveable Bearing (roller, sliding, etc)
312	Enclosed/Concealed Bearing
313	Fixed Bearing
314	Pot Bearing
315	Disk Bearing

BME No.	Description
Wearing Surfaces and Protection Systems	
510	Wearing Surfaces
515	Steel Protective Coating
525	Concrete Protective Coating

The unit quantity for the superstructure elements is feet. The total length is distributed among the four available condition states depending on the extent and severity of the deficiency. The unit of quantity for bearings is each, with each bearing element placed in one of the four available condition states depending on the extent and severity of the deficiency. The unit quantity for wearing surfaces and protective coatings is area, and the total area is distributed among the four available condition states depending on the extent and severity of the deficiency. The sum of all condition states equals the total quantity of the National Bridge Element or Bridge Management Element. Condition State 1 is the best possible rating. See the *AASHTO Guide Manual for Bridge Element Inspection* for condition state descriptions.

The following Defect Flags are applicable in the evaluation of floating bridges:

Defect Flag No.	Description
356	Steel Cracking/Fatigue
357	Pack Rust
358	Concrete Cracking
359	Concrete Efflorescence
360	Settlement
361	Scour
362	Superstructure Traffic Impact (load capacity)
363	Steel Section Loss
364	Steel out-of-plane (Compression Member)

See the AASHTO *Guide Manual for Bridge Element Inspection* for the application of Defect Flags.

Appendix A

Sample Inspection Report

PORT AUTHORITY OF ALLEGHENY COUNTY
PITTSBURGH, PENNSYLVANIA

REPORT ON THE
NBIS INSPECTION
OF
CHARTIERS CREEK BRIDGE

BMS No. 02 7421 0000 9061

Submitted By:

Michael Baker Jr., Inc.
100 Airside Drive
Coraopolis, Pennsylvania
15108

September, 2011

STRUCTURE B.M.S. NUMBER:	02 7421 0000 9061
BRIDGE NAME:	Chartiers Creek Bridge
LOCATION:	Crafton, Pennsylvania
INSPECTION DATE:	June 23, 2011
INSPECTED BY:	Michael Baker Jr., Inc. Patrick A. Leach, P.E. Charles L. Molnar
PREPARED FOR:	Port Authority of Allegheny County
PREPARED BY:	Michael Baker Jr., Inc. Written By: Joseph E. Salvadori, E.I.T. Reviewed By: Raymond A. Hartle, P.E.
PORT AUTHORITY AGREEMENT NUMBER:	11-08
OWNER OF BRIDGE:	Port Authority of Allegheny County
COST INFORMATION:	Inspection & Report $4,662.00 Rigging $2,340.00 Traffic Control $ 0 Railroad $ 0 Insurance $ 0

DATE SUBMITTED: (Seal removed for BIRM)

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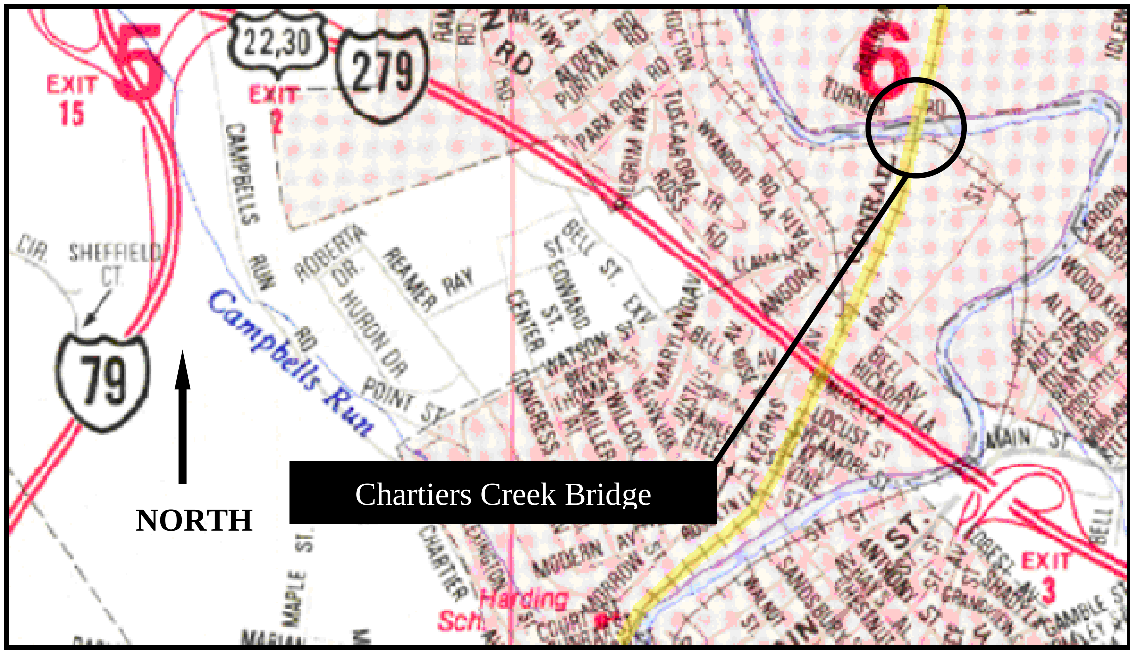

Location Map

(No Scale)

REPORT ON THE INITIAL NBIS INSPECTION
OF
CHARTIERS CREEK BRIDGE

PORT AUTHORITY OF ALLEGHENY COUNTY

II. INTRODUCTION:

- Location

  Located in the Borough of Crafton, the Chartiers Creek Bridge carries two (2) lanes of the Port Authority of Allegheny County's West Busway over Chartiers Creek, and the Pittsburgh Industrial Railroad, Inc.

- Year Built

  The approximate date of the original construction of the Chartiers Creek Bridge is 1948. The structure was built by the Pennsylvania Railroad Company. Rehabilitation was completed in July 1997.

- Load Posting

  None required.

- Description

  The Chartiers Creek Bridge is a three (3) span, non-composite, riveted and bolted built-up plate girder bridge with a total length of 253'- 11" (see photo no. 1). The 3 spans consist of one (1) main simple span 124'-0", one (1) simple south end span 55'-3", and one (1) simple north end span 68'-3" long. The span lengths are measured between centerline of bearings. The skew angle measured between the centerline of the abutment and West Busway is 90^0. There are AT&T conduits mounted under the deck, and light poles mounted on top of the concrete parapets (see photo no.'s 8 & 2, respectively).

Chartiers Creek Bridge

The superstructure consists of four girders spaced at 7’-0” – 6’-0” – 7’-0” on centers, are laterally restrained with angle cross framing, and support an 8 1/2” reinforced concrete deck. The deck thickness includes a 1/2” integral-wearing surface. The deck measures 28’-0” between the reinforced concrete parapets present on both sides of the structure. Galvanized stay-in-place deck forms are present on the underside of the deck (see photo no. 8).

Span 1 girders are made up of a 5’-11” deep by 1/2” thick web plates, and 18” wide by 3/4” thick top and bottom flange plates (see photo no. 8). The main span consists of a 10’- 4 1/2” deep by 1/2” thick web plate, and top and bottom flange plates varying from 20” wide by 7/8” thick, to 20” wide by 1” thick (see photo no. 9). Span 3 girders are made up of a 6’- 10 1/2” deep by 1/2” thick web plate, and 18” wide by 3/4” and 7/8” thick top and bottom flange plates (see photo no. 10). New knee brackets, bolted to the fascia girders, measure 4’-9” wide, from the centerline of existing fascia girders to the centerline of the new W24x55 fascia stringers, with 1/2” thick web plates, and 6” wide by 1/2” thick top and bottom flange plates (see photo no. 4). Lateral bracing and diaphragms consist of angles, and angle x-bracing, respectively. Laminated elastomeric bearing pads are present at the girder ends.

The main span vertical underclearance, from the existing concrete channel bottom, at the centerline of the railroad measures 60’-9” and 36’-7” in span 1.

Gravity type substructures consist of a combination of original stone construction with newly constructed reinforced concrete abutment backwalls and pier caps (see photo no.’s 4 to 7).

III. INSPECTION FINDINGS:

Michael Baker Jr., Inc. performed this initial inspection, which follows NBIS procedures, on June 23, 2011, via a UB-40 underbridge inspection crane. In general, the structure was in good condition with a few minor problems. Several conduits at the south abutment and in span 1 have severely buckled segments, and broken couplers and/or adapters (see photo no.'s 12 & 13). In addition, a conduit in span 3 is split and leaking water (see photo no. 14). These problems are due to the junction boxes being allowed to fill with rainwater during construction.

Approach

The north and south approach roadway and slabs are newly constructed with no deficiencies noted.

Deck

No deficiencies noted – new construction (see photo no. 11). All PennDOT Type 1 scuppers are in excellent condition. A few scuppers exhibit minor debris accumulation but are fully functional (see photo no. 15). Random hairline (< 0.01") shrinkage cracks along the length of the concrete parapets are present (see photo no. 16). Deck expansion joints consist of strip seals in good condition with minor debris accumulation (see photo no. 17).

Superstructure

The superstructure has no visible structural deficiencies. Girders, fascia stringers, knee brackets, and lateral bracing are newly painted. The paint shows no visual defects, but the girders and bracing exhibit evidence of prior minor section loss and member pitting. Fascia stringers and knee brackets are in new condition with no deficiencies noted (see photo no. 4). Diaphragms are in good condition, but show areas of freckled surface rust under the broken conduit in span 1. Approximately 50% of lateral bracing connections between girders 3 & 4, in span 2, were not painted with final paint coat (see photo no. 18). Laminated elastomeric bearing pads are functioning properly with no problems noted.

Substructure

The north and south abutments are in good condition, with a few minor problems noted. Both abutments have newly constructed reinforced concrete backwalls, bridge seats, and wingwalls with no visual deficiencies noted (see photo no.'s 4 & 5). The stem tops consist of new reinforced concrete construction, also with no visual deficiencies noted, and are attached to the existing stone masonry bases. Some locations of the stone masonry show minor cracking and loosening of mortar.

Piers 1 & 2 are in good condition with minor cracking and loosening of mortar on the existing stone masonry portion of the stems. The bridge seats, caps, and stem tops are newly constructed reinforced concrete with no visual deficiencies noted (see photo no.'s 6 & 7).

Photo No. 1 General Elevation (Upstream)

Photo No.2 South Approach (near)

Photo No.3 North Approach (far)

Photo No.4 South Abutment (near) - Elevation

Photo No.5 North Abutment (far) - Elevation

Photo No.6 Pier 1 - North Face (Looking South)

Photo No.7 Pier 2 - North Face (Looking South), note electrical lines

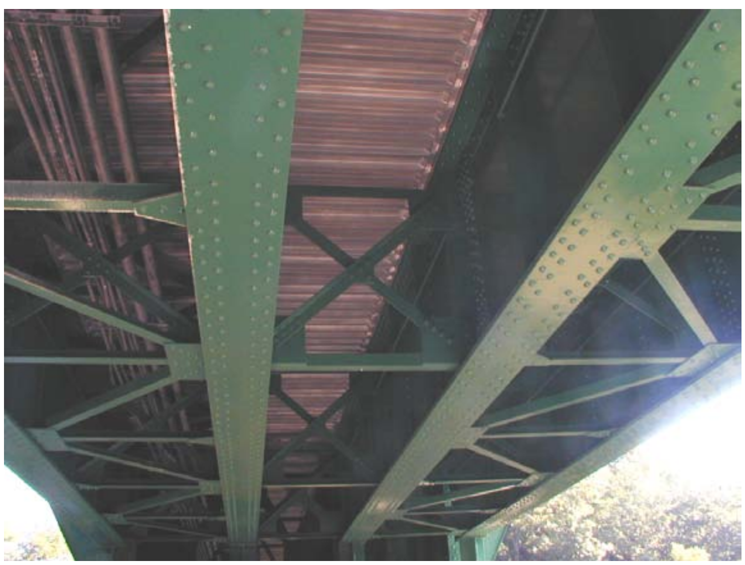

Photo No.8 General Underside View – Span 1

Photo No.9 General Underside View – Span 2

Photo No. 10 General Underside View – Span 3

Photo No. 11 General Deck View

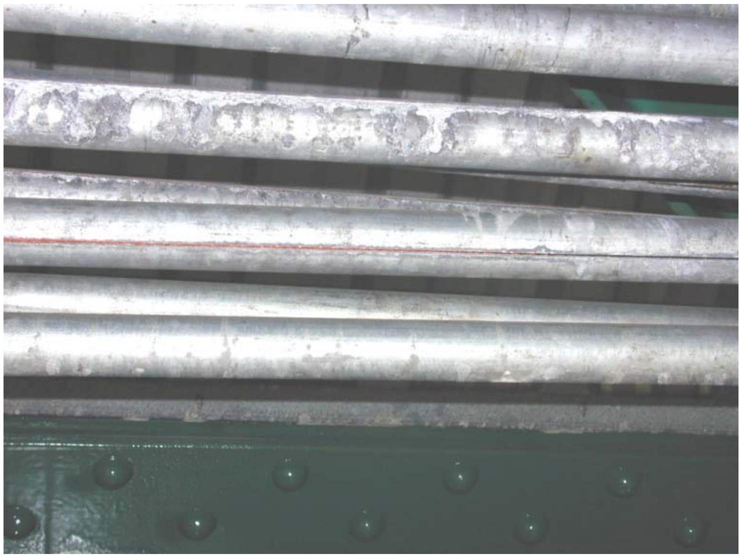

Photo No. 12 Conduit, Span 1 – note longitudinal crack/split

Photo No. 13 Conduit and Couplers, Span 1 – note bend in conduit, and coupler separation

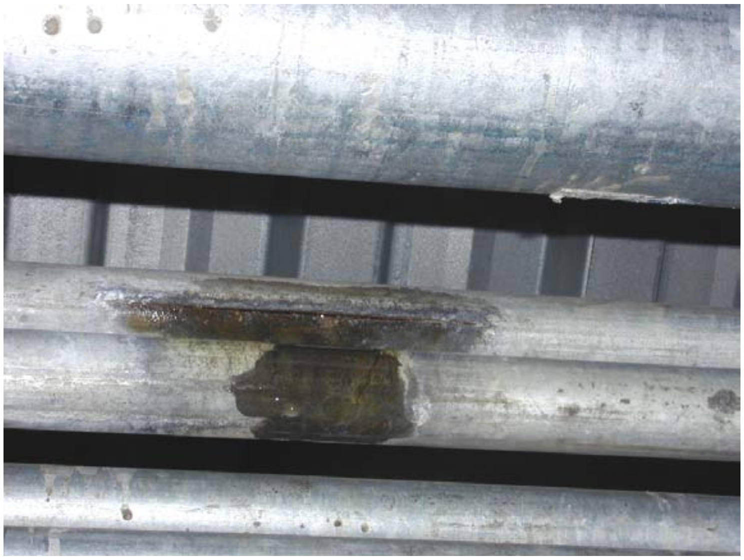

Photo No.14 Conduit , Span 3 – note conduit is split and leaking water

Photo No. 15 Typical PennDOT Type 1 Scupper

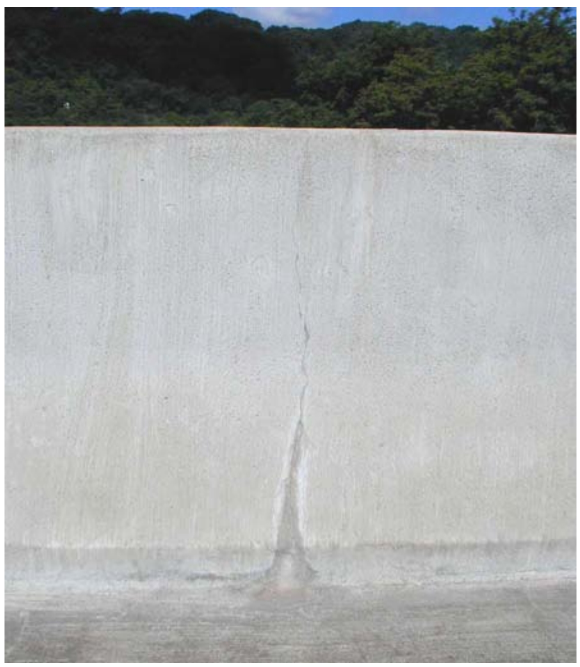

Photo No.16 Typical parapet crack

Photo No. 17 Strip Seal at North Abutment (typ.) – note minor debris accumulation

Photo No.18 Lateral bracing connection between beam #3 and #4, in span 2 – note no final paint coat, and rust freckles

IV. STRUCTURAL ANALYSIS:

Bridge Load Ratings (Tons)

LOAD FACTOR	H	HS	ML	P
Inventory w/o F.W.S	115	159	152	---
Inventory w/ F.W.S	112	155	148	---
Operating w/o F.W.S	191	265	253	346
Operating w/ F.W.S	187	259	247	338

Note: 1) Critical rating is for a beam controlled by shear in span 3
2) Due to no analysis being performed as part of the inspection, the above table is reproduced from contract drawings.

V. RECOMMENDATIONS AND COST ESTIMATE:

Repairs

Item	Estimated Quantity	Unit Cost	Total Cost
Drain junction boxes, and conduits filled with water. Repair bent conduits, and broken couplers/adapters.	N/A	Lump Sum	$7,500.00
Paint locations requiring final paint coat between girders 3 & 4 in span 2.	20 SF	Lump Sum	$1,500.00

TOTAL COST $9,000.00

Note: The above costs are only for the items listed and do not include additional costs which would be incurred when the work is performed, such as mobilization, maintenance and traffic protection, engineering, etc.

PDT Form D-450A (DEC 1996) **Site Data** **BRIDGE MANAGEMENT SYSTEM** BRIDGE INSPECTION REPORT

BMS Updated by ______ Date ______

C05 Structure Type (Dept.)

A01 | 0 | 2 | 7 | 4 | 2 | 1 | 0 | 0 | 0 | 0 | 9 | 0 | 6 | 1

Main STL. RIVETED I-BEAM | 1 | 9 | 1 | 1 | 0

CHARTIERS CREEK BRIDGE Over CHARTIERS CREEK

Approach

Inspection Date
E06 | 0 | 6 | 2 | 3 | 0 | 0

Name of Consultant and/or Inspectors
E12 | M | I | C | H | A | E | L | | B | A | K | E | R | | J | R., | | I | N | C.

Inspection Type E07 | 1

Inspected by E08 | 8

Hired by E13 | 8

Time started 7:30 A.M.

