U.S. Department of Transportation
Federal Highway Administration

national highway institute

Bridge Inspector's Reference Manual (BIRM)

(2022 NBIS)

REVISED MARCH 2023

ACCREDITED IACET PROVIDER

PUBLICATION NO. FHWA-NHI-23-024

FOREWORD

The *Bridge Inspector's Reference Manual (BIRM), (2022 NBIS)* includes updated content based on the May 6, 2022 update to the National Bridge Inspection Standards (NBIS). The NBIS are required by statute (23 U.S.C. 144(h)) and defined in regulation (23 CFR Part 650 Subpart C). Applicable revisions were also made per the Federal Highway Administration's (FHWA) *Specifications for the National Bridge Inventory (SNBI)*, which was released in March 2022 and is incorporated by reference to the NBIS. The *SNBI* supersedes the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (Coding Guide)*. National Bridge Inventory data must be reported to FHWA using the *SNBI* after the annual submittal date specified in FHWA's Implementation Memo, May 25, 2022.

TECHNICAL REPORT DOCUMENTATION PAGE

<table>
<tr><td>1. Report No. FHWA-NHI-23-024</td><td>2. Government Accession No.</td><td>3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2">4. Title and Subtitle
Bridge Inspector's Reference Manual (BIRM)</td><td>5. Report Date
March 2023
6. Performing Organization Code:</td></tr>
<tr><td colspan="2">7. Author(s)
Thomas W. Ryan, P.E., Cassandra E. Lloyd, P.E., Michael S. Pichura, P.E., Darrin M. Tarasovich, P.E., and Sandy Fitzgerald</td><td>8. Performing Organization Report No.</td></tr>
<tr><td colspan="2">9. Performing Organization Name and Address
Michael Baker International, Inc.
Airside Business Park, 100 Airside Drive
Moon Township, PA 15108</td><td>10. Work Unit No.
11. Contract or Grant No.
DTFH6117D00036L–693JJ318F000369</td></tr>
<tr><td colspan="2">12. Sponsoring Agency Name and Address
Federal Highway Administration
National Highway Institute
2600 Park Tower Drive, Suite 500
Vienna, VA 22180</td><td>13. Type of Report and Period
Revised Reference Manual: March 2023
14. Sponsoring Agency Code
OTWDTD-IMEP-IWD-NHI</td></tr>
<tr><td colspan="3">15. Supplementary Notes
Performing Organization's Principal Investigator: Thomas W. Ryan, P.E.
Performing Organization's Technical Leads: Cassandra E. Lloyd, P.E., and Michael S. Pichura, P.E.
FHWA Contracting Officer's Representative: Melonie Barrington, Ed.D., M.E., P.E., PMP and Mignon J. Whitted
Team Leaders, FHWA Technical Review Team: Dennis O'Shea, P.E. and John Thiel, P.E.</td></tr>
</table>

16. Abstract

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

The *BIRM* is a revision and upgrade of the previous manual conforming with FHWA *Specifications for the National Bridge Inventory* (*SNBI*). Improved bridge inspection techniques are presented, and state-of-the-art inspection equipment is included. New or expanded coverage is provided on culverts, nonredundant members, fatigue-prone details, redundancy, cable stayed bridges, prestressed segmental bridges, movable bridge inspection, underwater inspection, and nondestructive evaluation, critical findings, and inspection intervals.

A series of NHI courses on bridge inspection, based on the *BIRM*, have been developed. These courses include a two-week course, FHWA-NHI-130055, "Safety Inspection of In-Service Bridges" and a one-week course designed specifically for professional engineers (PEs), FHWA-NHI-130056, "Safety Inspection of In-Service Bridges for Professional Engineers." Together, these two courses meet the definition of approved comprehensive training in bridge inspection as defined in the National Bridge Inspection Standards (NBIS).

The catalog for NHI Courses can be found on the National Highway Institute website: www.nhi.fhwa.dot.gov

<table>
<tr><td colspan="2">17. Key Words
Bridge Inspection, Bridge Evaluation, Element Level Evaluation, Component Rating, Culvert Inspection, Critical Findings, Nonredundant Steel Tension Members, Underwater Inspection</td><td colspan="2">18. Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.
http://www.ntis.gov</td></tr>
<tr><td>19. Security Classification (of this report) Unclassified</td><td>20. Security Classification (of this page) Unclassified</td><td>21. No. of Pages
1412</td><td>22. Price</td></tr>
</table>

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

This page intentionally left blank.