Weather Conditions: Temp: 84

CRAFTON

Time completed 4:30 P.M.

MOSTLY SUNNY

☐ City ☒ Borough ☐ Township

Optional Reminder:
Check boxes if Maintenance
Activities are needed --> X

Bridge Signing Verification

BMS Item	Type of Sign	Required Sign	SIGNING IN FIELD: Near Advance	Bridge Site: Near	Bridge Site: Far	Far Advance	Comments
D15	Bridge Weight Limit	N/A T					NONE POSTED
D15	Except Combination	N/A T					
D14	One Truck at a Time	Yes / No					
B22/B23	Vert. Clearance - On	N/A					See Sketch
B22/B23	Vert. Clearance - Und	N/A					See Sketch
	One Lane Bridge	Yes / No	(Opt)			(Opt)	
	Narrow Bridge	Yes / No	(Opt)			(Opt)	
	Hazard Clearance	Yes / No					
	Other						
(Opt)	Other						

Key --> OK: Signs properly installed M: Signs missing D: Signs damaged / incorrect

New Wearing Surface Under Bridge: YES ☐ NO ☐

Notes

Vert. Clear. Sign **On Feature:** B01 = ☐ B31 = ☐ **Under Feature:** B01 = ☐ B31 = ☐

E26 Underclearance Appraisal | 5 | Controlling: Lateral 12'-2" Vertical 36'-7"

E28-A Traffic Safety Features (Subfields shown vertically) Posted Speed Limit ______ mph

6 Bridge Railing PARAPET - JERSEY BARRIER. (GOOD CONDITION - MINOR CRACKING THROUGHOUT)

8 Transition PARAPET EXTENSIONS.

8 Approach Guiderail ON RIGHT - CONTINUOUS NJ BARRIER - GOOD. W-BEAM AND STL. POSTS ON NEAR LT. AND FAR LT.

6 Approach Rail Ends FLARED AND TURNED DOWN W-BEAM ON NEAR LT. AND FAR LT.

E28 Approach Alignment | 8 | NO SPEED REDUCTION. GOOD SIGHT DISTANCE.

E15 Approach Roadway | 8 | NEW PAVEMENT GOOD CONDITION.

Pavement GOOD

Drainage GOOD (ALL NEW CONSTRUCTION)

Shoulders GOOD

E14 Approach Slab | 8 | NEW CONSTRUCTION.

Bump at Bridge Yes ☐ No X

C19 Relief Joint | 1 |

PDT Form D-450B (DEC 1996)

Bridge 1 Data

A01 | 0 2 | 7 4 2 1 | 0 0 0 0 | 9 0 6 1

Inspection Date E06 | 0 6 2 3 1 1

For Non-State Roadways

B01 Ref	B27 ADT	B28 ADT YR	B30A ADTT %

For State highways, data from RMS will be used.

E25 Deck Geometry 6 Table ____ Controlling Values: B27 / B34 / B22 ____ A31 / A31 / B18 ____

Design Exception granted ? ____

E16 Deck Wearing Surface 9 NEW CONSTRUCTION (CONCRETE INTEGRAL)

C10 Wearing Surface Type 1 0 1 C10A Wearing Surface Thickness 0 5

E17 Deck 9 Estimated Spall or Delamination ____ % Est. Chloride Content ____

Top EXCELLENT CONDITION - NEW CONSTRUCTION.

Underside STAY IN PLACE FORMS (NO RUSTING NOTED) GALVANIZED AND IN GOOD CONDITION.

Exp Joint No. 4 C22 Exp Jt Types M B G

GOOD CONDITION - SOME MINOR DIRT BUILD UP. (STRIP SEALS)

Deck Drainage GOOD - SOME SCUPPERS HAVE DEBRIS BUT NOT IN THE DOWNSPOUT.

E18 Superstructure 7 See Sheet ____ for Additional Details. Form 491-J attached for FCM details Yes/No

Girders / Beams GOOD CONDITION - SUPERSTRUCTURE HAS BEEN RECONSTRUCTED FOR NEW BUSWAY BRIDGE. NEW PAINT/COATING OVER PREVIOUS PITTING/MORE SECTION LOSS. ALSO, SOME AREAS OVER LIGHT SURFACE RUST ON BOTTOM FLANGE. (THROUGHOUT)

Floorbeams N/A

Stringers NEW (FASCIA STRINGERS) W24 X 55 EXCELLENT CONDITION.

Diaphrams GOOD CONDITION. FEW AREAS OF FRECKLED SURFACE RUST UNDER BROKEN CONDUIT IN SPAN 1.

Truss Members N/A

Portals / Bracing FEW AREAS OF FRECKLED SURFACE RUST UNDER BROKEN CONDUIT IN SPAN 1. SEVERAL AREAS BETWEEN G3 AND G4 IN SPAN 2 WERE NOT PAINTED WITH FINAL COAT.

Bearings GOOD CONDITION. (LAMINATED ELASTOMERIC)

Drainage System (Below Deck) EXCELLENT CONDITION. (TYPE 1 SCUPPERS)

PDT Form D-450C **Abutment Data** Inspection Date

(DEC 1996) A01 | 0 2 | 7 4 2 1 | 0 0 0 0 | 9 0 6 1 | E06 | 0 6 2 3 1 1

E20 Substructure 7 Details on Sheet

NAB - Near Abutment (Use same notation as W09)

Backwall GOOD CONDITION - NEW CONSTRUCTION.

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION. VERY MINOR DEBRIS.

Cheekwalls

Stem GOOD CONDITION - NEW CONCRETE CONSTRUCTION AT TOP ON EXISTING STONE MASONRY BASE. SOME LOCATIONS HAVE MINOR CRACKING AND LOOSENING OF MORTAR.

Wings GOOD CONDITION - NEW CONSTRUCTION.

Footing NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

ABUTMENT IS NOT IN CHANNEL. ALSO, CHANNEL IS CONCRETE LINED.

Settlement NONE NOTED.

Embank-Slope-Wall GOOD CONDITION - HEAVY VEGETATION.

Wall Drainage

FAB - Far Abutment (Use same notation as W09)

Backwall GOOD CONDITION - NEW CONSTRUCTION.

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION. MINOR DEBRIS.

Cheekwalls

Stem GOOD CONDITION - SAME AS NEAR ABUTMENT.

Wings GOOD CONDITION - NEW CONSTRUCTION.

Footing NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

ABUTMENT IS NOT IN THE CHANNEL.

Settlement NONE NOTED.

Embank-Slope-Wall HEAVY VEGETATION.

Wall Drainage

PDT Form D-450D **Pier Data** Inspection Date

(DEC 1996) A01 | 0 | 2 | 7 | 4 | 2 | 1 | 0 | 0 | 0 | 0 | 9 | 0 | 6 | 1 | E06 | 0 | 6 | 2 | 3 | 1 | 1

Substructure (Cont.)

Pier / Bent Number 1 (Use same notation as W09)

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION.

Caps GOOD CONDITION - NEW CONSTRUCTION.

Cheekwalls

Columns/Stems GOOD CONDITION - NEW CONSTRUCTION ON TOP OF EXISTING STONE MASONRY BASE. MINOR CRACKING AND LOOSE MORTAR.

Footings NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

NOT IN CHANNEL - CHANNEL IS CONCRETE LINED.

Settlement NONE NOTED.

Pier / Bent Number 2 (Use same notation as W09)

Bridge Seats GOOD CONDITION - NEW CONSTRUCTION.

Caps GOOD CONDITION - NEW CONSTRUCTION.

Cheekwalls

Columns/Stems GOOD CONDITION - SAME AS PIER 1.

Footings NOT VISIBLE.

Piles NOT VISIBLE.

Scour / Undermine Yes [ ] No [X] See Details on Form Sheet

CHANNEL IS CONCRETE LINED.

Settlement NONE NOTED.

PDT Form D-450E (DEC 1996)

Waterway 1 Data

BRIDGE MANAGEMENT SYSTEM
BRIDGE INSPECTION REPORT

BMS Updated by ________ Date ________

A01 | 0 2 | 7 4 2 1 | 0 0 0 0 | 9 0 6 1

U.W. Inspection Date W01-A

Over ________ Weather Conditions ________

Inspection Type W02 N

U.W. Inspection Type W02-A

Regular U.W. Insp. Freq. W03

Interim U.W. Inps. Freq. W04

Time started ________

Time completed ________

Name of Consultant and/or Inspectors W16

Hired by W17

Inspection Cost W15

E29A Scour Critical Rating W06 9 based on: [X] Observed Scour [ ] Scour Calculation

No. of Units Inspected W14

Streambed Material W07 C 8 (36 SPACES) CONCRET LINED CHANNEL.

E21 Channel/Channel Protection - Cond. Rating 7 Details on Sheet ________

Channel: CHANNEL IS LINED WITH CONCRETE.

Banks: GOOD CONDITION - HEAVY VEGETATION.

Streambed Movements: NONE NOTED.

Debris, Vegetation: SOME DEBRIS IN CHANNEL.

River (Stream) Control Devices: N/A

Embankment / Streambed Controls: N/A

Drift, Other: NONE NOTED.

E27 Waterway Adequacy 9

Risk of Overtopping [X] Remote [ ] Slight [ ] Occasional [ ] Frequent

Traffic Delay [X] Insignificant [ ] Significant [ ] Severe **B18 - Functional Class.** ________

High Water Mark: ELEV : ________ DATE (mmyyyy) ________ [ ] New HW Mark [ ] HW since last inspecion

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures
N A B	P	0 0	9	E	0 0 0	

Findings: ABUTMENT OUT OF FLOOD PLANE.

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures
P 0 1	P	0 0	9	E	0 0 0	

Findings:

Form D-450F
(DEC 1996)

Waterway 2 Data

U.W. Inspection Date

A01					W01-A	
	0 2	7 4 2 1	0 0 0 0	9 0 6 1		

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
P 0 2	P	0 0	9	E	0 0 0	

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
F A B	P	0 0	9	E	0 0 0	

Findings: ABUTMENT OUT OF FLOOD PLANE.

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

W09 Substructure Unit	W10 Foundation Type	W11 Water Depth	W11-A Observed Scour Rating	W11-B U.W. Insp Performed	W11-C Observed Depth	W11-F Counter-Measures

Findings:

PDT Form D-450G **Waterway 3 Data**

(DEC 1996) U.W. Inspection Date

A01	0	2	7	4	2	1	0	0	0	0	9	0	6	1	W01-A					

OBSERVED SCOUR RATING GUIDE

Rating	ITEM NUMBER								Rating
	1	2	3	4	5	6	7	8	
	Change Since Last Inspection	Scour Hole	Debris Potential	Substructure Scourability	Opening Adequacy/ Channel	Sediment	Alignment	Velocity/ Stream Slope	
9	None	None	None	NF/P9/R9	Good	None	Good	Low	9
8	None	Minor	None	P8/C8/R8	Good	Minor	Good	Low	8
7	Minor	Minor	Minor	P7/C7/R7	Fair	Minor	Good	Medium	7
6	Minor	Advanced	Medium*	A6	Fair	Medium	Medium	Medium	6
5	Medium*	Advanced	High*	A5	Fair	High	Medium	High	5
4	Medium	Serious*	High	R4*/A4*	Poor*	High	Poor*+	High	4
3	High*	Serious*	Present*	A3	Overtop*	High	Poor	High	3
2	Bridge is scour critical, IMMEDIATE action is required *								2
1	Bridge is scour critical, bridge is CLOSED *								1
0	Bridge has failed due to scour *								0

NOTES:

Rating considerations given in highest to lowest level of importance from left to right.

* If an item is so marked, it cannot be given a higher ranking.

s founded on competent rock and no problems exist.

C = Effective Countermeasures

P = Pile Supported Substructures

DETERMINATION OF RATING FOR BMS ITEM W11-A

Substructure Unit	1	2	3	4	5	6	7	8	**W11-A**
	Change Since Last Inspection	Scour Hole	Debris Potential	Scourability	Opening Adequacy/ Channel	Sediment	Alignment	Velocity/ Stream Slope	Overall Observed Scour Rating
P02	9	9	8	8	9	7	9	7	9

<u>If Underwater Inspection only</u>

Signatures and Date:

PDT Form D-450K **Bridge 2 Data**

(DEC 1996)

A01 | 0 | 2 | 7 | 4 | 2 | 1 | 0 | 0 | 0 | 0 | 9 | 0 | 6 | 1

Inspection Date E06 | 0 | 6 | 2 | 3 | 1 | 1

E19 Paint Condition 8 8 New Paint Y / N If Yes: [ ] Spot [ ] Zone [X] Full [ ] Revise item G08-G17

Interior Beam / Girder VERY GOOD - RECENTLY REPAINTED.

Fascias VERY GOOD - NEW.

Splash Zone: Truss / Girder

Truss

Bearings VERY GOOD.

Other

E23 Est. Remaining Life BMS to Calculate Yes/No 3 4 Comments

Recalculate IR/OR: Yes [ ] Due to: Deterioration [ ] New Wearing Surf. [ ] Other [ ]

No [X] Previous Rating Dated ________ is still valid

	H	HS	ML-80	Other	Other	HS Load Factor
E30 Inventory Rating	1 9 8	2 9 8	8 9 8			2 9 8
E31 Operating Rating	1 9 8	2 9 8	8 9 8			2 9 8

E32 Rate Meth 2 S E33 Typ Mem 1 AASHTO E37 Spec 9 4 E38 Manual 9 4

E29 Bridge Post 9 CONTROLLING: H ____ HS ____ ML80 X Engineering Judgement ____

E24 Structural Condition Appraisal 7 Based upon [ ] Table 1 B27-ADT ____ B30-IR ____

or E18-Super 7 E20-Sub 7 , E22-Culvert ____ ,

E01 Next Insp. Freq. 2 4 E03 Equip. Next Insp. B SNOOPER TRUCK (UB-40)

E04 Spec. Insp. Type [ ] E05 By Date [ ]

Is bridge over water? [X] Yes. E22 = N Complete Forms D-450E through G

[ ] No. E22 = N E21 = N E27 = N E29A = N

Notes: ONE SPAN IS OVER WATER AND ONE SPAN IS OVER RAILROAD.

HAD RAILROAD REPRESENTITIVE ON SITE. CREW WAS OUT OF SPAN 1 (RR LOCATION) BY TIME REQUIRED. (9 A.M.)

INSPECTION WAS FIRST ON NEWLY CONSTRUCTED BUSWAY BRIDGE.WHICH USED AN EXISTING RR BRIDGE. CONDUITS ON BRIDGE WERE BUSTED AT ADAPTERS AT ABUTMENT 1. ALSO, ONE EXPANSION COUPLER WAS BROKEN AND NEEDED REPLACED. SEVERAL CONDUIT SEGMENTS IN SPAN 1 WERE SEVERELY BUCKLED AND NEEDED REPLACED.

Signatures and Date: PATRICK LEACH, P.E. - 6/23/11

CHARLES MOLNAR - 6/23/11

PDT Form D-450M **Maintenance Needs Data**

(DEC 1996)

A01															E06						
	0	2	7	4	2	1	0	0	0	0	9	0	6	1	Inspection Date	0	6	2	3	1	1

Approach Roadway Work

	Item # (H01)	Location (H03)		Quantity (H05)	PR (H08)	D/C (H09)
Pavement (Patch/Raise)	RDPAVMT	L N R L F R	SY			
Pavement Relief Jt (Rep/Repl)	RDRLFJT	L N R L F R	SY			
Shoulders (Repair/Reconstr)	RDSHLDR	L N R L F R	SY			
Drainage-Off Bridge (Improve)	RDDRAIN	L N R L F R	EA			
GR/Trans/End (Rep/Repl/Imp)	RDGDERL	L N R L F R	EA			
Load Limit Signs (Replace)	RDLDSGN	L N R L F R	EA			
Clearance Signs (Replace)	RDCLSGN	L N R L F R	EA			
Cut Brush to Clear Signs	RDBRUSH	L N R L F R	EA			
Approach Slab (Replace)	A744201	L N R L F R	SY			

Cleaning - Flushing

Deck	A743101	-- --	EB			
Scupper/Down Spouting	B743101	1 2 3 4 5 O	EB			
Bearing/Bearing Seat	C743102	1 2 3 4 5 O	EB			
Steel-Horizontal Surfaces	D743102	1 2 3 4 5 O	EB			

Deck

Bitum Deck W Surf (Rep/Repl)	BITWRGS	1 2 3 4 5 O	SY			
Timber Deck (Rep/Repl)	B744301	1 2 3 4 5 O	SY			
Open Steel Grid (Rep/Repl)	C744302	1 2 3 4 5 O	SY			
Concrete Deck (Repair)	D744303	1 2 3 4 5 O	SY			
Concrete Sidewalk (Repair)	E744303	1 2 3 4 5 O	SY			
Concrete Curb/Parapet (Rep)	F744303	1 2 3 4 5 O	SY			

Deck Joints - Expansion Joints

Reseal	A743301	N 1 2 3 O F	LF			
Repair/Reseal	A744101	N 1 2 3 O F	LF			
Compression Seal (Rep/Rehab)	B744102	N 1 2 3 O F	LF			
Modular Dam (Rep/Rehab)	C744102	N 1 2 3 O F	LF			
Steel Dams (Rep/Rehab)	D744102	N 1 2 3 O F	LF			
Other Types (Rep/Rehab)	E744102	N 1 2 3 O F	LF			

Bridge Railings - Parapets

Bridge Parapet (Rep/Repl)	RLGBRPR	N 1 2 3 O F	LF			
Struct Mount GR (Rep/Repl)	RLGSTRM	N 1 2 3 O F	LF			
Pedestrian (Rep/Repl)	RLGPEDN	N 1 2 3 O F	LF			
Median Barrier (Rep/Repl)	RLGMEDB	1 2 3 4 5 O	LF			

Deck Drainage

Scupper Grate (Replace)	DRNGRAT	1 2 3 4 5 O	EA			
Drain/Scupper (Install)	B744401	1 2 3 4 5 O	EA			
Downspouting (Rep/Repl)	C744402	N 1 2 3 O F	EA			

Bearings

Lubricate	A743501	N 1 2 3 O F	EA			
Steel (Rep/Rehab)	A744501	N 1 2 3 O F	EA			
Steel (Replace)	B744501	N 1 2 3 O F	EA			
Expansion (Reset)	C744502	N 1 2 3 O F	EA			
Pedestal/Seat (Reconstruct)	D744503	N 1 2 3 O F	EA			

Timber

Stringer (Rep/Repl)	A744601	1 2 3 4 5 O	EA			
Other Members (Rep/Repl)	B744601	1 2 3 4 5 O	EA			

Steel

	Item # (H01)	Location (H03)		Quantity (H05)	PR (H08)	D/C (H09)
Stringer (Rep/Repl)	A744602	1 2 3 4 5 O	EA			
Floorbeam (Rep/Repl)	B744602	1 2 3 4 5 O	EA			
Girder (Repair)	C744602	1 2 3 4 5 O	EA			
Diaph/Lat. Bracing (Rep/Repl)	D744602	1 2 3 4 5 O	EA			

Reinforced, PS, PC, and PT Concrete

Stringer (Rep/Repl)	A744603	1 2 3 4 5 O	EA			
Diaphragm (Rep/Repl)	B744603	1 2 3 4 5 O	EA			
Other Members (Rep/Repl)	C744603	1 2 3 4 5 O	EA			

Truss

Members (Strengthen/Rep/Repl)	A744701	1 2 3 4 5 O	EA			
Portal (Modify)	B744701	1 2 3 4 5 O	EA			
Members(Tighten/Flameshorten)	C744702	1 2 3 4 5 O	EA			

Painting

Superstructure - Spot	A743201	1 2 3 4 5 O	EB	1	4	
Substructure - Spot	B743201	N 1 2 3 O F	EB			
Superstructure - Full	C743201	1 2 3 4 5 O	EB			
Substructure - Full	D743201	N 1 2 3 O F	EB			

Abutment - Wings - Piers

Backwall (Rep/Repl)	A744801	L N R L F R	CY			
Abutments (Repair)	B744802	L N R L F R	CY			
Wing (Rep/Repl)	C744802	L N R L F R	CY			
Piers (Repair)	D744802	1 2 3 4 5 O	CY			
Footing (Underpin)	E744803	N 1 2 3 O F	CY			
Masonry (Repoint)	F744804	N 1 2 3 O F	LF			
Abut Slopewall (Rep/Repl)	A745101	L N R L F R	SY			
Abut Slopewall (Construct New)	B745102	L N R L F R	CY			
Pile Repair	A745901	N 1 2 3 O F	EA			

Scour - Erosion Control

Streambed Paving (Rep/Constr)	A745301	UP UN DN	CY			
Rock Protection	B745301	UP UN DN	CY			
Scour Hole (Backfill)	C745301	UP UN DN	CY			
Stream Deflector (Rep/Constr)	D745302	UP UN DN	CY			
Vegetation/Debris (Remove)	ECREMVG	UP UN DN	CY			
Deposition (Remove)	ECREMDP	UP UN DN	CY			

Culvert

Headwall/Wings (Rep/Repl)	A745201	IN OUT	SY			
Apron/Cutoff Wall (Rep/Repl)	B745202	IN OUT	SY			
Barrel (Repair)	C745203	-- --	SY			

FOR COMPLETION BY REVIEW ENGINEER

Apply Protective Coating

Deck/Parapet/Sidewalk	A743401	DK PARA SW	SY			
Substructure	B743401	N 1 2 3 O F	SY			

Construct Temporary

Support Pier	A745401	N 1 2 3 O F	EA			
Pipe/Culvert Crossing	B745401	LT CL RT	EB			
Bridge	C745401	LT CL RT	EB			

REP.......	Repair	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: ....

Note: The Appendix section for this report is not included here. The BMS 491 Forms for PENNDOT are that state's version of the FHWA SI&A sheet with additional state items. The documents included in the report are typically red marked revisions to the file copy and reflect changes identified during the inspection.

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Appendix B

National Bridge Inspection Standards

Subpart C

National Bridge Inspection Standards

Source: 69 FR 74436, Dec. 14, 2004, unless otherwise noted.

§ 650.301 Purpose.

This subpart sets the national standards for the proper safety inspection and evaluation of all highway bridges in accordance with 23 U.S.C. 151.

§ 650.303 Applicability.

The National Bridge Inspection Standards (NBIS) in this subpart apply to all structures defined as highway bridges located on all public roads.

§ 650.305 Definitions.

Terms used in this subpart are defined as follows:

American Association of State Highway and Transportation Officials (AASHTO) Manual . "The Manual for Bridge Evaluation," First Edition, 2008, published by the American Association of State Highway and Transportation Officials (incorporated by reference, *see* §650.317).

Bridge. A structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening.

Bridge inspection experience. Active participation in bridge inspections in accordance with the NBIS, in either a field inspection, supervisory, or management role. A combination of bridge design, bridge maintenance, bridge construction and bridge inspection experience, with the predominant amount in bridge inspection, is acceptable.

Bridge inspection refresher training. The National Highway Institute "Bridge Inspection Refresher Training Course"[1] or other State, local, or federally developed instruction aimed to improve quality of inspections, introduce new techniques, and maintain the consistency of the inspection program. [1] The National Highway Institute training may be found at the following URL: *http://www.nhi.fhwa.dot.gov./*

Bridge Inspector's Reference Manual (BIRM). A comprehensive FHWA manual on programs, procedures and techniques for inspecting and evaluating a variety of in-service highway bridges. This manual may be purchased from the U.S. Government Printing Office, Washington, DC 20402 and from National Technical Information Service, Springfield, Virginia 22161, and is available at the following URL: *http://www.fhwa.dot.gov/bridge/bripub.htm.*

Complex bridge . Movable, suspension, cable stayed, and other bridges with unusual characteristics.

Comprehensive bridge inspection training. Training that covers all aspects of bridge inspection and enables inspectors to relate conditions observed on a bridge to established criteria (see the

Bridge Inspector's Reference Manual for the recommended material to be covered in a comprehensive training course).

Critical finding. A structural or safety related deficiency that requires immediate follow-up inspection or action.

Damage inspection. This is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions.

Fracture critical member (FCM). A steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse.

Fracture critical member inspection. A hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation.

Hands-on. Inspection within arms length of the component. Inspection uses visual techniques that may be supplemented by nondestructive testing.

Highway. The term “highway” is defined in 23 U.S.C. 101(a)(11).

In-depth inspection. A close-up, inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures; hands-on inspection may be necessary at some locations.

Initial inspection. The first inspection of a bridge as it becomes a part of the bridge file to provide all Structure Inventory and Appraisal (SI&A) data and other relevant data and to determine baseline structural conditions.

Legal load. The maximum legal load for each vehicle configuration permitted by law for the State in which the bridge is located.

Load rating. The determination of the live load carrying capacity of a bridge using bridge plans and supplemented by information gathered from a field inspection.

National Institute for Certification in Engineering Technologies (NICET). The NICET provides nationally applicable voluntary certification programs covering several broad engineering technology fields and a number of specialized subfields. For information on the NICET program certification contact: National Institute for Certification in Engineering Technologies, 1420 King Street, Alexandria, VA 22314–2794.

Operating rating. The maximum permissible live load to which the structure may be subjected for the load configuration used in the rating.

Professional engineer (PE). An individual, who has fulfilled education and experience requirements and passed rigorous exams that, under State licensure laws, permits them to offer engineering services directly to the public. Engineering licensure laws vary from State to State, but, in general, to become a PE an individual must be a graduate of an engineering program accredited by the Accreditation Board for Engineering and Technology, pass the Fundamentals of Engineering exam, gain four years of experience working under a PE, and pass the Principles of Practice of Engineering exam.

Program manager. The individual in charge of the program, that has been assigned or delegated the duties and responsibilities for bridge inspection, reporting, and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance.

Public road. The term “public road” is defined in 23 U.S.C. 101(a)(27).

Quality assurance (QA). The use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program.

Quality control (QC). Procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level.

Routine inspection. Regularly scheduled inspection 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 requirements.

Routine permit load. A live load, which has a gross weight, axle weight or distance between axles not conforming with State statutes for legally configured vehicles, authorized for unlimited trips over an extended period of time to move alongside other heavy vehicles on a regular basis.

Scour. Erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges.

Scour critical bridge. A bridge with a foundation element that has been determined to be unstable for the observed or evaluated scour condition.

Special inspection. An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency.

State transportation department. The term “State transportation department” is defined in 23 U.S.C. 101(a)(34).

Team leader. Individual in charge of an inspection team responsible for planning, preparing, and performing field inspection of the bridge.

Underwater diver bridge inspection training. Training that covers all aspects of underwater bridge inspection and enables inspectors to relate the conditions of underwater bridge elements to established criteria (see the Bridge Inspector's Reference Manual section on underwater inspection for the recommended material to be covered in an underwater diver bridge inspection training course).

Underwater inspection. Inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.

[69 FR 74436, Dec. 14, 2004, as amended at 74 FR 68379, Dec. 24, 2009]

§ 650.307 Bridge inspection organization.

(a) Each State transportation department must inspect, or cause to be inspected, all highway bridges located on public roads that are fully or partially located within the State's boundaries, except for bridges that are owned by Federal agencies.

(b) Federal agencies must inspect, or cause to be inspected, all highway bridges located on public roads that are fully or partially located within the respective agency responsibility or jurisdiction.

(c) Each State transportation department or Federal agency must include a bridge inspection organization that is responsible for the following:

(1) Statewide or Federal agency wide bridge inspection policies and procedures, quality assurance and quality control, and preparation and maintenance of a bridge inventory.

(2) Bridge inspections, reports, load ratings and other requirements of these standards.

(d) Functions identified in paragraphs (c)(1) and (2) of this section may be delegated, but such delegation does not relieve the State transportation department or Federal agency of any of its responsibilities under this subpart.

(e) The State transportation department or Federal agency bridge inspection organization must have a program manager with the qualifications defined in §650.309(a), who has been delegated responsibility for paragraphs (c)(1) and (2) of this section.

§ 650.309 Qualifications of personnel.

(a) A program manager must, at a minimum:

(1) Be a registered professional engineer, or have ten years bridge inspection experience; and

(2) Successfully complete a Federal Highway Administration (FHWA) approved comprehensive bridge inspection training course.

(b) There are five ways to qualify as a team leader. A team leader must, at a minimum:

(1) Have the qualifications specified in paragraph (a) of this section; or

(2) Have five years bridge inspection experience and have successfully completed an FHWA approved comprehensive bridge inspection training course; or

(3) Be certified as a Level III or IV Bridge Safety Inspector under the National Society of Professional Engineer's program for National Certification in Engineering Technologies (NICET) and have successfully completed an FHWA approved comprehensive bridge inspection training course, or

(4) Have all of the following:

(i) A bachelor's degree in engineering from a college or university accredited by or determined as substantially equivalent by the Accreditation Board for Engineering and Technology;

(ii) Successfully passed the National Council of Examiners for Engineering and Surveying Fundamentals of Engineering examination;

(iii) Two years of bridge inspection experience; and

(iv) Successfully completed an FHWA approved comprehensive bridge inspection training course, or

(5) Have all of the following:

(i) An associate's degree in engineering or engineering technology from a college or university accredited by or determined as substantially equivalent by the Accreditation Board for Engineering and Technology;

(ii) Four years of bridge inspection experience; and

(iii) Successfully completed an FHWA approved comprehensive bridge inspection training course.

(c) The individual charged with the overall responsibility for load rating bridges must be a registered professional engineer.

(d) An underwater bridge inspection diver must complete an FHWA approved comprehensive bridge inspection training course or other FHWA approved underwater diver bridge inspection training course.

§ 650.311 Inspection frequency.

(a) *Routine inspections.* (1) Inspect each bridge at regular intervals not to exceed twenty-four months.

(2) Certain bridges require inspection at less than twenty-four-month intervals. Establish criteria to determine the level and frequency to which these bridges are inspected considering such factors as age, traffic characteristics, and known deficiencies.

(3) Certain bridges may be inspected at greater than twenty-four month intervals, not to exceed forty-eight-months, with written FHWA approval. This may be appropriate when past inspection findings and analysis justifies the increased inspection interval.

(b) *Underwater inspections.* (1) Inspect underwater structural elements at regular intervals not to exceed sixty months.

(2) Certain underwater structural elements require inspection at less than sixty-month intervals. Establish criteria to determine the level and frequency to which these members are inspected considering such factors as construction material, environment, age, scour characteristics, condition rating from past inspections and known deficiencies.

(3) Certain underwater structural elements may be inspected at greater than sixty-month intervals, not to exceed seventy-two months, with written FHWA approval. This may be appropriate when past inspection findings and analysis justifies the increased inspection interval.

(c) *Fracture critical member (FCM) inspections.* (1) Inspect FCMs at intervals not to exceed twenty-four months.

(2) Certain FCMs require inspection at less than twenty-four-month intervals. Establish criteria to determine the level and frequency to which these members are inspected considering such factors as age, traffic characteristics, and known deficiencies.

(d) Damage, in-depth, and special inspections. Establish criteria to determine the level and frequency of these inspections.

§ 650.313 Inspection procedures.

(a) Inspect each bridge in accordance with the inspection procedures in the AASHTO Manual (incorporated by reference, *see* §650.317).

(b) Provide at least one team leader, who meets the minimum qualifications stated in §650.309, at the bridge at all times during each initial, routine, in-depth, fracture critical member and underwater inspection.

(c) Rate each bridge as to its safe load-carrying capacity in accordance with the AASHTO Manual (incorporated by reference, *see* §650.317). Post or restrict the bridge in accordance with the AASHTO Manual or in accordance with State law, when the maximum unrestricted legal loads or State routine permit loads exceed that allowed under the operating rating or equivalent rating factor.

(d) Prepare bridge files as described in the AASHTO Manual (incorporated by reference, *see* §650.317). Maintain reports on the results of bridge inspections together with notations of any action taken to address the findings of such inspections. Maintain relevant maintenance and inspection data to allow assessment of current bridge condition. Record the findings and results of bridge inspections on standard State or Federal agency forms.

(e) Identify bridges with FCMs, bridges requiring underwater inspection, and bridges that are scour critical.

(1) Bridges with fracture critical members. In the inspection records, identify the location of FCMs and describe the FCM inspection frequency and procedures. Inspect FCMs according to these procedures.

(2) Bridges requiring underwater inspections. Identify the location of underwater elements and include a description of the underwater elements, the inspection frequency and the procedures in the inspection records for each bridge requiring underwater inspection. Inspect those elements requiring underwater inspections according to these procedures.

(3) Bridges that are scour critical. Prepare a plan of action to monitor known and potential deficiencies and to address critical findings. Monitor bridges that are scour critical in accordance with the plan.

(f) *Complex bridges.* Identify specialized inspection procedures, and additional inspector training and experience required to inspect complex bridges. Inspect complex bridges according to those procedures.

(g) *Quality control and quality assurance.* Assure systematic quality control (QC) and quality assurance (QA) procedures are used to maintain a high degree of accuracy and consistency in the inspection program. Include periodic field review of inspection teams, periodic bridge inspection refresher training for program managers and team leaders, and independent review of inspection reports and computations.

(h) *Follow-up on critical findings.* Establish a statewide or Federal agency wide procedure to assure that critical findings are addressed in a timely manner. Periodically notify the FHWA of the actions taken to resolve or monitor critical findings.

§ 650.315 Inventory.

(a) Each State or Federal agency must prepare and maintain an inventory of all bridges subject to the NBIS. Certain Structure Inventory and Appraisal (SI&A) data must be collected and retained by the State or Federal agency for collection by the FHWA as requested. A tabulation of this data is contained in the SI&A sheet distributed by the FHWA as part of the “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges,” (December 1995) together with subsequent interim changes or the most recent version. Report the data using FHWA established procedures as outlined in the “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.”

(b) For routine, in-depth, fracture critical member, underwater, damage and special inspections enter the SI&A data into the State or Federal agency inventory within 90 days of the date of inspection for State or Federal agency bridges and within 180 days of the date of inspection for all other bridges.

(c) For existing bridge modifications that alter previously recorded data and for new bridges, enter the SI&A data into the State or Federal agency inventory within 90 days after the completion of the work for State or Federal agency bridges and within 180 days after the completion of the work for all other bridges.

(d) For changes in load restriction or closure status, enter the SI&A data into the State or Federal agency inventory within 90 days after the change in status of the structure for State or Federal agency bridges and within 180 days after the change in status of the structure for all other bridges.

§ 650.317 Reference manuals.

(a) The materials listed in this subpart are incorporated by reference in the corresponding sections noted. These incorporations by reference were approved by the Director of the Federal Register in accordance with 5 U.S.C. 552(a) and 1 CFR part 51. These materials are incorporated as they exist on the date of the approval, and notice of any change in these documents will be published in the Federal Register. The materials are available for purchase at the address listed below, and are available for inspection at the National Archives and Records Administration (NARA). These materials may also be reviewed at the Department of Transportation Library, 1200 New Jersey Avenue, SE., Washington, DC 20590, (202) 366–0761. For information on the availability of these materials at NARA call (202) 741–6030, or go to the following URL: *http://www.archives.gov/federal_register/code_of_federal_regulations/ibr_locations.htm* . In the event there is a conflict between the standards in this subpart and any of these materials, the standards in this subpart will apply.

(b) The following materials are available for purchase from the American Association of State Highway and Transportation Officials, Suite 249, 444 N. Capitol Street, NW., Washington, DC 20001, (202) 624–5800. The materials may also be ordered via the AASHTO bookstore located at the following URL: *http://www.transportation.org* .

(1) The Manual for Bridge Evaluation, First Edition, 2008, AASHTO, incorporation by reference approved for §§650.305 and 650.313.