LIST OF ACRONYMS/ABBREVIATIONS

AADT	annual average daily traffic
AADTT	annual average daily truck traffic
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
AASHTO FCP	American Association of State Highway and Transportation Officials Fracture Control Plan
ACC	acid copper chromate
ACQ	alkaline copper quaternary
ACZA	ammoniacal copper zinc arsenate
ADC	Association of Diving Contractors International
ADE	Agency Defined Element
ADT	average daily traffic
ADTT	average daily truck traffic
AE	Acoustic Emission
AED	automatic external defibrillator
AISC	American Institute of Steel Construction
ANSI	American National Standards
ASD	Allowable Stress Design
ASR	Alkali-Silica Reaction
ASR	Allowable Stress Rating
ASTM	American Society for Testing and Materials
BIRM	Bridge Inspector's Reference Manual
BMS	Bridge Management System
BME	Bridge Management Element
CBSI	Certified Bridge Safety Inspector
CCA	chromated copper arsenate
CFR	Code of Federal Regulations
CFRP	carbon fiber reinforced polymer
CIP	cast-in-place

CMP	corrugated metal pipe
CPG	Crack Propagation Gage
CPR	cardiopulmonary resuscitation
CVN	Charpy V-notch
E	Modulus of Elasticity
EDM	Electronic Distance Meter
EFS	Electrical Fatigue Sensor
EIC	environmentally induced cracking
ER	Electrical Resistivity
ET	Eddy Current Testing
EV2/EV3	Emergency Vehicle 2/3
F	Fahrenheit
FAA	Federal Aviation Administration
FAST Act	Fixing America's Surface Transportation Act
FHWA	Federal Highway Administration
FRC	fiber reinforced concrete
FRP	Fiber Reinforced Polymers
ft	feet
GFRP	Glass Fiber Reinforced Polymer
GPR	Ground-Penetrating Radar
GPM	Galvanostatic Pulse Measurement
GRS	Geosynthetic Reinforced Soil
HCP	Half-Cell Potential
HDPE	High Density Polyethylene
HDS-6	Hydraulic Design Series Number 6
HEC-18	Hydraulic Engineering Circular No. 18
HIC	hydrogen-induced cracking
HMWM	High Molecular Weight Methacrylate or Methyl Methacrylate
HPC	High Performance Concrete
HPMS	Highway Performance Monitoring System

HPS	High Performance Steel
IE	Impact Echo
IR	Infrared Thermography
ISTEA	Intermodal Surface Transportation Efficiency Act
k	kip
ksi	kips per square inch
lbs	pounds
LFD	Load Factor Design
LMC	latex modified concrete
LPR	Linear Polarization
LRFD	Load and Resistance Factor Design
LRFR	Load and Resistance Factor Rating
LRS	Linear Referencing System
LSDC	low slump dense concrete
LTBP	Long-Term Bridge Performance
LVL	laminated veneer lumber
LWC	lightweight concrete
MAP-21	Moving Ahead for Progress in the 21st Century Act
MASH	AASHTO Manual for Assessing Safety Hardware
MBE	AASHTO Manual for Bridge Evaluation
MBEI	AASHTO Manual for Bridge Element Inspection
MFL	Magnetic Flux Leakage
MM	Magnetometer
MPT	Maintenance and Protection of Traffic
MSE	Mechanically Stabilized Earth
MSR	mechanical stress rating or grading
MT	Magnetic Particle Testing
MUTCD	Manual on Uniform Traffic Control Devices for Streets and Highways
NBE	National Bridge Element
NBI	National Bridge Inventory

NBIP	National Bridge Inspection Program
NBIS	National Bridge Inspection Standards
NCHRP	National Cooperative Highway Research Program
NDE	Nondestructive Evaluation
NDS	National Design Specifications
NDT	Nondestructive Testing
NHI	National Highway Institute
NHS	National Highway System
NRL	Notional Rating Load
NSTM	Nonredundant Steel Tension Member
NTSB	National Transportation Safety Board
OSHA	Occupational Safety and Health Administration
PAUT	Phased Array Ultrasonic Testing
pcf	pounds per cubic foot
PCI	Prestressed Concrete Institute
PE	Professional Engineer
POA	Plan of Action
PPC	Polyester Polymer Concrete
PPE	Personal Protective Equipment
psi	pounds per square inch
PSL	parallel strand lumber
PT	Dye Penetrant Testing
PTFE	polytetrafluoroethylene
QA	Quality Assurance
QC	Quality Control
RABIT™	Robotically Assisted Bridge Deck Assessment Tool
RC	Reinforced concrete
ROV	Remote Operated Vehicle
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users