(2) [Reserved]

[74 FR 68379, Dec. 24, 2009]

Glossary

GLOSSARY

A

AASHTO - American Association of State Highway and Transportation Officials, name changed from AASHO (American Association of State Highway Officials) in 1973

***American Association of State Highway and Transportation Officials (AASHTO) Manual* -** “Manual for Condition Evaluation of Bridges,” second edition, published by the American Association of State Highway and Transportation Officials (incorporated by reference into 23 CFR 650)

abrasion - wearing or grinding away of material by friction; usually caused by sand, gravel, or stones, carried by wind or water

absorption - the process of a liquid being taken into a permeable solid (e.g., the wetting of concrete)

abutment - part of bridge substructure at either end of bridge which transfers loads from superstructure to foundation and provides lateral support for the approach roadway embankment

ADT - Average Daily Traffic

ADTT - Average Daily Truck Traffic

admixture - an ingredient added to concrete other than cement, aggregate or water (e.g., air entraining agent)

aggradation - progressive raising of a streambed by deposition of sediment

aggregate - hard inert material such as sand, gravel, or crushed rock that may be combined with a cementing material to form mortar or concrete

air entrainment - the addition of air into a concrete mixture in order to increase the durability and resist thermal forces

alkali silica reactivity (ASR) - an expansive reaction that results in swelling and expansion of concrete.

alignment - the relative horizontal and vertical positioning between components, such as the bridge and its approaches

alignment bearing - a bearing embedded in a bridge seat to prevent lateral movements (see BEARING)

alligator cracking - cracks initiated by inadequate base support or drainage that form on the surface of a road in adjacent, rectangular shapes (like the skin of an alligator)

alloy - two or more metals, or metal and non-metal, intimately combined, usually by dissolving together in a molten state to form a new base metal

anchorage - the complete assemblage of members and parts, embedded in concrete, rock or other fixed material, designed to hold a portion of a structure in correct position

anchor bolt - a metal rod or bar commonly threaded and fitted with a nut and washer at one end only, used to secure in a fixed position upon the substructure the bearings of a bridge, the base of a column, a pedestal, shoe, or other member of a structure

anchor span - the span that counterbalances and holds in equilibrium the cantilevered portion of an adjacent span; also called the back span; see CANTILEVER BEAM, GIRDER, or TRUSS

angle - a basic member shape, usually steel, in the form of an "L"

anisotropy - the property of certain materials, such as crystals, that exhibits different strengths in different directions

anode - the positively charged pole of a corrosion cell at which oxidation occurs

anti-friction bearing - a ball or roller-type bearing; a bearing that reduces transfer of horizontal loads between components

appraisal rating - a judgment of a bridge component's adequacy in comparison to current standards

approach - the part of the roadway immediately before and after the bridge structure

approach pavement - an approach which has a cross section that is either the same as or slightly wider than the bridge deck width

approach slab - a reinforced concrete slab placed on the approach embankment adjacent to and usually resting upon the abutment back wall; the function of the approach slab is to carry wheel loads on the approaches directly to the abutment, thereby transitioning any approach roadway misalignment due to approach embankment settlement

appurtenance - an element that contributes to the general functionality of the bridge site (e.g., lighting, signing)

apron - a form of scour (erosion) protection consisting of timber, concrete, riprap, paving, or other construction material placed adjacent to abutments and piers to prevent undermining

arch - a curved structure element primarily in compression that transfers vertical loads through inclined reactions to its end supports

arch barrel - a single arch member that extends the width of the structure

arch rib - the main support element used in open spandrel arch construction; also known as arch ring

armor - a secondary steel member installed to protect a vulnerable part of another member, e.g., steel angles placed over the edges of a joint; also scour protection such as rip rap

armoring countermeasures - devices that resist erosive forces caused by the flow, but do not alter the flow direction.

as-built plans - plans made after the construction of a project, showing all field changes to the final design plans (i.e.. showing how the bridge was actually built)

asphalt - a brown to black bituminous substance that is found in natural beds and is also obtained as a residue in petroleum refining and that consists chiefly of hydrocarbons; an asphaltic composition used for pavements and as a waterproof cement

ASTM - American Society for Testing and Materials

auger - a drill with a spiral channel used for boring

axial - in line with the longitudinal axis of a member

axial force - the force that acts through the longitudinal axis of a member.

axle load - the load borne by one axle of a traffic vehicle, a movable bridge, or other motive equipment or device and transmitted through a wheel or wheels

B

back - see EXTRADOS

backfill - material, usually soil or coarse aggregate, used to fill the unoccupied portion of a substructure excavation such as behind an abutment stem and backwall

backstay - cable or chain attached at the top of a tower and extending to and secured upon the anchorage to resist overturning stresses exerted upon the tower by a suspended span

backwall - the topmost portion of an abutment above the elevation of the bridge seat, functioning primarily as a retaining wall with a live load surcharge; it may serve also as a support for the extreme end of the bridge deck and the approach slab

backwater - the back up of water in a stream due to a downstream obstruction or constriction

bank - sloped sides of a waterway channel or approach roadway, short for embankment

bascule bridge - a bridge over a waterway with one or two leaves which rotate from a horizontal to a near-vertical position, providing unlimited overhead clearance

base course - a layer of compacted material found just below the wearing course that supports the pavement

base metal - the surface metal of a steel element to be incorporated in a welded joint; also known as structure metal, parent metal

base plate - steel plate, whether cast, rolled or forged, connected to a column, bearing or other member to transmit and distribute its load to the substructure

batten plate - a plate with two or more fasteners at each end used in lieu of lacing to tie together the shapes comprising a built-up member

batter - the inclination of a surface in relation to a horizontal or a vertical plane; commonly designated on bridge detail plans as a ratio (e.g., 1:3, H:V); see RAKE

battered pile - a pile driven in an inclined position to resist horizontal forces as well as vertical forces

bay - the area of a bridge floor system between adjacent multi-beams or between adjacent floor beams

beam - a linear structural member designed to span from one support to another and support vertical loads

bearing - a support element transferring loads from superstructure to substructure while permitting limited movement capability

bearing capacity - the load per unit area which a structural material, rock, or soil can safely carry

bearing failure - crushing of material under extreme compressive load

bearing pile - a pile which provides support through the tip (or lower end) of the pile

bearing plate - a steel plate, which transfers loads from the superstructure to the substructure

bearing pressure - the bearing load divided by the area to which it is applied

bearing seat - a prepared horizontal surface at or near the top of a substructure unit upon which the bearings are placed

bearing stiffener - a vertical web stiffener at the bearing location

bearing stress - see BEARING PRESSURE

bedding - the soil or backfill material used to support pipe culverts

bedrock - the undisturbed rock layer below the surface soil

bench mark - an established reference point with known elevation and coordinates, used to document dimensions, elevations or position movement

bending moment - a combination of tension and compression forces developed when an external load is applied transversely to a bridge member, causing it to bend

bent - a substructure unit made up of two or more column or column-like members connected at their top-most ends by a cap, strut, or other member holding them in their correct positions

berm - the line that defines the location where the top surface of an approach embankment or causeway is intersected by the surface of the side slope

beveled washer - a wedge-shaped washer used in connections incorporating members with sloped flange legs, e.g., channels and S-beams

bitumen - a black sticky mixture of hydrocarbons obtained from natural deposits or from distilling petroleum; tar

bituminous concrete - a mixture of aggregate and liquid asphalt or bitumen, which is compacted into a dense mass

blanket - a streambed protection against scour placed adjacent to abutments and piers

BMS - Bridge Management System

bolt -a mechanical fastener with machine threads at one end to receive a nut, and an integral head at the other end

bolster - a block-like member used to support a bearing on top of a pier cap or abutment bridge seat; see PEDESTAL

bond - in reinforced concrete, the grip of the concrete on the reinforcing bars, which prevents slippage of the bars relative to the concrete mass

bond stress - a term commonly applied in reinforced concrete construction to the stress developed by a force tending to produce movement or slippage at the interface between the concrete and the reinforcement bars

bowstring truss - a general term applied to a truss of any type having a polygonal arrangement of its top chord members conforming to or nearly conforming to the arrangement required for a parabolic truss; a truss with a curved top chord

box beam - a hollow structural beam with a square, rectangular, or trapezoidal cross-section that supports vertical loads and provides torsional rigidity

box culvert - a culvert of rectangular or square cross-section

box girder - a hollow, rectangular or trapezoidal shaped girder, a primary member along the longitudinal axis of the bridge, which provides good torsional rigidity

bracing - a system of secondary members that maintains the geometric configuration of primary members

bracket - a projecting support fixed upon two intersecting members to strengthen and provide rigidity to the connection

breastwall - the portion of an abutment between the wings and beneath the bridge seat; the breast wall supports the superstructure loads, and retains the approach fill; see STEM

bridge - a structure including supports erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the center of the roadway of more than 20 feet between undercopings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it may also include multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening

bridge deficiency - a defect in a bridge component or member that makes the bridge less capable or less desirable for use

bridge elements - structural elements that are commonly used in highway bridge construction and are encountered on bridge safety inspections

bridge inspection experience - active participation in bridge inspections in accordance with the NBIS, in either a field inspection, supervisory, or management role. A combination of bridge design, bridge maintenance, bridge construction and bridge inspection experience, with the predominant amount in bridge inspection, is acceptable.

bridge inspection refresher training - the National Highway Institute "Bridge Inspection Refresher Training Course" [1] or other State, local, or federally developed instruction aimed to improve quality of inspections, introduce new techniques, and maintain the consistency of the inspection program.

[1] The National Highway Institute training may be found at the following URL: *http://www.nhi.fhwa.dot.gov./*

***Bridge Inspector's Reference Manual (BIRM)* -** a comprehensive FHWA manual on programs, procedures and techniques for inspecting and evaluating a variety of in-service highway bridges. This manual may be purchased from the U.S. Government Printing Office, Washington, DC 20402 and from National Technical Information Service, Springfield, Virginia 22161, and is available at the following URL: *http://www.fhwa.dot.gov/bridge/bripub.htm.*

bridge pad - the raised, leveled area upon which the pedestal, masonry plate or other corresponding element of the superstructure bears on the substructure; also called bridge seat bearing area

bridge seat - the top surface of an abutment or pier upon which the superstructure span is placed and supported; for an abutment it is the surface forming the support for the superstructure and from which the backwall rises; for a pier it is the entire top surface

bridge site - the position or location of a bridge and its surrounding area

bridging - a carpentry term applied to the crossbracing fastened between timber beams to increase the rigidity of the floor construction, limit differential deflection and minimize the effects of impact and vibration

brittle fracture - the failure of a steel member occurring without warning, prior to plastic deformation

brittleness - the ability of a material to break while exhibiting little to no plastic deformation

brush curb - a narrow curb, 9 inches or less in width, which prevents a vehicle from brushing against the railing or parapet

buckle - to fail by an inelastic change in alignment (deflection) as a result of compression in axial loaded members

buckle plate - an obsolete style of steel deck using dished steel plates as structural members

built-up member - a column or beam composed of plates and angles or other structural shapes united by bolting, riveting or welding to enhance section properties

bulb t-girder - a t-shaped concrete girder with a bulb shape at the bottom of the girder cross section

bulkhead - a retaining wall-like structure commonly composed of driven sheet piles or a barrier of wooden timbers or reinforced concrete members

buoyancy - upward pressure exerted by the fluid in which an object is immersed

butt joint - a joint between two pieces of metal that have been connected in the same plane

buttress - a bracket-like wall, of full or partial height, projecting from another wall; the buttress strengthens and stiffens the wall against overturning forces; all parts of a buttress act in compression

buttressed wall - a retaining wall designed with projecting buttresses to provide strength and stability

butt weld - a weld joining two plates or shapes end to end; also splice weld

C

cable - a tension member comprised of numerous individual steel wires or strands twisted and wrapped in such a fashion to form a rope of steel; see SUSPENSION BRIDGE

cable band - a steel casting with clamp bolts which fixes a floor system suspender cable to the catenary cable of a suspension bridge

cable-stayed bridge - a bridge in which the superstructure is directly supported by cables, or stays, passing over or attached to towers located at the main piers

caddisfly - a winged insect closely related to the moth and butterfly whose aquatic larvae seek shelter by digging small shallow holes into submerged timber elements

caisson - a rectangular or cylindrical chamber for keeping water or soft ground from flowing into an excavation

camber - the slightly arched or convex curvature provided in beams to compensate for dead load deflection; in general, a structure built with perfectly straight lines appears slightly sagged

cantilever - a structural member that has a free end projecting beyond a support; length of span overhanging the support

cantilever abutment - an abutment that resists lateral earth pressure through the opposing cantilever action of a vertical stem and horizontal footing

cantilever bridge - a general term applying to a bridge having a superstructure incorporating cantilever design

cantilever span - a superstructure span composed of two cantilever arms, or of a suspended span supported by one or two cantilever arms

cap - the topmost portion of a pier or a pile bent serving to distribute the loads upon the columns or piles and to hold them in their proper relative positions; see PIER CAP, PILE CAP

cap beam - the top member in a bent that ties together the supporting members

capstone - the topmost stone of a masonry pillar, column or other structure requiring the use of a single capping element

carbon steel - steel (iron with dissolved carbon) owing its properties principally to its carbon content; ordinary, unalloyed steel

cast-in-place (C.I.P.) - the act of placing and curing concrete within formwork to construct a concrete element in its final position

cast iron - relatively pure iron, smelted from iron ore, containing 1.8 to 4.5% free carbon and cast to shape

catch basin - a receptacle, commonly box shaped and fitted with a grilled inlet and a pipe outlet drain, designed to collect the rainwater and floating debris from the roadway surface and retain the solid material so that it may be periodically removed

catchment area - see DRAINAGE AREA

catenary - the curve obtained by suspending a uniformly loaded rope or cable between two points

cathode - the negatively charged pole of a corrosion cell that accepts electrons and does not corrode

cathodic protection - a means of preventing metal from corroding by making it a cathode through the use of impressed direct current or by attaching a sacrificial anode

catwalk - a narrow walkway for access to some part of a structure

causeway - an elevated roadway crossing a body of water

cellular abutment - an abutment in which the space between wings, abutment stem, approach slab, and footings is hollow. Also known as a vaulted abutment

cement mortar - a mixture of sand and cement with enough water to make it plastic

cement paste - the plastic combination of cement and water that supplies the cementing action in concrete

centerline of bearings - a horizontal line that passes through the centers of the bearings, used in abutment/pier layout and beam erection

center of gravity - the point at which the entire mass of a body acts; the balancing point of an object

centroid - that point about which the static moment of all the elements of area is equal to zero

chain drag - a chain or a series of short medium weight chains attached to a T-shaped handle; used as a preliminary technique for sounding a large deck area for delamination

chamfer - an angled edge or corner, typically formed in concrete

channel - a waterway connecting two bodies of water or containing moving water; a rolled steel member having a C-shaped cross section

channel lining - rigid concrete pavement or flexible protective revetment mats placed on the bottom of a streambed

channel profile - a longitudinal section of a channel along its centerline

checks - a crack in wood occurring parallel with the grain and through the rings of annual growth

cheek wall - see KNEE WALL

chipping hammer - hammer such as a geologist's pick or masonry hammer used to remove corrosion from steel members and to sound concrete for delamination; a welder's tool for cleaning slag from steel after welding

chloride - an ingredient in deicing agents that can damage concrete and steel bridge elements

chloride contamination - the presence of recrystalized soluble salts, which causes accelerated corrosion of the steel reinforcement

chord - a generally horizontal member of a truss

circular arch - an arch in which the intrados surface has a constant radius

clearance - the unobstructed vertical or horizontal space provided between two objects

clear headroom - the vertical clearance beneath a bridge structure available for navigational use

clear span - the unobstructed space or distance between support elements of a bridge or bridge member

clip angle - see CONNECTION ANGLE

closed spandrel arch - a stone, brick or reinforced concrete arch span having spandrel walls to retain the spandrel fill or to support either entirely or in part the floor system of the structure when the spandrel is not filled

coarse aggregate - aggregate that stays on a sieve of 5 mm (¼") square opening

coating - a material that provides a continuous film over a surface in order to protect or seal it; a film formed by the material

coefficient of thermal expansion - the unit change in dimension produced in a material by a change of one degree in temperature

cofferdam - a temporary dam-like structure constructed around an excavation to exclude water; see SHEET PILE COFFERDAM

cold chisel - short bar with a sharp end used for cold-cutting soft metals when struck with a hammer

collision damage - a special case of overload that occurs when any vehicle, railroad car, marine traffic or flowing ice strikes a bridge member, railing or column

column - a general term applying to a vertical member resisting compressive stresses and having, in general, a considerable length in comparison with its transverse dimensions

column bent - a bent shaped pier that uses columns incorporated with a cap beam

compaction - the process by which a sufficient amount of energy (compressive pressure) is applied to soil or other material to increase its density

complex bridge - movable, suspension, cable stayed, and other bridges with unusual characteristics

component - a general term reserved to define a bridge deck, superstructure or substructure

composite action - the contribution of a concrete deck to the moment resisting capacity of the superstructure beam when the superstructure beams are not the same material as the deck

composite construction - a method of construction whereby a cast-in-place concrete deck is mechanically attached to superstructure members by shear connectors

comprehensive bridge inspection training - training that covers all aspects of bridge inspection and enables inspectors to relate conditions observed on a bridge to established criteria (see the Bridge Inspector's Reference Manual for the recommended material to be covered in a comprehensive training course).

compression - a type of stress involving pressing together; tends to shorten a member; opposite of tension

compression failure - buckling, crushing, or collapse caused by compression stress

compression flange - the part of a beam that is compressed due to a bending moment

compression seal joint - a joint consisting of a neoprene elastic seal squeezed into the joint opening

concentrated load - a force applied over a small contact area; also known as point load

concrete - a stone-like mass made from a mixture of aggregates and cementing material, which is moldable prior to hardening; see BITUMINOUS CONCRETE and PORTLAND CEMENT CONCRETE

concrete beam - a structural member of reinforced concrete designed to carry bending loads

concrete pile - a pile constructed of reinforced concrete either precast and driven into the ground or cast-in-place in a hole bored into the ground

concrete tee team - "T" shaped section of reinforced concrete; cast-in-place monolithic deck and beam system

condition rating - a judgment of a bridge component condition in comparison to its original as-built condition

conductor - a material that is suitable for carrying electric current

connection angle - a piece of angle serving to connect two elements of a member or two members of a structure; also known as clip angle

consolidation - the time dependent change in volume of a soil mass under compressive load caused by water slowly escaping from the pores or voids of the soil

construction joint - a pair of adjacent surfaces in reinforced concrete where two pours have met, reinforcement steel extends through this joint

continuous beam - a general term applied to a beam that spans uninterrupted over one or more intermediate supports

continuous bridge - a bridge designed to extend without joints over one or more interior supports

continuous footing - a common footing that is underneath a wall, or columns

continuous span - spans designed to extend without joints over one or more intermediate supports

continuous truss - a truss without hinges having its chord and web members arranged to continue uninterrupted over one or more intermediate points of support

continuous weld - a weld extending throughout the entire length of a connection

contraction - the thermal action of the shrinking of an object when cooled; opposite of expansion

contraction scour - the removal of the material under the structure only

coping - a course of stone laid with a projection beyond the general surface of the masonry below it and forming the topmost portion of a wall; a course of stone capping the curved or V-shaped extremity of a pier, providing a transition to the pier head proper, when so used it is commonly termed the "starling coping," "nose coping," the "cutwater coping" or the "pier extension coping"

corbel - a piece constructed to project from the surface of a wall, column or other portion of a structure to serve as a support for another member

core - a cylindrical sample of concrete or timber removed from a bridge component for the purpose of destructive testing to determine the condition of the component

corrosion - the general disintegration of metal through oxidation

corrugated - an element with alternating ridges and valleys

counter - a truss web member that undergoes stress reversal and resists only live load tension; see WEB MEMBERS

counterfort - a bracket-like wall connecting a retaining wall stem to its footing on the side of the retained material to stabilize the wall against overturning; a counterfort, as opposed to a buttress, acts entirely in tension

counterforted abutment - an abutment that develops resistance to bending moment in the stem by use of counterforts. This permits the breast wall to be designed as a horizontal beam or slab spanning between counterforts, rather than as a vertical cantilever slab

counterforted wall - a retraining wall designed with projecting counterforts to provide strength and stability

counterweight - a weight which is used to balance the weight of a movable member; in bridge applications counterweights are used to balance a movable span so that it rotates or lifts with minimum resistance. Also sometimes used in continuous structures to prevent uplift

couplant - a viscous fluid material used with ultrasonic gages to enhance transmission of sound waves

couple - two forces that are equal in magnitude, opposite in direction, and parallel with respect to each other

coupon - a sample of steel taken from an element in order to test material properties

course - a horizontal layer of bricks or stone

cover - the clear thickness of concrete between a reinforcing bar and the surface of the concrete; the depth of backfill over the top of a pipe or culvert

covered bridge - an indefinite term applied to a wooden bridge having its roadway protected by a roof and enclosing sides

cover plate - a plate used in conjunction with a flange or other structural shapes to increase flange section properties in a beam, column, or similar member

crack - a break without complete separation of parts; a fissure

crack comparator card - A crack comparator 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 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.

cracking (reflection) - visible cracks in an overlay indicating cracks in the concrete underneath

crack initiation - the beginning of a crack usually at some microscopic defect

crack propagation - the growth of a crack due to energy supplied by repeated stress cycles

creep - an inelastic deformation that occurs under a constant load, below the yield point, and increases with time

creosote - an oily liquid obtained by the distillation of coal or wood tar and used as a wood preservative

crevice corrosion - occurs between adjacent surfaces but the rust may not expand, even though significant section loss may have occurred

crib - a structure consisting of a foundation grillage combined with a superimposed framework providing compartments or coffers which are filled with gravel, concrete or other material satisfactory for supporting the structure to be placed thereon

cribbing - a construction consisting of wooden, metal or reinforced concrete units so assembled as to form an open cellular-like structure for supporting a superimposed load or for resisting horizontal or overturning forces acting against it.

cribwork - large timber cells that are submerged full of concrete to make an underwater foundation

critical finding - a structural or safety related deficiency that requires immediate follow-up inspection or action

cross - transverse bracings between two main longitudinal members; see DIAPHRAGM, BRACING

cross frame - steel elements placed in "X" shaped patterns to act as stiffeners between the main carrying superstructure members

cross girders - transverse girders, supported by bearings, which support longitudinal beams or girders

cross-section - the shape of an object cut transversely to its length

cross-sectional area - the area of a cross-section

crown - the highest point of the transverse cross section of a roadway, pipe or arch; also known as soffit or vertex

crown of roadway - the vertical dimension describing the total amount the surface is convexed or raised from gutter to centerline; this is sometimes termed the cross fall or cross slope of roadway

crushing - occurs perpendicular to the grain, usually at support points

culvert - a drainage structure beneath an embankment (e.g., corrugated metal pipe, concrete box culvert)

curb - a low barrier at the side limit of the roadway used to guide the movement of vehicles

curb inlet - see SCUPPER

curtain wall - a term commonly applied to a thin wall between main columns designed to withstand only secondary loads. Also the wall portion of a buttress or counterfort abutment that spans between the buttresses or counterforts

curvature - the degree of curving of a line or surface

curved girder - a girder that is curved in the horizontal plane in order to adjust to the horizontal alignment of the bridge

cutoff wall - vertical wall at the end of an apron or slab to prevent scour undermining

cutwater - a sharp-edged structure, facing the water channel current, built around a bridge pier to protect if from the flow of water and debris in the water

cyclic stress - stress that varies with the passage of live loads; see STRESS RANGE

D

damage inspection - this is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions

dead load - a static load due to the weight of the structure itself

debris - material including floating wood, trash, suspended sediment or bed load moved by a flowing stream

decay - the result of fungi feeding on the cell walls of the wood

deck - that portion of a bridge which provides direct support for vehicular and pedestrian traffic, supported by a superstructure

deck arch - an arch bridge with the deck above the top of the arch

deck bridge - a bridge in which the supporting members are all beneath the roadway

decking - bridge flooring installed in panels, e.g., timber planks

deck joint - a gap allowing for rotation or horizontal movement between two spans or an approach and a span

deficiency - see BRIDGE DEFICIENCY

deflection - elastic movement of a structural member under a load

deformation - distortion of a loaded structural member; may be elastic or inelastic

deformed bars - concrete reinforcement consisting of steel bars with projections or indentations (deformations) to increase the mechanical bond between the steel and concrete

degradation - general progressive lowering of a stream channel by scour

delamination - surface separation of concrete into layers; separation of glulaminated timber plies

design load - the force for which a structure is designed; the most severe combination of loads

distributed loads - loads that are applied along a significant length of a structure

deterioration - decline in quality over a period of time due to chemical or physical degradation

diagonal - a sloping structural member of a truss or bracing system

diagonal stay - a cable support in a suspension bridge extending diagonally from the tower to the roadway to add stiffness to the structure and diminish the deformations and undulations resulting from traffic service

diagonal tension - the tensile force due to horizontal and vertical shear in a beam

diaphragm - a transverse member placed within a member or superstructure system to distribute stresses and improves strength and rigidity; see BRACING

diaphragm wall - a wall built transversely to the longitudinal centerline of a spandrel arch serving to tie together and reinforce the spandrel walls, together with providing a support for the floor system in conjunction with the spandrel walls; also known as cross wall

differential settlement - uneven settlement of individual or independent elements of a substructure; tilting in the longitudinal or transverse direction due to deformation or loss of foundation material

dike - an earthen embankment constructed to retain or redirect water; when used in conjunction with a bridge, it prevents stream erosion and localized scour andor so directs the stream current such that debris does not accumulate; see SPUR

discharge - the volume of fluid per unit of time flowing along a pipe or channel

displacement induced stress - stresses caused by differential deflection of adjacent parts

distributed load - a load uniformly applied along the length of an element or component of a bridge

ditch - a trough-like excavation made to collect water

diver - a specially trained individual who inspects the underwater portion of a bridge substructure and the surrounding channel

dolphin - a group of piles driven close together or a caisson placed to protect portions of a bridge exposed to possible damage by collision with river or marine traffic

double movable bridge - a bridge in which the clear span over the navigation channel is produced by joining the arms of two adjacent swing spans or the leaves of two adjacent bascule spans at or near the center of the navigable channel; see MOVABLE BRIDGE

dowel - a length of bar embedded in two parts of a structure to hold the parts in place and to transfer stress

drainage - a system designed to remove water from a structure

drainage area - an area in which surface run-off collects and from which it is carried by a drainage system; also known as catchment area

drain hole - hole in a box shaped member or a wall to provide means for the exit of accumulated water or other liquid; also known as drip hole; see WEEP HOLE

drain pipes - pipes that carry storm water

drawbridge - a general term applied to a bridge over a navigable body of water having a movable superstructure span of any type

drift bolt - a short length of metal bar used to connect and hold in position wooden members placed in contact; similar to a dowel

drift pin - tapered steel rod used by ironworkers to align bolt holes

drip notch - a recess cast on the underside of an overhang that prevents water from following the concrete surface onto the supporting beams

drop inlet - a type of inlet structure that conveys the water from a higher elevation to a lower outlet elevation smoothly without a free fall at the discharge

duct - the hollow space where a prestressing tendon is placed in a post-tensioned prestressed concrete girder

ductile - capable of being molded or shaped without breaking; plastic

ductile fracture - a fracture characterized by plastic deformation

ductility - the ability to withstand nonelastic deformation without rupture

dumbbell pier - a pier consisting of two cylindrical or rectangular shaped piers joined by an integral web

dummy member - truss member that carries no primary loads; may be included for bracing or for appearance

E

E - modulus of elasticity of a material; Young's modulus; the stiffness of a material

efflorescence - a deposit on concrete or brick caused by crystallization of carbonates brought to the surface by moisture in the masonry or concrete

elastic - capable of sustaining deformation without permanent loss of shape

elastic strain - the reversible distortion of a material

elastic deformation - non-permanent deformation; when the stress is removed, the material returns to its original shape

elasticity - the property whereby a material changes its shape under the action of loads but recovers its original shape when the loads are removed

elastomer - a natural or synthetic rubber-like material

elastomeric pad - a synthetic rubber pad used in bearings that compresses under loads and accommodates horizontal movement by deforming

electrolyte - a medium of air, soil, or liquid carrying ionic current between two metal surfaces, the anode and the cathode

electrolytic cell - a device for producing electrolysis consisting of the electrolyte and the electrodes

electrolytic corrosion - corrosion of a metal associated with the flow of electric current in an electrolyte

elevation view - a drawing of the side view of a structure

elliptic arch - an arch in which the intrados surface is a full half of the surface of an elliptical cylinder; this terminology is sometimes incorrectly applied to a multicentered arch

elongation - the elastic or plastic extension of a member

embankment - a mound of earth constructed above the natural ground surface to carry a road or to prevent water from passing beyond desirable limits; also known as bank

end block - in a prestressed concrete I-beam, the widened beam web at the end to provide adequate anchorage bearing for the post tensioning steel and to resist high shear stresses; similarly, the solid end diaphragm of a box beam

end post - the end compression member of a truss, either vertical or inclined in position and extending from top chord to bottom chord

end rotation - Occurs when a structure deflects

end section - a concrete or steel appurtenance attached to the end of a culvert for the purpose of hydraulic efficiency, embankment retention or anchorage

end span - a span adjacent to an abutment

epoxy - a synthetic resin which cures or hardens by chemical reaction between components which are mixed together shortly before use

epoxy coated reinforcement - reinforcement steel coated with epoxy; used to prevent corrosion

equilibrium - in statics, the condition in which the forces acting upon a body are such that no external effect (or movement) is produced

equivalent uniform load - a load having a constant intensity per unit of its length producing an effect equal to that of a live load consisting of vehicle axle or wheel concentrations spaced at varying distances

erosion - wearing away of soil by flowing water not associated with a channel; see SCOUR

expansion - an increase in size or volume

expansion bearing - a bearing designed to permit longitudinal or lateral movements resulting from temperature changes and superimposed loads with minimal transmission of horizontal force to the substructure; see BEARING

expansion dam - the part of an expansion joint serving as an end form for the placing of concrete at a joint; also applied to the expansion joint device itself; see EXPANSION JOINT

expansion joint - a joint designed to permit expansion and contraction movements produced by temperature changes, loadings or other forces

expansion rocker - a bearing device at the expansion end of a beam or truss that allows the longitudinal movements resulting from temperature changes and superimposed loads through a tilting motion

expansion roller - a cylinder so mounted that by revolution it facilitates expansion, contraction or other movements resulting from temperature changes, loadings or other forces

expansion shoe - expansion bearing, generally of all metal construction

exterior girder - an outermost girder supporting the bridge floor

extrados - the curve defining the exterior (upper) surface of an arch; also known as back

eyebar - a member consisting of a rectangular bar with enlarged forged ends having holes for engaging connecting pins

F

failure - a condition at which a structure reaches a limit state such as cracking or deflection where it is no longer able to perform its usual function; collapse; fracture

falsework - a temporary wooden or metal framework built to support the weight of a structure during the period of its construction and until it becomes self-supporting

fascia - an outside, covering member designed on the basis of architectural effect rather than strength and rigidity, although its function may involve both

fascia girder - an exposed outermost girder of a span sometimes treated architecturally or otherwise to provide an attractive appearance

fatigue - the tendency of a member to fail at a stress below the yield point when subjected to repetitive loading

fatigue crack - any crack caused by repeated cyclic loading at a stress below the yield point

fatigue damage - member damage (crack formation) due to cyclic loading

fatigue life - the length of service of a member subject to fatigue, based on the number of cycles it can undergo

fender - a structure that acts as a buffer to protect the portions of a bridge exposed to floating debris and water-borne traffic from collision damage; sometimes called an ice guard in regions with ice floes

fender pier - a pier-like structure which performs the same service as a fender but is generally more substantially built; see GUARD PIER

field coat - a coat of paint applied after the structure is assembled and its joints completely connected; quite commonly a part of the field erection procedure; field painting

fill - material, usually earth, used to change the surface contour of an area, or to construct an embankment

filler - a piece used primarily to fill a space beneath a batten, splice plate, gusset, connection angle, stiffener or other element; also known as filler plate

filler metal - metal prepared in wire, rod, electrode or other form to be fused with the structure metal in the formation of a weld

filler plate - see FILLER

fillet - a curved portion forming a junction of two surfaces that would otherwise intersect at an angle

fillet weld - a weld of triangular or fillet shaped crosssection between two pieces at right angles

filling - see FILL

fine aggregate - sand or grit for concrete or mortar that passes a No. 4 sieve (4.75 mm)

finger dam - expansion joint in which the opening is spanned by meshing steel fingers or teeth

fish belly - a term applied to a girder or a truss having its bottom flange or its bottom chord constructed either haunched or bowshaped with the convexity downward; see LENTICULAR TRUSS

fixed beam - a beam with a fixed end

fixed bearing - a bearing that allows only rotational movement; see BEARING

fixed bridge - a bridge having constant position, i.e., without provision for movement to create increased navigation clearance

fixed end - movement is restrained

fixed-ended arch - see VOUSSOIR ARCH

fixed span - a superstructure span having its position practically immovable, as compared to a movable span

fixed support - a support that will allow rotation only, no longitudinal movement

flange - the (usually) horizontal parts of a rolled I-shaped beam or of a built-up girder extending transversely across the top and bottom of the web

flange angle - an angle used to form a flange element of a built-up girder, column, strut or similar member

floating bridge - see PONTOON BRIDGE

floating foundation - used to describe a soil-supported raft or mat foundation with low bearing pressures; sometimes applied to a "foundation raft" or "foundation grillage"

flood frequency - the average time interval in years in which a flow of a given magnitude will recur

flood plain - area adjacent to a stream or river subject to flooding

floor - see DECK

floorbeam - a primary horizontal member located transversely to the general bridge alignment

floor system - the complete framework of members supporting the bridge deck and the traffic loading

flow capacity - maximum flow rate that a channel, conduit, or culvert structure is hydraulically capable of carrying

flux - a material that protects the weld from oxidation during the fusion process

footbridge - a bridge designed and constructed to provide means of traverse for pedestrian traffic only; also known as pedestrian bridge

footing - the enlarged, lower portion of a substructure, which distributes the structure load either to the earth or to supporting piles; the most common footing is the concrete slab; footer is a colloquial term for footing

footing aprons - protective layers of material surrounding the footing of a substructure unit

foot wall - see TOE WALL

force - an influence that tends to accelerate a body or to change its movement

forms - the molds that hold concrete in place while it is hardening; also known as form work, shuttering; see LAGGING, STAY-IN-PLACE FORMS

form work - see FORMS

foundation - the supporting material upon which the substructure portion of a bridge is placed

foundation excavation - the excavation made to accommodate a footing for a structure; also known as foundation pit

foundation failure - failure of a foundation by differential settlement or by shear failure of the soil

foundation grillage - a construction consisting of steel, timber, or concrete members placed in layers; each layer is perpendicular to those above and below it and the members within a layer are generally parallel, producing a crib or grid-like effect. Grillages are usually placed under very heavy concentrated loads

foundation load - the load resulting from traffic, superstructure, substructure, approach embankment, approach causeway, or other incidental load increment imposed upon a given foundation area

foundation pile - see PILE

foundation pit - see FOUNDATION EXCAVATION

foundation seal - a mass of concrete placed underwater within a cofferdam for the base portion of structure to close or seal the cofferdam against incoming water; see TREMIE

fracture - see BRITTLE FRACTURE

fracture critical member (FCM) - a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse

fracture critical member inspection - a hands-on inspection of a fracture critical member or member components that may include visual and other nondestructive evaluation

frame - a structure which transmits bending moments from the horizontal beam member through rigid joints to vertical or inclined supporting members

framing - the arrangement and connection of the component members of a bridge superstructure

freeboard - the vertical distance between the design flood water surface and the lowest point of the structure to account for waves, surges, drift and other contingencies

free end - movement is not restrained

freeze-thaw - freezing of water within the capillaries and pores of cement paste and aggregate resulting in internal overstressing of the concrete, which leads to deterioration including cracking, scaling, and crumbling.

fretting corrosion - occurs in elements in close contact that are subject to vibrations such as intersecting truss diagonals

friction pile - a pile that provides support through friction resistance between the pile and the surrounding earth along the lateral surface of the pile

friction roller - a roller placed between members intended to facilitate change in their relative positions by reducing the frictional resistance to translation movement

frost heave - the upward movement of, or force exerted by, soil due to freezing of retained moisture

frost line - the depth to which soil may be frozen

functionally obsolete – a bridge that has deck geometry, load carrying capacity, clearance or approach roadway alignment that no longer meets the criteria for the system of which the bridge is a part