SCC	Self-Consolidating Concrete
SCC	stress corrosion cracking
SCL	structural composite lumber
SCUBA	Self-contained underwater breathing apparatus
SHM	structural health monitoring
SHRP	Strategic Highway Research Program
SHV	Special Hauling Vehicle
SI&A	Structure Inventory and Appraisal
SIP	Stay-in-place
SNBI	Specifications for the National Bridge Inventory
SPRAT	Society of Professional Rope Access Technicians
SRM	System Redundant Member
SSPC-VIS 2	Society for Protective Coatings Guide to Visual Standard Number 2
SU4	Single Unit 4 Truck
SU5	Single Unit 5 Truck
SU6	Single Unit 6 Truck
SU7	Single Unit 7 Truck
TEA-21	Transportation Equity Act of the 21st Century
TL	test level
TMA	truck mounted attenuator
TRB	Transportation Research Board
UAS	Unmanned Aerial Systems
UAV	Unmanned Aerial Vehicle
UHPC	Ultra High-Performance Concrete
UPE	Ultrasonic Pulse Echo
UPV	Ultrasonic Pulse Velocity
USGS	United States Geological Survey
USV	Unmanned Surface Vehicles
USW	Ultrasonic Surface Waves
UT	Ultrasonic Testing

UV	Ultraviolet
VARTM	vacuum assisted resin-transfer molding
V-WAC	visual weld acceptance criteria

Table of Contents

SECTION

SECTION

SECTION

LIST OF FIGURE AND TABLE SOURCES

CHAPTER 1 - LIST OF FIGURE SOURCES

CHAPTER 1 - LIST OF TABLE SOURCES

FHWA = Federal Highway Administration

CHAPTER 2 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
MBI = Michael Baker International

CHAPTER 3 - LIST OF FIGURE SOURCES

CHAPTER 3 - LIST OF TABLE SOURCES

AASHTO = American Association of State Highway and Transportation Officials
MBI = Michael Baker International

CHAPTER 4 - LIST OF FIGURE SOURCES

MBI = Michael Baker International
FHWA = Federal Highway Administration

CHAPTER 5 - LIST OF FIGURE SOURCES

MBI = Michael Baker International
FHWA = Federal Highway Administration

CHAPTER 6 - LIST OF FIGURE SOURCES

CHAPTER 6 - LIST OF TABLE SOURCES

AASHTO = American Association of State Highway and Transportation Officials
FHWA = Federal Highway Administration
MBI = Michael Baker International

CHAPTER 7 - LIST OF FIGURE SOURCES

CHAPTER 7 - LIST OF TABLE SOURCES

AASHTO = American Association of State Highway and Transportation Officials
FHWA = Federal Highway Administration
MBI = Michael Baker International
PennDOT = Pennsylvania Department of Transportation
PTC = Pennsylvania Turnpike Commission

CHAPTER 8 - LIST OF FIGURE SOURCES

CHAPTER 8 - LIST OF TABLE SOURCES

MBI = Michael Baker International
FHWA = Federal Highway Administration
WVDOH = West Virginia Division of Highways
WVU = West Virginia University

CHAPTER 9 - LIST OF FIGURE SOURCES

AASHTO = American Association of State Highway and Transportation Officials
FHWA = Federal Highway Administration
MBI = Michael Baker International
NJDOT = New Jersey Department of Transportation
PennDOT = Pennsylvania Department of Transportation

CHAPTER 10 - LIST OF FIGURE SOURCES

Caltrans = California Department of Transportation
FHWA = Federal Highway Administration
MBI = Michael Baker International
PTC = Pennsylvania Turnpike Commission

CHAPTER 11 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
MBI = Michael Baker International
WVDOH = West Virginia Department of Highways
WVU = West Virginia University

CHAPTER 12 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
MBI = Michael Baker International