G

gabion - rock filled wire baskets used to retain earth and provide erosion control

galvanic action - electrical current between two unlike metals

galvanize - to coat with zinc

gauge - the distance between parallel lines of rails, rivet holes, etc; a measure of thickness of sheet metal or wire; also known as gage

general scour - the lowering of a streambed across the waterway at the bridge, which may or may not be uniform

geometry - shape or form; relationship between lines or points

girder - a horizontal flexural member that is the main or primary support for a structure; any large beam, especially if built up

girder bridge - a bridge whose superstructure consists of two or more girders supporting a separate floor system as differentiated from a multi-beam bridge or a slab bridge

girder span - a span in which the major longitudinal supporting members are girders

glue laminated - a member created by gluing together two or more pieces of lumber

grade - the fall or rise per unit horizontal length; see GRADIENT

grade crossing - a term applicable to an intersection of two highways, two railroads or a railroad and a highway at a common grade or elevation; now commonly accepted as meaning the last of these combinations

grade intersection - the location where two roadway slopes meet in profile; to provide a smooth transition from one to the other they are connected by a vertical curve and the resulting profile is a sag or a crest

grade separation - roadways crossing each other at different elevations; see OVERPASS, UNDERPASS

gradient - the rate of inclination of the roadway and˜ or sidewalk surface(s) from the horizontal, applying to a bridge and its approaches; it is commonly expressed as a percentage relation (ratio) of horizontal to vertical dimensions

gravity abutment - a thick abutment that resists horizontal earth pressure through its own dead weight

gravity wall - a retaining wall that is prevented from overturning or sliding by its own dead weight

grid flooring - a steel floor system comprising a lattice pattern that may or may not be filled with concrete

grillage - assembly of parallel beams, usually steel or concrete, placed side by side, often in layers with alternating directions; see FOUNDATION GRILLAGE

groin - a wall built out from a river bank to check scour

grout - mortar having a sufficient water content to render it free-flowing, used for filling (grouting) the joints in masonry, for fixing anchor bolts and for filling cored spaces; usually a thin mix of cement, water and sometimes sand or admixtures

grouting - the process of filling in voids with grout

guard pier - a pier-like structure built to protect a swing span in its open position from collision with passing vessels or water-borne debris; may be equipped with a rest pier upon which the swing span in its open position may be latched; see FENDER PIER

guardrail - a safety feature element intended to redirect an errant vehicle

guide banks - dikes that extend upstream from the approach embankment at either or both sides of the bridge opening to direct flow through the opening

guide rail - see GUARDRAIL

gunite - the process of blowing Portland cement mortar or concrete onto a surface using compressed air

gusset plate - a plate that connects the members of a structure and holds them in correct position at a joint

gutter - a paved ditch; area adjacent to a roadway curb used for drainage

guy - a cable member used to anchor a structure in a desired position

H

H Loading - a combination of loads used to represent a two-axle truck developed by AASHTO

hairline cracks - very narrow cracks that form in the surface of concrete due to tension caused by loading

hammer - hand tool used for sounding and surface inspection

hammerhead pier - a pier with a single cylindrical or rectangular shaft and a relatively long, transverse cap; also known as a tee pier or cantilever pier

hand hole - hole provided in component plate of built-up box section to permit access to the interior for construction and maintenance purposes

hand rail - commonly applies only to sidewalk railing presenting a latticed, barred, balustered or other open web construction

hands-on - inspection within arms length of the component. Inspection uses visual techniques that may be supplemented by nondestructive testing

hands-on access - close enough to the member or component so that it can be touched with the hands and inspected visually

hanger - a tension member serving to suspend an attached member; allows for expansion between a cantilevered and suspended span

haunch - an increase in the depth of a member usually at points of support; the outside areas of a pipe between the spring line and the bottom of the pipe

haunched girder - a horizontal beam whose cross sectional depth varies along its length

H-beam - a rolled steel member having an H-shaped cross-section (flange width equals beam depth) commonly used for piling; also H-pile

head - a measure of water pressure expressed in terms of an equivalent weight or pressure exerted by a column of water; the height of the equivalent column of water is the head

head loss - the loss of energy between two points along the path of a flowing fluid due to fluid friction; reported in feet of head

headwall **-** a concrete structure at the ends of a culvert to retain the embankment slopes, anchor the culvert, and prevent undercutting

headwater - the source or the upstream waters of a stream

heat treatment - any of a number of various operations involving controlled heating and cooling that are used to impart specific properties to metals; examples are tempering, quenching, and annealing

heave - the upward motion of soil caused by outside forces such as excavation, pile driving, moisture or soil expansion; see FROST HEAVE

heel - the portion of a footing behind the stem

helical - having the form of a spiral

high carbon steel - carbon steel containing 0.5 to 1.5% dissolved carbon

high strength bolt - bolt and nut made of high strength steel, usually A325 or A490

highway - the term 'highway' includes:

A) a road, street, and parkway;
B) a right-of-way, bridge, railroad-highway crossing, tunnel, drainage structure, sign, guardrail, and protective structure, in connection with a highway; and
C) a portion of any interstate or international bridge or tunnel and the approaches thereto, the cost of which is assumed by a State transportation department, including such facilities as may be required by the United States Customs and Immigration Services in connection with the operation of an international bridge or tunnel

hinge - a point in a structure at which a member is free to rotate

hinged joint - a joint constructed with a pin, cylinder segment, spherical segment or other device permitting rotational movement

honeycomb - an area in concrete where mortar has separated and left spaces between the coarse aggregate, usually caused by improper vibration during concrete construction

horizontal alignment - a roadway's centerline or baseline alignment in the horizontal plane

horizontal curve - a roadway baseline or centerline alignment defined by a radius in the horizontal plane

horizontal shear splits - separations of the wood fibers parallel to the grain due to excessive loading

Howe truss - a truss of the parallel chord type with a web system composed of vertical (tension) rods at the panel points with an X pattern of diagonals

HS Loading - a combination of loads developed by AASHTO used to represent a truck and trailer

hybrid girder - a girder whose flanges and web are made from steel of different grades

hydraulic countermeasures - man-made or man-placed devices designed to direct streamflow and to protect against lateral migration and scour

hydraulics - the mechanics of fluids

hydrology - study of the accumulation and flow of water from watershed areas

hydroplaning - loss of contact between a tire and the roadway surface when the tire planes or glides on a film of water

I

I-beam - a structural member with a crosssectional shape similar to the capital letter "I"

ice guard - see FENDER

impact - A factor that describes the effect on live load due to dynamic and vibratory effects of a moving load; in bridge design, a load based on a percentage of live load to include dynamic and vibratory effects; in fracture mechanics, a rapidly applied load, such as a collision or explosion

incomplete fusion - a weld flaw where the weld metal has not combined metallurgically with the base metal

in-depth inspection - a close-up, inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures; hands-on inspection may be necessary at some locations

indeterminate stress - stress in a structural member which cannot be calculated directly; it is computed by the iterative application of mathematical equations, usually with an electronic computer; indeterminate stresses arise in continuous span and frame type structures

individual column footing - footing supporting one column

inelastic compression - compression beyond the yield point

initial inspection - the first inspection of a bridge as it becomes a part of the bridge file to provide all Structure Inventory and Appraisal (SI&A) data and other relevant data and to determine baseline structural conditions.

inlet - an opening in the floor of a bridge leading to a drain; roadway drainage structure which collects surface water and transfers it to pipes

inspection frequency - the frequency with which the bridge is inspected -- normally every two years

integral abutment - an abutment cast monolithically with the end diaphragm of the deck; such abutments usually encase the ends of the deck beams and are pile supported

integral deck - a deck which is monolithic with the superstructure; concrete tee beam bridges have integral decks

intercepting ditch - a ditch constructed to prevent surface water from flowing in contact with the toe of an embankment or causeway or down the slope of a cut

interior girder - any girder between exterior or fascia girders

interior span - a span of which both supports are intermediate substructure units

intermittent weld - a noncontinuous weld commonly composed of a series of short welds separated by spaces of equal length

internal redundancy - a bridge member having several elements that are mechanically fastened together

internal steel corrosion - occurs due to the elimination of the protection of steel caused by chlorides

intrados - the curve defining the interior (lower) surface of the arch; also known as soffit

inventory item - data contained in the structure file pertaining to bridge identification, structure type and material, age and service, geometric data, navigational data, classification, load rating and posting, proposed improvements, and inspections

inventory rating - the capacity of a bridge to withstand loads under normal service conditions based on 55% of yield strength

invert elevation - the bottom or lowest point of the internal surface of the transverse cross section of a pipe or culvert

iron - a metallic element used in cast iron, wrought iron and steel

isotropic - having the same material properties in all directions, e.g., steel

J

jack arch - a deck support system comprised of a brick or concrete arch springing from the bottom flanges of adjacent rolled steel beams

jacking - the lifting of elements using a type of jack (e.g., hydraulic), sometimes acts as a temporary support system

jack stringer - the outermost stringer supporting the bridge floor in a panel or bay

jacket - a protective shell surrounding a pile made of fabric, concrete or other material

jersey barrier - a concrete barrier with sloping front face that was developed by the New Jersey Department of Transportation

joint - in masonry, the space between individual stones or bricks; in concrete, a division in continuity of the concrete; in a truss, point at which members of a truss are joined

K

keeper plate - a plate, which is connected to a sole plate, designed to prohibit a beam from becoming dislodged from the bearing

key - a raised portion of concrete on one face of a joint that fits into a depression on the adjacent face

keystone - the symmetrically shaped, wedgẽ like stone located in a head ring course at the crown of an arch; the final stone placed, thereby closing the arch

king-post - the vertical member in a "king-post" type truss; also known as king rod

king-post truss - two triangular panels with a common center vertical; the simplest of triangular system trusses

kip - a kilo pound (1000 lb.); convenient unit for structural calculations

knee brace - a short member engaging at its ends two other members that are joined to form a right angle or a near-right angle to strengthen and stiffen the connecting joint

knee wall - a return of the abutment backwall at its ends to enclose the bridge seat on three of its sides; also called cheek wall

knife edge - a condition in which corrosion of a steel member has caused a sharp edge

knuckle - an appliance forming a part of the anchorage of a suspension bridge main suspension member permitting movement of the anchorage chain

knots - separations of the wood fibers due to the trunk growing around an embedded limb

K-truss - a truss having a web system wherein the diagonal members intersect the vertical members at or near the mid-height; the assembly in each panel forms a letter "K"

L

L-abutment - a cantilever abutment with the stem flush with the toe of the footing, forming an "L" in cross section

laced column - a riveted, steel built-up column of usually four angles or two channels tied together laterally with lacing

lacing - small flat plates, usually with one rivet at each end, used to tie individual sections of built up members; see LATTICE

lagging - horizontal members spanning between piles to form a wall; forms used to produce curved surfaces; see FORMS

lamellar tear - incipient cracking parallel to the face of a steel member

laminated timber - timber planks glued together face to face to form a larger member; see GLUE LAMINATED

lane loading - a design loading which represents a line of trucks crossing over a bridge

lap joint - a joint between two members in which the end of one member overlaps the end of the other

lateral - a member placed approximately perpendicular to a primary member

lateral bracing - the bracing assemblage engaging a member perpendicular to the plane of the member; intended to resist transverse movement and deformation; also keeps primary parallel elements in truss bridges and girder bridges aligned; see BRACING

lateral stream migration - the relocation of the channel due to lateral streambank erosion

lattice - a crisscross assemblage of diagonal bars, channels, or angles on a truss; also known as latticing, lacing

lattice truss - in general, a truss having its web members inclined but more commonly the term is applied to a truss having two or more web systems composed entirely of diagonal members at any interval and crossing each other without reference to vertical members

leaching - the action of removing substances from a material by passing water through it

lead line - a weighted cord incrementally marked, used to determine the depth of a body of water; also known as sounding line

leaf - the movable portion of a bascule bridge that forms the span of the structure

legal load - the maximum legal load for each vehicle configuration permitted by law for the State in which the bridge is located

lenticular truss - a truss having parabolic top and bottom chords curved in opposite directions with their ends meeting at a common joint; also known as a fish belly truss

levee - an embankment built to prevent flooding of low-lying land

leveling course - a layer of bituminous concrete placed to smooth an irregular surface

light-weight concrete - concrete of less than standard unit weight; may be no-fines concrete, aerated concrete, or concrete made with lightweight aggregate

link - a hanger plate in a pin and hanger assembly whose shape is similar to an eyebar, e.g., the head (at the pinhole) is wider than the shank

link and roller - a movable bridge element consisting of a hinged strutlike link fitted with a roller at its bottom end, supported upon a shoe plate or pedestal and operated by a thrust strut serving to force it into a vertical position and to withdraw it therefrom; when installed at each outermost end of the girders or the trusses of a swing span their major function is to lift them to an extent that their camber or droop will be removed and the arms rendered free to act as simple spans; when the links are withdrawn to an inclined position fixed by the operating mechanism the span is free to be moved to an open position

live load - a temporary dynamic load such as vehicular traffic that is applied to a structure; also accompanied by vibration or movement affecting its intensity

load - a force carried by a structure component

load factor design - a design method used by AASHTO, based on limit states of material and arbitrarily increased loads

load indicating washer - a washer with small projections on one side, which compress as the bolt is tightened; gives a direct indication of the bolt tension that has been achieved

load path redundancy - a bridge having three or more main load-carrying members

load rating - the determination of the live load carrying capacity of a bridge using bridge plans and supplemented by information gathered from a field inspection

load and resistance factor design (LRFD) - design method used by AASHTO, based on limit states of material with increased loads and reduced member capacity based on statistical probabilities

local buckling - localized buckling of a beam's plate element, can lead to failure of member

local scour - the removal of streambed material adjacent to an obstruction in a waterway, that has been placed within the stream (such as a pier or abutment), and causes the acceleration of the flow induced by the obstruction

longitudinal bracing - bracing that runs lengthwise with a bridge and provides resistance against longitudinal movement and deformation of transverse members

loss of prestress - loss of prestressing force due to a variety of factors, including shrinkage and creep of the concrete, creep of the prestressing tendons, and loss of bond

low-carbon steel - steel with 0.04 to 0.25% dissolved carbon; also called mild steel

lower chord - the bottom horizontal member of a truss

luminaire - a lighting fixture

M

macadam - roadway pavement made with crushed stone aggregate, of coarse open gradation, compacted in place; asphaltic macadam included asphalt as a binder

main beam - a horizontal structural member which supports the span and bears directly on a column or wall

maintenance - basic repairs performed on a facility to keep it at an adequate level of service

maintenance and protection of traffic - the management of vehicular and pedestrian traffic through a construction zone to ensure the safety of the public and the construction workforce; MPT; TRAFFIC PROTECTION

marine borers - mollusks and crustaceans that live in water and destroy wood by digesting it

masonry - that portion of a structure composed of stone, brick or concrete block placed in courses and usually cemented with mortar

masonry cement - Portland cement and lime used to make mortar for masonry construction

masonry plate - a steel plate placed on the substructure to support a superstructure bearing and to distribute the load to the masonry beneath

mattress - a flexible scour protection blanket composed of interconnected timber, gabions, or concrete units.

meander - a twisting, winding action from side to side; characterizes the serpentine curvature of a narrow, slow flowing stream in a wide flood plain

median - separation between opposing lanes of highway traffic; also known as median strip

member - an individual angle, beam, plate, or built component piece intended ultimately to become an integral part of an assembled frame or structure

metal corrosion - oxidation of metal by electro-galvanic action involving an electrolyte (moisture), an anode (the metallic surface where oxidation occurs), a cathode (the metallic surface that accepts electrons and does not corrode), and a conductor (the metal piece itself)

midspan - a reference point halfway between the supports of a beam or span

mild steel - steel containing from 0.04 to 0.25% dissolved carbon; see LOW CARBON STEEL

military loading - a loading pattern used to simulate heavy military vehicles passing over a bridge

mill scale - dense iron oxide on iron or steel that forms on the surface of metal that has been forged or hot worked

modular joint - a bridge joint designed to handle large movements consisting of an assembly of several strip or compression seals

modulus of elasticity - the ratio between the stress applied and the resulting elastic strain

moisture content - the amount of water in a material expressed as a percent by weight

moment - the couple effect of forces about a given point; see BENDING MOMENT

monolithic - forming a single mass without joints

mortar - a paste of portland cement, sand, and water laid between bricks, stones or blocks

movable bridge - a bridge having one or more spans capable of being raised, turned, lifted, or slid from its normal service location to provide a clear navigation passage; see BASCULE BRIDGE, VERTICAL LIFT BRIDGE, PONTOON BRIDGE, RETRACTILE DRAW BRIDGE, ROLLING LIFT BRIDGE, and SWING BRIDGE

movable span - a general term applied to a superstructure span designed to be swung, lifted or otherwise moved longitudinally, horizontally or vertically, usually to provide increased navigational clearance

moving load - a live load which is moving, for example, vehicular traffic

MPT - see MAINTENANCE AND PROTECTION OF TRAFFIC

MSE - mechanically stabilized earth; see REINFORCED EARTH

multi-centered arch - an arch in which the intrados surface is outlined by two or more arcs symmetrically arranged and having different radii that intersect tangentially

N

nail laminated - a laminated member produced by nailing two or more pieces of timber together face to face

National Bridge Inspection Standards NBIS - National Bridge Inspection Standards, first established in 1971 to set national policy regarding bridge inspection frequency, inspector qualifications, report formats, and inspection and rating procedures

National Bridge Inventory (NBI) - A database of Structure Inventory and Appraisal data collected by each state or Federal bridge-owning agency to fulfill the requirements of the NBIS

NCHRP - National Cooperative Highway Research Program

NICET - National Institute for Certification in Engineering Technologies, the NICET provides nationally applicable voluntary certification programs covering several broad engineering technology fields and a number of specialized subfields. For information on the NICET program certification contact: National Institute for Certification in Engineering Technologies, 1420 King Street, Alexandria, VA 22314–2794.

NDE - nondestructive evaluation

NDT - nondestructive testing; any testing method of checking structural quality of materials that does not damage them

necking - the elongation and contraction in area that occurs when a ductile material is stressed

negative bending - bending of a member that causes tension in the surface adjacent to the load, e.g., moment at interior supports of a span or at the joints of a frame

negative moment - bending moment in a member such that tension stresses are produced in the top portions of the member; typically occurs in continuous beams and spans over the intermediate supports

neoprene - a synthetic rubber-like material used in expansion joints and elastomeric bearings

neutral axis - the internal axis of a member in bending along which the strain is zero; on one side of the neutral axis the fibers are in tension, on the other side the fibers are in compression

Non-homogeneous -

nose - a projection acting as a cut water on the upstream end of a pier; see STARLING

notch effect - stress concentration caused by an abrupt discontinuity or change in section

O

offset - a horizontal distance measured at right angles to a survey line to locate a point off the line

on center - a description of a typical dimension between the centers of the objects being measured

open spandrel arch - a bridge that has open spaces between the deck and the arch members allowing "open" visibility through the bridge

open spandrel ribbed arch - a structure in which two or more comparatively narrow arch rings, called ribs, function in the place of an arch barrel; the ribs are rigidly secured in position by arch rib struts located at intervals along the length of the arch; the arch ribs carry a column type open spandrel construction which supports the floor system and its loads

operating rating - the capacity of a bridge to withstand loads based on 75% of yield strength; the maximum permissible live load to which the structure may be subjected for the load configuration used in the rating

operator's house - the building containing control devices required for opening and closing a movable bridge span

orthotropic - having different properties in two or more directions at right angles to each other (e.g., wood); see ANISOTROPY

outlet - in hydraulics, the discharge end of drains, sewers, or culverts

out-of-plane distortion - distortion of a member in a plane other than that which the member was designed to resist

overlay - see WEARING SURFACE

overload - a weight greater than the structure is designed to carry

overload damage - occurs when concrete members are sufficiently overstressed

overpass - bridge over a roadway or railroad

overturning - tipping over; rotational movement

oxidation - the chemical breakdown of a substance due to its reaction with oxygen from the air

oxidized steel - rust

P

pack - a steel plate inserted between two others to fill a gap and fit them tightly together; also known as packing; fill; filler plate

pack rust - rust forming between adjacent steel surfaces in contact which tends to force the surfaces apart due to the increase in material volume

paddleboard - striped, paddle-shaped signs or boards placed on the roadside in front of a narrow bridge as a warning of reduced roadway width

panel -the portion of a truss span between adjacent points of intersection of web and chord members

panel point - the point of intersection of primary web and chord members of a truss

parabolic arch - an arch in which the intrados surface is a segment of a symmetrical parabolic surface (suited to concrete arches)

parabolic truss - a polygonal truss having its top chord and end post vertices coincident with the arc of a parabola, its bottom chord straight and its web system either triangular or quadrangular; also known as a parabolic arched truss

parapet - a low wall along the outmost edge of the roadway of a bridge to protect vehicles and pedestrians

pedestal - concrete or built-up metal member constructed on top of a bridge seat for the purpose of providing a specific bearing seat elevation

pedestal pier - one or more piers built in block-like form that may be connected by an integrally built web between them; when composed of a single, wide blocklike form, it is called a wall or solid pier

pedestrian bridge - see FOOT BRIDGE

penetration - when applied to creosoted lumber, the depth to which the surface wood is permeated by the creosote oil; when applied to pile driving; the depth a pile tip is driven into the ground

permanent loads - loads that are constant for the life of the structure

physical testing - the testing of bridge members in the field or laboratory

pier - a substructure unit that supports the spans of a multi-span superstructure at an intermediate location between its abutments

pier cap - the topmost horizontal portion of a pier that distributes loads from the superstructure to the vertical pier elements

pile - a shaft-like linear member which carries loads to underlying rock or soil strata

pile bent - a row of driven or placed piles extending above the ground surface supporting a pile cap; see BENT

pile bridge - a bridge carried on piles or pile bents

pile cap - a slab or beam which acts to secure the piles in position laterally and provides a bridge seat to receive and distribute superstructure loads

pile foundation - a foundation supported by piles in sufficient number and to a depth adequate to develop the bearing resistance required to support the substructure load

pile pier - see PILE BENT

piling - collective term applied to group of piles in a construction; see PILE, SHEET PILES

pin - a cylindrical bar used to connect elements of a structure

pin-connected truss - a general term applied to a truss of any type having its chord and web members connected at each panel point by a single pin

pin and hanger - a hinged connection detail designed to allow for expansion and rotation between a cantilevered and suspended span at a point between supports.

pin joint - a joint in a truss or other frame in which the members are assembled upon a single cylindrical pin

pin packing - arrangement of truss members on a pin at a pinned joint

pin plate - a plate rigidly attached upon the end of a member to develop the desired bearing upon a pin or pin-like bearing, and secure additional strength and rigidity in the member; doubler plate

pintle - a relatively small steel pin engaging the rocker of an expansion bearing, in a sole plate or masonry plate, thereby preventing sliding of the rocker

pipe - a hollow cylinder used for the conveyance of water, gas, steam etc.

piping - removal of fine particles from within a soil mass by flowing water

plain concrete - concrete with no structural reinforcement except, possibly, light steel to reduce shrinkage and temperature cracking

plan and profile - a drawing that shows both the roadway plan view and profile view in the same scale; see PLAN VIEW, PROFILE

plan view - drawing that represents the top view of the road or a structure

plastic deformation - permanent deformation of material beyond the elastic range

plastic strain - the irreversible or permanent distortion of a material

plate - a flat sheet of metal which is relatively thick; see SHEET STEEL

plate girder - a large I-shaped beam composed of a solid web plate with flange plates attached to the web plate by flange angles or fillet welds

plug weld - a weld joining two members produced by depositing weld metal within holes cut through one or more of the members; also known as slot weld

plumb bob - a weight hanging on a cord used to provide a true vertical reference

plumb line - a true vertical reference line established using a plumb bob

pneumatic caisson - an underwater caisson in which the working chamber is kept free of water by compressed air at a pressure nearly equal to the water pressure outside it

point loads - loads that are applied to a localized area

pointing - the compacting of the mortar into the outermost portion of a joint and the troweling of its exposed surface to secure water tightness or desired architectural effect; replacing deteriorated mortar

ponding - accumulation of water

pontoon bridge - a bridge supported by floating on pontoons moored to the riverbed; a portion may be removable to facilitate navigation

pony truss - a through truss without top chord lateral bracing

pop-out - conical fragment broken out of a concrete surface by pressure from reactive aggregate particles

portable bridge - a bridge that may be readily erected for a temporary communication-transport service and disassembled and reassembled at another location

portal - the clear unobstructed space of a through truss bridge forming the entrance to the structure

portal bracing - a system of sway bracing placed in the plane of the end posts of the trusses

portland cement - a fine dry powder made by grinding limestone clinker made by heating limestone in a kiln; this material reacts chemically with water to produce a solid mass

portland cement concrete - a mixture of aggregate, portland cement, water, and usually chemical admixtures

positive moment - a force applied over a distance that causes compression in the top fiber of a beam and tension in the bottom fiber

post - a member resisting compressive stresses, located vertical to the bottom chord of a truss and common to two truss panels; sometimes used synonymously for vertical; see COLUMN

posting - a limiting dimension, speed, or loading indicating larger dimensions, higher speeds, or greater loads cannot be safely taken by the bridge

post-stressing - see POSTTENSIONING

posttensioning - a method of prestressing concrete in which the tendons are stressed after the concrete has been cast and hardens

pot bearing - a bearing type that allows for multi-dimensional rotation by using a piston supported on an elastomer contained on a cylinder ("pot"), or spherical bearing element

pot holes - irregular shaped, disintegrated areas of bridge deck or roadway pavement caused by the failure of the surface material

Pratt truss - a truss with parallel chords and a web system composed of vertical posts with diagonal ties inclined outward and upward from the bottom chord panel points toward the ends of the truss; also known as N-truss

precast concrete - concrete members that are cast and cured before being placed into their final positions on a construction site

prestressed concrete - concrete with strands, tendons, or bars that are stressed before the live load is applied

prestressing - applying forces to a structure to deform it in such a way that it will withstand its working loads more effectively; see POSTTENSIONING, PRETENSIONING

pretensioning - a method of prestressing concrete in which the strands are stressed before the concrete is placed; strands are released after the concrete has hardened, inducing internal compression into the concrete

primary member - a member designed to resist flexure and distribute primary live loads and dead loads

priming coat - the first coat of paint applied to the metal or other material of a bridge; also known as base coat, or primer

probing - investigating the location and condition of submerged foundation material using a rod or shaft of appropriate length; checking the surface condition of a timber member for decay using a pointed tool, e.g., an ice pick

Professional engineer (PE) - an individual, who has fulfilled education and experience requirements and passed rigorous exams that, under State licensure laws, permits them to offer engineering services directly to the public. Engineering licensure laws vary from State to State, but, in general, to become a PE an individual must be a graduate of an engineering program accredited by the Accreditation Board for Engineering and Technology, pass the Fundamentals of Engineering exam, gain four years of experience working under a PE, and pass the Principles of Practice of Engineering exam

profile - a section cut vertically along the center line of a roadway or waterway to show the original and final ground levels

program manager - the individual in charge of the program, that has been assigned or delegated the duties and responsibilities for bridge inspection, reporting, and inventory. The program manager provides overall leadership and is available to inspection team leaders to provide guidance

programmed repair - those repairs that may be performed in a scheduled program

protective system - a system used to protect bridges from environmental forces that cause steel and concrete to deteriorate and timber to decay, typically a coating system

PS&E - Plans, Specifications, and Estimate; the final submission of the designers to the owner

public road. **-** the term ``public road'' means any road or street under the jurisdiction of and maintained by a public authority and open to public travel

punching shear - shear stress in a slab due to the application of a concentrated load

Q

quality assurance (QA) - the use of sampling and other measures to assure the adequacy of quality control procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program

quality control (QC) - procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level

queen-post truss - a parallel chord type of truss having three panels with the top chord occupying only the length of the center panel

R

railing - a fence-like construction built at the outermost edge of the roadway or the sidewalk portion of a bridge to protect pedestrians and vehicles; see HANDRAIL

rake - an angle of inclination of a surface in relation to a vertical plane; also known as batter

ramp - an inclined traffic-way leading from one elevation to another

range of stress - the algebraic difference between the minimum and maximum stresses in a member

raveling - the consistent loss of aggregate from a pavement resulting in a poor riding surface

reaction - the resistance of a support to a load

rebar - see REINFORCING BAR

redundancy - the structural condition where there are more elements of support than are necessary for stability.

redundant member - a member in a bridge which renders it a statically indeterminate structure; the structure would be stable without the redundant member whose primary purpose is to reduce the stresses carried by the determinate structure

rehabilitation - significant repair work to a structure

reinforced concrete - concrete with steel reinforcing bars embedded in it to supply increased tensile strength and durability

reinforced concrete pipe - pipe manufactured of concrete reinforced with steel bars or welded wire fabric

Reinforced Earth - proprietary retaining structure made of earth and steel strips connected to concrete facing; the steel strips are embedded in backfill and interlock with the facing; see MSE

reinforcement - rods or mesh embedded in concrete to strengthen it

reinforcing bar - a steel bar, plain or with a deformed surface, which bonds to the concrete and supplies tensile strength to the concrete

relaxation - a decrease in stress caused by creep

residual stress - a stress that is trapped in a member after it is formed into its final shape

resistivity of soil - an electrical measurement in ohm-cm that estimates the corrosion activity potential of a given soil

resurfacing - a layer of wearing surface material that is put over the approach or deck surface in order to create a more uniform riding surface

Retained Earth - proprietary retaining structure made of weld wire fabric strips connected to concrete facing; see MSE

retaining wall - a structure designed to restrain and hold back a mass of earth

retractile draw bridge - a bridge with a superstructure designed to move horizontally, either longitudinally or diagonally, from "closed" to "open" position, the portion acting in cantilever being counterweighted by that supported on rollers; also known as traverse draw bridge

rib - curved structural member supporting a curved shape or panel

rigger - an individual who erects and maintains scaffolding or other access equipment such as that used for bridge inspection

rigid frame - a structural frame in which bending moment is transferred between horizontal and vertical or inclined members by joints

rigid frame bridge - a bridge with moment resisting joints between the horizontal portion of the superstructure and vertical or inclined legs

rigid frame pier - a pier with two or more columns and a horizontal beam on top constructed monolithically to act like a frame

rip-rap - stones, blocks of concrete or other objects placed upon river and stream beds and banks, lake, tidal or other shores to prevent scour by water flow or wave action

river training structures - devices that alter the flow of the river

rivet - a one-piece metal fastener held in place by forged heads at each end

riveted joint - a joint in which the assembled members are fastened by rivets

roadway - the portion of the road intended for the use of vehicular traffic

roadway shoulder - drivable area immediately adjoining the traveled roadway

rocker bearing - a bridge support that accommodates expansion and contraction of the superstructure through a tilting action

rocker bent - a bent hinged or otherwise articulated at one or both ends to provide the longitudinal movements resulting from temperature changes and superimposed loads

rolled shape - forms of rolled steel having "I", "H", "C", "Z" or other cross sectional shapes

rolled-steel section - any hot-rolled steel section including wide flange shapes, channels, angles, etc.

roller - a steel cylinder intended to provide longitudinal movements by rolling contact

roller bearing - a single roller or a group of rollers so installed as to permit longitudinal movement of a structure

roller nest - a group of steel cylinders used to facilitate the longitudinal movements resulting from temperature changes and superimposed loads

rolling lift bridge - a bridge of bascule type devised to roll backward and forward upon supporting girders when operated through an "open and closed" cycle

routine inspection - regularly scheduled inspection 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 requirements.

routine permit load - a live load, which has a gross weight, axle weight or distance between axles not conforming with State statutes for legally configured vehicles, authorized for unlimited trips over an extended period of time to move alongside other heavy vehicles on a regular basis.

rubble - irregularly shaped pieces of stone in the undressed condition obtained from a quarry and varying in size

runoff - the quantity of precipitation that flows from a catchment area past a given point over a certain period

S

sacrificial anode - the anode in a cathodic protection system

sacrificial coating - a coating over the base material to provide protection to the base material; examples include galvanizing on steel and aluclading on aluminum

sacrificial protection - see CATHODIC PROTECTION

sacrificial thickness - additional material thickness provided for extra service life of a member in an aggressive environment

saddle - a member located upon the topmost portion of the tower of a suspension bridge which acts as a bearing surface for the catenary cable passing over it

safe load - the maximum load that a structure can support with an appropriate factor of safety

safety belt - a belt worn in conjunction with a safety line to prevent falling a long distance when working at heights; no longer acceptable as fall protection under OSHA rules

safety curb - a curb between 9 inches and 24 inches wide serving as a limited use refuge or walkway for pedestrians crossing a bridge

safety factor - the difference between the ultimate strength of a member and the maximum load it is expected to carry

safety harness - harness with shoulder, leg, and waist straps of approved OSHA design used as personal fall protection in conjunction with appropriate lanyards and tie off devices

sag - to sink or bend downward due to weight or pressure

scab - a plank bolted over the joint between two timber members to hold them in correct alignment and strengthen the joint; a short piece of I-beam or other structural shape attached to the flange or web of a metal pile to increase its resistance to penetration; also known as scab piece

scaling - the gradual disintegration of a concrete surface due to the failure of the cement paste caused by chemical attack or freeze-thaw cycles

scour - removal of a streambed or bank area by stream flow; erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges

scour critical bridge - a bridge with a foundation element that has been determined to be unstable for the observed or evaluated scour condition.

scour protection - protection of submerged material by steel sheet piling, rip rap, concrete lining, or combination thereof

scuba - self-contained underwater breathing apparatus; a portable breathing device for free swimming divers

scupper - an opening in the deck of a bridge to provide means for water accumulated upon the roadway surface to drain

seam weld - a weld joining the edges of two members placed in contact; in general, it is not a stress-carrying weld

seat - a base on which an object or member is placed

seat angle - a piece of angle attached to the side of a member to provide support for a connecting member either temporarily during its erection or permanently; also known as a shelf angle

secondary member - a member that does not carry calculated live loads; bracing members

section loss - loss of a member's cross sectional area usually by corrosion or decay

section view - an internal representation of a structure element as if a slice was made through the element

seepage - the slow movement of water through a material

segmental - constructed of individual pieces or segments which are collectively joined to form the whole

segmental arch - a circular arch in which the intrados is less than a semi-circle

segregation - in concrete construction, the separation of large aggregate from the paste during placement

seismic - a term referring to earthquakes (e.g., seismic forces)

semi-stub abutment - cantilever abutment founded part way up the slope, intermediate in size between a full height abutment and a stub abutment

service load design - AASHTO's description for Working Stress Design

settlement - the movement of substructure elements due to changes in the soil properties

shadow vehicle - vehicle used to prevent vehicles from entering the work zone if the motorist drifts into the lane closure

shakes -separations of the wood fibers parallel to the grain between the annual growth rings

shear - the load acting across a beam near its support

shear connectors - devices that extend from the top flange of a beam and are embedded in the above concrete slab, forcing the beam and the concrete to act as a single unit

shear force - equal but opposite forces that tend to slide one section of a member past the adjacent section

shear spiral - a coil-shaped component welded to the top flange of a beam, as a shear connector

shear stress - the shear force per unit of crosssectional area; also referred to as diagonal tensile stress

shear stud - a type of shear connector in the form of a road with a head that is attached to a beam with an automatic stud-welding gun

sheet pile cofferdam - a wall-like barrier composed of driven piling constructed to surround the area to be occupied by a structure and permit dewatering of the enclosure so that the excavation may be performed in the open air

sheet piles - flattened Z-shaped interlocking piles driven into the ground to keep earth or water out of an excavation or to protect an embankment

sheet piling - a general or collective term used to describe a number of sheet piles installed to form a crib, cofferdam, bulkhead, etc.; also known as sheeting

sheet steel - steel in the form of a relatively thin sheet or plate; for flat rolled steel, specific thicknesses vs. widths are classified by AISI as bar, strip, sheet or plate

shelf angle - see SEAT ANGLE

shim - a thin plate inserted between two elements to fix their relative position and to transmit bearing stress

shoe - a steel or iron member, usually a casting or weldment, beneath the superstructure bearing that transmits and distributes loads to the substructure bearing area

shop - a factory or workshop

shop drawings - detailed drawings developed from the more general design drawings used in the manufacture or fabrication of bridge components

shoring - a strut or prop placed against or beneath a structure to restrain movement; temporary soil retaining structure

shoulder abutment - a cantilever abutment extending from the grade line of the road below to that of the road overhead, usually set just off the shoulder; see FULL HEIGHT ABUTMENT

shoulder area - see ROADWAY SHOULDER

shrinkage – a reduction in volume caused by moisture loss in concrete or timber while drying

sidewalk - the portion of the bridge floor area serving pedestrian traffic only

sidewalk bracket - frame attached to and projecting from the outside of a girder to serve as a support for the sidewalk stringers, floor and railing or parapet

sight distance - the length of roadway ahead that is easily visible to the driver; required sight distances are defined by AASHTO's "A Policy on Geometric Design of Highways and Streets"

silt - very finely divided siliceous or other hard rock material removed from its mother rock through erosive action rather than chemical decomposition

simple span - beam or truss with two unrestraining supports near its ends

S-I-P forms - see STAY-IN-PLACE FORMS, FORMS

skew angle - the angle produced when the longitudinal members of a bridge are not perpendicular to the substructure; the skew angle is the acute angle between the alignment of the bridge and a line perpendicular to the centerline of the substructure units

skewback - the inclined support at each end of an arch

skewback shoe - the member transmitting the thrust of an arch to the skewback course or cushion course of an abutment or piers; also known as skewback pedestal

slab - a wide beam, usually of reinforced concrete, which supports load by flexure