CHAPTER 13 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
MBI = Michael Baker International
PennDOT = Pennsylvania Department of Transportation

CHAPTER 14 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
KYTC = Kentucky Transportation Cabinet
MBI = Michael Baker International
TxDOT = Texas Department of Transportation

CHAPTER 15 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
MBI = Michael Baker International
NJDOT = New Jersey Department of Transportation

CHAPTER 16 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
KYTC = Kentucky Transportation Cabinet
MBI = Michael Baker International
TxDOT = Texas Department of Transportation

CHAPTER 17 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
MBI = Michael Baker International
MDOT = Michigan Department of Transportation
NHI = National Highway Institute

CHAPTER 18 - LIST OF FIGURE SOURCES

FHWA = Federal Highway Administration
MBI = Michael Baker International

PART I – BRIDGE SAFETY INSPECTION PROGRAM

CHAPTER 1 TABLE OF CONTENTS

CHAPTER 1 LIST OF FIGURES

CHAPTER 1 LIST OF TABLES

Chapter 1 Bridge Inspection Program

Section 1.1 History of the National Bridge Inspection Program

1.1.1 Background

Many of the nation's bridges were constructed after the Federal Aid Highway Act of 1956 was signed into law. The system of roadways then known as the Dwight D. Eisenhower National System of Interstate and Defense Highways is now commonly referred to as the Interstate Highway System. The intention of the proposal was to provide a nationwide network of safe highways to accommodate the increase in transportation by automobile. The funding associated with this Act led to a massive effort to construct over 48,000 miles of roadway and the bridges necessary for the routes. Many of the structures were built in the years following the passage of the Act. Approximately 45 percent of interstate bridges, including both urban and rural, were constructed between 1961 and 1970.

During the bridge construction boom of the 1950s and 1960s, 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 1.1.1).

Figure 1.1.1 Silver Bridge Failure, 1967

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-Aid Highway Act of 1968" that 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.

Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

The 1970s

In 1971, the Federal regulation defining the National Bridge Inspection Standards (NBIS) was developed. It established national requirements regarding (CFR Part 650, Title 23, Subpart C):

- Inspection organization.
- Qualifications of personnel.
- Frequency of inspections.
- Inspection procedures, including load rating.
- 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's (FHWA) *Bridge Inspector's Training Manual 70 (Manual 70)*. This manual set the standard for inspector training. This was the original predecessor of the *Bridge Inspector's Reference Manual (BIRM)*.

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's *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 FHWA's *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 progressed. Several states began in-house training programs, and bridge inspections became more thorough and consistent across the country in the 1970s. 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 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 crucial funding for rehabilitation and new construction and required that all public bridges over 20 ft in length be inspected and inventoried in accordance with the NBIS by December 31, 1980. Any bridge not inspected and inventoried in compliance with the NBIS would be ineligible for Federal funding.

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

The 1980s

The National Bridge Inspection Program was now maturing and well positioned for the coming decade. Two additional supplements to *Manual 70* were published in response to a few problem areas that surfaced during this time. 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 in bridges came about with the collapse of Connecticut's Mianus River Bridge in June 1983 (see Figure 1.1.2). *Inspection of Fracture Critical Bridge Members* was published in September 1986. These manuals were the products of lessons learned and ongoing research in these problem areas.

Figure 1.1.2 Mianus Bridge Failure

With the April 1987 collapse of New York's Schoharie Creek Bridge as a result of scour, national focus turned to underwater inspection and scour vulnerability. Of the over 593,000 bridges in the national inventory, over 80 percent 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 FHWA's *Hydraulic Engineering Circular No. 18 (HEC-18)*.

In September 1988, the NBIS was revised based on 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 and bridges with special or unique features. The NBIS revisions also provided 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 inspection team leader qualifications.

In December 1988, the FHWA issued a revision to the *Coding Guide*. This revision would be one of major proportions, supporting the NBIS changes and shaping the National Bridge Inspection Program for the next decade.

The 1990s

The 1990s was the decade for development of Bridge Management Systems (BMS). Several states developed their own comprehensive bridge management systems, which relied heavily on bridge inspection data.

FHWA's *Bridge Inspector's Reference Training Manual 90 (Manual 90)* was published July 1991 and replaced *Manual 70*. Improved bridge inspection techniques were presented, and state-of-the-art inspection equipment was included. New or expanded coverage was provided on culverts, fracture critical members, cable-stayed bridges, prestressed segmental bridges, and underwater inspection.

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 had sufficient flexibility to enable 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 was primarily targeted to smaller bridge inventories or local highway systems.

As more and more bridge needs were identified, the need for bridge maintenance, repair, rehabilitation, and replacement far exceeded the available funding from Federal and state resources. Even with the infusion of financial support provided by the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, funding for bridge projects was difficult to obtain. This was due in part to the enormous demand from across the nation.

A December 1992 revision to the NBIS permitted bridge owners to request approval from FHWA of extended inspection cycles of up to forty-eight months for bridges meeting certain requirements. This revision enabled structures in good condition, with low risk factors, to be inspected less frequently and provide more time for other bridges.

In 1994, AASHTO revised their *Manual for Condition Evaluation of Bridges*. In 1995, the FHWA *Coding Guide* was also revised. These publications, along with *Manual 90,* Revised July 1995, continue to provide state agencies with definitive guidelines for compliance with the NBIS and conducting bridge inspections.

Although later rescinded in the next transportation bill, the ISTEA legislation required that each state implement a comprehensive BMS by October 1995. This deadline was challenging since few states had implemented systems that could meet the definition of a comprehensive BMS.

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 2000s

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 incorporated *Manual 70* supplements for culvert inspection and fracture critical members.

On December 14, 2004, the revised NBIS regulation was published in the *Federal Register*. Significant changes in the 2004 revision include requirements to identify bridges that are scour critical and develop scour Plans of Action (POA). It outlined specialized procedures for inspection of complex features and recommendations for addressing critical findings. Quality Assurance and Quality Control (QA/QC) procedures were implemented. There were also updates to the required frequency for certain underwater inspections, permitting them to performed on up to 72-month intervals. The updated NBIS took affect January 13, 2005.

The *Manual for Bridge Evaluation (MBE)* was first adopted by the AASHTO Highways Subcommittee on Bridges and Structures in 2005. The *MBE* combined the AASHTO *Manual for Condition Evaluation of Bridges* with the *Guide Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* to provide owners with a single document for evaluating and load rating bridges.

The Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (SAFETEA-LU) was signed into law in August 2005. SAFETEA-LU represented the largest surface transportation investment in the Nation's history. SAFETEA-LU built on and improved the initiatives established in ISTEA and TEA-21.

On August 1, 2007, the eight-lane, 1,907-foot-long I-35W highway bridge, over the Mississippi River in Minneapolis collapsed resulting in the death of 13 people and another 145 people injured. Major safety issues that were identified during the investigation included insufficient quality control processes during design, and inadequate identification of gusset plate distortion.

The 2010s

On July 6, 2012, MAP-21 (Moving Ahead for Progress in the 21st Century Act) was signed into law. This Act provided funding for surface transportation programs. MAP-21 declared "it is in the vital interest of the United States to inventory, inspect, and improve the condition of highway bridges and tunnels of the United States." It also required bridge and tunnel inspection standards and inventories (for the first time for tunnels), element level inspection data for bridges on the NHS, and reporting critical findings on bridges. It also required FHWA to develop and perform an annual review of compliance with the NBIS and was the first long term highway authorization enacted since 2005.

On December 4, 2015, FAST Act (Fixing America's Surface Transportation Act) was signed into law. The FAST Act authorized $305 billion over fiscal year 2016 through 2020 for highway, highway and motor vehicle safety, public transportation, motor carrier safety, hazardous materials safety, rail, and research, technology, and statistics programs. Several new load rating vehicles were developed to represent the emergency and towing vehicles covered within this Act.

The *BIRM* was previously updated in 2012 and 2015.

The 2020s

In May 2022, Title 23 Code of Federal Regulations (CFR) 650 Subpart C was revised for the NBIS Final Rule. This Final Rule included updates to applicability of the NBIS, definitions, inspection organization responsibilities, personnel qualifications, inspection intervals, procedures, and inventory.

The 2022 update included several updates to terminology and policy for bridge inspection. The applicability of the NBIS regulations was extended to Tribally owned bridges, privately owned bridges that are connected to public roads on both ends, and temporary bridges or bridges under construction with portions open to traffic. A major terminology example is the change from the term "fracture critical member" to "nonredundant steel tension member" to describe steel members in tension without redundancy. In turn, "FCM inspection" is now "NSTM inspection" as a result. Two new inspection types were added: Service and Scour Monitoring.