slab bridge - a bridge having a superstructure composed of a reinforced concrete slab constructed either as a single unit or as a series of narrow slabs placed parallel with the roadway alignment and spanning the space between the supporting substructure units

slide - movement on a slope because of an increase in load or a removal of support at the toe; also known as landslide

slip form - to form concrete by advancing a mold

slope - the inclination of a surface expressed as a ratio of one unit of rise or fall for so many horizontal units

slope protection - a thin surfacing of stone, concrete or other material deposited upon a sloped surface to prevent its disintegration by rain, wind or other erosive action; also known as slope pavement

slot weld - see PLUG WELD

slump - a measurement taken to determine the stiffness of concrete; the measurement is the loss in height after a cone-shaped mold is lifted

soffit - underside of a bridge deck; also see INTRADOS

soldier beam - a steel pile driven into the earth with its projecting butt end used as a cantilever beam

soldier pile wall - a series of soldier beams supporting horizontal lagging to retain an excavated surface; commonly used in limited right-of-way applications

soil interaction structure - a subsurface structure that incorporates both the strength properties of a flexible structure and the support properties of the soil surrounding the structure

sole plate - a plate attached to the bottom flange of a beam that distributes the reaction of the bearing to the beam

solid sawn beam – a section of tree cut to the desired size at a saw mill

sounding - determining the depth of water by an echo-sounder or lead line; tapping a surface to detect delaminations (concrete) or decay (timber)

spall - depression in concrete caused by a separation of a portion of the surface concrete, revealing a fracture parallel with or slightly inclined to the surface

span - the distance between the supports of a beam; the distance between the faces of the substructure elements; the complete superstructure of a single span bridge or a corresponding integral unit of a multiple span structure; see CLEAR SPAN

spandrel - the space bounded by the arch extrados and the horizontal member above it

spandrel column - a column constructed on the rib of an arch span and serving as a support for the deck construction of an open spandrel arch; see OPEN SPANDREL ARCH

spandrel fill - the fill material placed within the spandrel space of a closed spandrel arch

spandrel tie - a wall or a beam-like member connecting the spandrel walls of an arch and securing them against bulging and other deformation; in stone masonry arches the spandrel tie walls served to some extent as counterforts

spandrel wall - a wall built on the extrados of an arch filling the space below the deck; see TIE WALLS

special inspection - an inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency

specifications - a detailed description of requirements, materials, tolerances, etc., for construction which are not shown on the drawings; also known as specs

spider - inspection access equipment consisting of a bucket or basket which moves vertically on wire rope, driven by an electric or compressed air motor

spillway - a channel used to carry water away from the top of a slope to an adjoining outlet

splice - a structural joint between members to extend their effective length

splits - advanced checks that extended completely through the piece of wood

spread footing - a foundation, usually a reinforced concrete slab, which distributes load to the earth or rock below the structure

spring line - the horizontal line along the face of an abutment or pier at which the intrados of an arch begins

spurs - a projecting jetty-like construction placed adjacent to an abutment or embankment to prevent scour

stage - inspection access equipment consisting of a flat platform supported by horizontal wire-rope cables; the stage is then slid along the cables to the desired position; a stage is typically 20 inches wide, with a variety of lengths available

staged construction - construction performed in phases, usually to permit the flow of traffic through the site

state transportation department - the term “state transportation department” means that department, commission, board, or official of any State charged by its laws with the responsibility for highway construction

statics - the study of forces and bodies at rest

station - 100 feet (U.S. customary); 100 meters (metric)

stationing - a system of measuring distance along a baseline

stay-in-place forms - a corrugated metal sheet for forming deck concrete that will remain in place after the concrete has set; the forms do not contribute to deck structural capacity after the deck has cured; see FORMS, S.I.P FORMS

stay plate - a tie plate or diagonal brace to prevent movement

steel - an alloy of iron, carbon, and various other elements

stem - the vertical wall portion of an abutment retaining wall, or solid pier; see BREASTWALL

stiffener - a small member attached to another member to transfer stress and to prevent buckling

stiffening girder - a girder incorporated in a suspension bridge to distribute the traffic loads uniformly among the suspenders and reduce local deflections

stiffening truss - a truss incorporated in a suspension bridge to distribute the traffic loads uniformly among the suspenders and reduce local deflections

stirrup - U-shaped bar used as a connection device in timber and metal bridges; U-shaped bar placed in concrete to resist diagonal tension (shear) stresses

stone masonry - the portion of a structure composed of stone, generally placed in courses with mortar

straight abutment - an abutment whose stem and wings are in the same plane or whose stem is included within a length of retaining wall

strain - the change in length of a body produced by the application of external forces, measured in units of length; this is the proportional relation of the amount of change in length divided by the original length

strain hardening - the effect of increased yield strength when a material has been plastically deformed

strand - a number of wires grouped together usually by twisting

streambanks - the sloped sides of the channel

streambed - the bottom of the channel

streamflow - the water, suspended sediment and any debris moving through the channel

strengthening - adding to the capacity of a structural member

stress - the force acting across a unit area in a solid material

stress concentration - local increases in stress caused by a sudden change of cross section in a member

stress corrosion – occurs in metals with high tensile forces such as prestressed reinforcement exposed to contaminants such as chlorides

stress range - the variation in stress at a point with the passage of live load, from initial dead load value to the maximum additional live load value and back

stress raiser - a detail that causes stress concentration

stress reversal - change of stress type from tension (+) to compression (−) or vice versa

stress sheet - a drawing showing all computed stresses resulting from the application of a system of loads together with the design composition of the individual members resulting from the application of assumed unit stresses for the material to be used in the structure

stress-laminated timber – consists of multiple planks mechanically clamped together to perform as one unit

stringer - a longitudinal beam spanning between transverse floorbeams and supporting a bridge deck

strip seal joint - a joint using a relatively thin neoprene seal fitted into the joint opening

structural analysis - engineering computation to determine the carrying capacity of a structure

structural member - an individual piece, such as a beam or strut, which is an integral part of a structure

structural redundancy - the ability of an interior continuous span to resist total collapse by cantilever action in the event of a fracture

structural shapes - the various types of rolled iron and steel having flat, round, angle, channel, "I", "H", "Z" and other cross-sectional shapes adapted to heavy construction

structural stability - the ability of a structure to maintain its normal configuration, not collapse or tip in any way, under existing and expected loads

structural tee - a tee-shaped rolled member formed by cutting a wide flange longitudinally along the centerline of web

structurally deficient – bridges where 1) significant load carrying elements are found to be in poor or worse condition due to deterioration and/or damage or, 2) the adequacy of the waterway opening provided by the bridge is determined to be extremely insufficient to the point of causing intolerable traffic interruptions

structure - something, such as a bridge, that is designed and built to sustain a load

strut - a member acting to resist axial compressive stress; usually a secondary member

stub abutment - an abutment within the topmost portion of an embankment or slope having a relatively small vertical height and usually pile supported; stub abutments may also be founded on spread footings

subbase - a layer of material placed between the base course and the subgrade within a flexible pavement structure

subgrade - natural earth below the roadway pavement structure

sub-panel - a truss panel divided into two parts by an intermediate web member, generally a subdiagonal or a hanger

substructure - the abutments and piers built to support the span of a bridge superstructure

superelevation - the difference in elevation between the inside and outside edges of a roadway in a horizontal curve; required to counteract the effects of centrifugal force

superimposed dead load - dead load that is applied to a compositely designed bridge after the concrete deck has cured; for example, the weight of parapets or railings placed after the concrete deck has cured

superstructure - the entire portion of a bridge structure that primarily receives and supports traffic loads and in turn transfers these loads to the bridge substructure

surface breakdown - see scaling

surface corrosion - rust that has not yet caused measurable section loss

suspended span - a simple span supported from the free ends of cantilevers

suspender - a vertical wire cable, metal rod, or bar connecting the catenary cable of a suspension bridge or an arch rib to the bridge floor system, transferring loads from the deck to the main members

suspension bridge - a bridge in which the floor system is supported by catenary cables that are supported upon towers and are anchored at their extreme ends

suspension cable - a catenary cable which is one of the main members upon which the floor system of a suspension bridge is supported; a cable spanning between towers

swale - a drainage ditch with moderately sloping sides

sway anchorage - a guy, stay cable or chain attached to the floor system of a suspension bridge and anchored upon an abutment or pier to increase the resistance of the suspension span to lateral movement; also known as sway cable

sway bracing - diagonal brace located at the top of a through truss, transverse to the truss and usually in a vertical plane, to resist transverse horizontal forces

sway frame - a complete panel or frame of sway bracing

swedged anchor bolt - anchor bolt with deformations to increase bond in concrete; see ANCHOR BOLT

swing span bridge - a movable bridge in which the span rotates in a horizontal plane on a pivot pier, to permit passage of marine traffic

T

tack welds - small welds used to hold member elements in place during fabrication or erection

tail water - water ponded below the outlet of a waterway, thereby reducing the amount of flow through the waterway; see HEADWATER

tape measure - a long, flexible strip of metal or fabric marked at regular intervals for measuring

team leader - individual in charge of an inspection team responsible for planning, preparing, and performing field inspection of the bridge

tee beam - a rolled steel section shaped like a "T"; reinforced concrete beam shaped like the letter "T"

temperature steel - reinforcement in a concrete member to prevent cracks due to stresses caused by temperature changes

temporary bridge - a structure built for emergency or interim use, intended to be removed in a relatively short time

tendon - a prestressing cable, strand, or bar

tensile force - a force caused by pulling at the ends of a member; see TENSION

tensile strength - the maximum tensile stress at which a material fails

tension - stress that tends to pull apart material

thalweg elevation – lowest elevation of the streambed

thermal movement - contraction and expansion of a structure due to a change in temperature

three-hinged arch - an arch that is hinged at each support and at the crown

through arch - an arch bridge in which the deck passes between the arches

through girder bridge - normally a two-girder bridge where the deck is between the supporting girders

tie - a member carrying tension

tie plate - relatively short, flat member carrying tension forces across a transverse member; for example, the plate connecting a floor beam cantilever to the main floor beam on the opposite side of a longitudinal girder; see STAY PLATE

tie rod - a rod-like member in a frame functioning to transmit tensile stress; also known as tie bar

tie walls - one of the walls built at intervals above an arch ring connecting and supporting the spandrel walls; any wall designed to serve as a restraining member to prevent bulging and distortion of two other walls connected thereby; see DIAPHRAGM WALL

timber - wood suitable for construction purposes

toe - the front portion of a footing from the intersection of the front face of the wall or abutment to the front edge of the footing; the line where the side slope of an embankment meets the existing ground

toe of slope - the location defined by the intersection of the embankment with the surface existing at a lower elevation; also known as toe

toe wall - a relatively low retaining wall placed near the "toe of slope" location of an embankment to protect against scour or to prevent the accumulation of stream debris; also known as footwall

ton - a unit of weight equal to 2,000 pounds

torque - the angular force causing rotation

torque wrench - a hand or power tool used to turn a nut on a bolt that can be adjusted to deliver a predetermined amount of torque

torsion - twisting about the longitudinal axis of a member

torsional force - an external moment that tends to rotate or twist a member about its longitudinal axis

torsional rigidity - a beam's capacity to resist a twisting force along the longitudinal axis

toughness - a measure of the energy required to break a material

tower - a pier or frame supporting the catenary cables of a suspension bridge

traffic control - modification of normal traffic patterns by signs, cones, flagmen, etc.

transducer - a device that converts one form of energy into another form, usually electrical into mechanical or the reverse; the part of ultrasonic testing device which transmits and receives sound waves\

transient loads - temporary loads that change over time

transverse bracing - the bracing assemblage engaging the columns of bents and towers in planes transverse to the bridge alignment that resists the transverse forces tending to produce lateral movement and deformation of the columns

transverse girder - see CROSS GIRDER

travel way - the roadway

tremie - a piece of construction equipment (e.g., pipe or funnel) used to place concrete underwater

trestle - a bridge structure consisting of spans supported on braced towers or frame bents

truck loading - a combination of loads used to simulate a single truck passing over a bridge

truss - a jointed structure made up of individual members primarily carrying axial loads arranged and connected in triangular panels

truss bridge - a bridge having a pair of trusses for a superstructure

trussed beam - a beam stiffened to reduce its deflection by a steel tie-rod that is held at a short distance from the beam by struts

truss panel - see PANEL

tubular sections - structural steel tubes, rectangular, square or circular; also known as hollow sections

tubular truss - a truss whose chords and struts are composed of pipes or cylindrical tubes

tunnel - an underground passage, open to daylight at both ends

turnbuckle - a long, cylindrical, internally threaded nut with opposite hand threads at either end used to connect the elements of adjustable rod and bar members

two-hinged arch - a rigid frame that may be arch-shaped or rectangular with hinges at both supports

U

U-bolt - a bar bent in the shape of the letter "U" and fitted with threads and nuts at its ends

ultimate strength - the highest stress that a material can withstand before breaking

ultrasonic thickness gage - an instrument used to measure the thickness of a steel element using a probe which emits and receives sound waves

ultrasonic testing - nondestructive testing of a material's integrity using sound waves

undermining - the scouring away of stream and supporting foundation material from beneath the substructure footing

underpass - the lowermost feature of a grade separated crossing; see OVERPASS

underwater diver bridge inspection training - training that covers all aspects of underwater bridge inspection and enables inspectors to relate the conditions of underwater bridge elements to established criteria (see the Bridge Inspector's Reference Manual section on underwater inspection for the recommended material to be covered in an underwater diver bridge inspection training course).

underwater inspection - inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.

uniform load - a load of constant magnitude along the length of a member

unit stress - the force per unit of surface or crosssectional area

uplift - a negative reaction or a force tending to lift a beam, truss, pile, or any other bridge element upwards

upper chord - the top longitudinal member of a truss

V

vertical - describes the axis of a bridge perpendicular to the underpass surface

vertical alignment - a roadway's centerline or baseline alignment in the vertical plane

vertical clearance - the distance between the structure and the underpass

vertical curve - a sag or crest in the profile of a roadway, usually in the form of a parabola, to transition between grades

vertical lift bridge - a bridge in which the span moves up and down while remaining parallel to the roadway

viaduct - a series of spans carried on piers at short intervals

vibration - the act of vibrating concrete to compact it

Vierendeel truss - a truss with only chords and verticals joined with rigid connections designed to transfer moment

voided slab - a precast concrete deck unit cast with cylindrical voids to reduce dead load

voids - an empty or unfilled space in concrete

Voussoir - one of the truncated wedge-shaped stones composing a ring course in a stone arch; also known as ring stone

voussoir arch - an arrangement of wedge shaped blocks set to form an arched bridge

W

wale, waler - horizontal bracing running along the inside walls of a sheeted pit or cofferdam

Warren truss - a triangular truss consisting of sloping members between the top and bottom chords and no verticals; members form the letter W

washer - a small metal ring used beneath the nut or the head of a bolt to distribute the load or reduce galling during tightening

watercement ratio - the weight of water divided by the weight of portland cement in concrete; this ratio is a major factor in the strength of concrete

waterproofing membrane - an impervious layer placed between the wearing surface and the concrete deck, used to protect the deck from water and corrosive chemicals that could damage it

waterway area - the entire area beneath the bridge which is available to pass flood flows

waterway opening - the available width for the passage of water beneath a bridge

wear - gradual removal of surface mortar due to friction

wearing surface - the topmost layer of material applied upon a roadway to receive the traffic loads and to resist the resulting disintegrating action; also known as wearing course

web - the portion of a beam located between and connected to the flanges; the stem of a dumbbell type pier

web crippling - damage caused by high compressive stresses resulting from concentrated loads

web members - the intermediate members of a truss, not including the end posts, usually vertical or inclined

web plate - the plate forming the web element of a plate girder, built-up beam or column

web stiffener - a small member welded to a beam web to prevent buckling of the web

weephole - a hole in a concrete retaining wall to provide drainage of the water in the retained soil

weld - a joint between pieces of metal at faces that have been made plastic and caused to flow together by heat or pressure

weldability - the degree to which steel can be welded without using special techniques, such as pre-heating

welded bridge structure - a structure whose metal elements are connected by welds

welded joint - a joint in which the assembled elements and members are connected by welds

welding - the process of making a welded joint

weld layer - a single thickness of weld metal composed of beads (runs) laid in contact to form a pad weld or a portion of a weld made up of superimposed beads

weld metal - fused filler metal added to the fused structure metal to produce a welded joint or a weld layer

weld penetration - the depth beneath the original surface to which the structure metal has been fused in the making of a fusion weld; see PENETRATION

weld sequence - the order of succession required for making the welds of a built-up piece or the joints of a structure, to minimize distortion and residual stresses

weld toe - particularly in a filet weld, the thin end of the taper furthest from the center of the weld cross section

wheel guard - a raised curb along the outside edge of traffic lanes to safeguard constructions outside the roadway limit from collision with vehicles

wheel load - the load carried by and transmitted to the supporting structure by one wheel of a traffic vehicle, a movable bridge, or other motive equipment or device; see AXLE LOAD

weep hole - a hole in a concrete element (abutment backwall or retaining wall) used to drain water from behind the element; any small hole installed for drainage

Whipple truss - a double-intersecting through Pratt truss where the diagonals extend across two panels

wide flange - a rolled I-shaped member having flange plates of rectangular cross section, differentiated from an S-beam (American Standard) in that the flanges are not tapered

wind bracing - the bracing systems that function to resist the stresses induced by wind forces

wind lock - a lateral restraining device found on steel girder and truss bridges

wingwall - the retaining wall extension of an abutment intended to restrain and hold in place the side slope material of an approach roadway embankment

wire mesh reinforcement - a mesh made of steel wires welded together at their intersections used to reinforce concrete; welded wire fabric

wire rope - steel cable of multiple strands which are composed of steel wires twisted together

working stress - the unit stress in a member under service or design load

working stress design - a method of design using the yield stress of a material and a factor of safety that determine the maximum allowable stresses

wrought iron - cast iron that has been mechanically worked to remove slag and undissolved carbon

wythe - a single layer of brick or stone in the thickness direction

X

X-ray testing - nondestructive testing technique used for detecting internal flaws by passing X-rays through a material to film or other detector

Y

yield - permanent deformation (permanent set) which a metal piece takes when it is stressed beyond the elastic limit

yield point - see YIELD STRESS

yield stress - the stress at which noticeable, suddenly increased deformation occurs under slowly increasing load

yield strength - the stress level at which plastic deformation begins

Z

zee - steel member shaped like a modified "Z" in cross section

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