Also included are some updates the personnel qualifications, such as specific training for NSTM and Underwater inspections and updated team leader qualifications. In addition, bridge inspection organizations are now required to maintain a registry of nationally certified bridge inspectors. The inspection interval section was updated to provide more specific guidance both on risk assessments and extended inspection intervals.

Also in May 2022, the FHWA published the *Specifications for the National Bridge Inventory* (*SNBI*) which established updated data collection and processing procedures that are compliant with the NBIS Final Rule. It also incorporated the *Specification for National Bridge Inventory Bridge Elements* (*SNBIBE*) document by including the element information to be reported within the *SNBI*. The *SNBI* is intended to replace the FHWA Coding Guide after a transition period.

The BIRM was updated one final time in accordance with the FHWA Coding Guide in 2022 and was then updated in accordance with the *SNBI* and NBIS Final Rule in 2023.

1.1.2 Today's National Bridge Inspection Program

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 650, Title 23, Subpart C which is on the FHWA Office of Bridges and Structures website (https://www.fhwa.dot.gov/bridge/nbis.cfm).

Much has been learned in the field of bridge inspection, and a national Bridge Inspection Training Program is now fully implemented. Inspection efforts across the nation are more organized, managed in a more detailed manner, and much broader in scope. The technology used to inspect and evaluate bridge members and bridge materials has significantly improved. Refer to Chapter 2, Inspection Fundamentals, for examples of new inspection techniques.

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 and ratings have been refined to enhance documentation, and to increase uniformity, thoroughness, and consistency of both. Data from bridge inspections has become critical input into a variety of analyses and decisions by state agencies and the FHWA.

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

- Purpose.
- Applicability.
- Definitions.
- Bridge inspection organization responsibilities.
- Qualifications of personnel.
- Inspection interval.
- Inspection procedures.
- Inventory.
- Incorporation by reference.

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

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 elements, and to make sound evaluations with accurate ratings regarding safety, serviceability, and condition. 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 should continue to be, an active process within agencies. This manual is designed to be an integral part of that training process.

The FHWA has developed and offers the following training courses relative to bridge inspection and ancillary highway structure inspection through the National Highway Institute (NHI). Reference https://www.nhi.fhwa.dot.gov/course-search?tab=0 for full course descriptions. The information within the parenthesis indicates NHI Course Number.

- **"Safety Inspection of In-Service Bridges" (FHWA-NHI-130055C and FHWA-NHI-130055).**
    - Ten-day course is based on the FHWA *BIRM*, with an emphasis on inspection applications and procedures.
    - Intended audience: Inspectors or engineers who perform or manage bridge inspections. This course qualifies as an approved comprehensive bridge inspection training course required for Team Leaders and Program Managers (per 23 CFR 650.309).
    - Possible prerequisites include: "Engineering Concepts for Bridge Inspectors", "Introduction to Safety Inspection of In-Service Bridges".
    - FHWA-NHI-130055C is based on *Coding Guide* content and FHWA-NHI-130055 is based on *SNBI* content.
- **"Safety Inspection of In-Service Bridges for Professional Engineers" (FHWA-NHI-130056C and FHWA-NHI-130056).**
    - Five-day course is based on the *BIRM* but is a streamlined version of FHWA-NHI 130055 and designed to more properly suit experienced Professional Engineers.
    - Intended audience: Certified professional engineers (PE) becoming certified Bridge Inspection Team Leaders or Program Managers. This course also qualifies as an approved comprehensive bridge inspection training course required for Team Leaders and Program Managers (per 23 CFR 650.309).
    - Possible prerequisites include: "Engineering Concepts for Bridge Inspectors", "Introduction to Safety Inspection of In-Service Bridges".
    - FHWA-NHI-130056C is based on *Coding Guide* content and FHWA-NHI-130056 is based on *SNBI* content.
- **"Engineering Concepts for Bridge Inspectors" (FHWA-NHI-130054).**
    - Five-day course that presents engineering concepts, as well as inspection procedures and information about bridge types, components, and materials.
    - Intended audience: New inspectors with little or no practical bridge inspection experience.
    - This is a prerequisite for the 130055 and 130056 courses.

- **"Introduction to Safety Inspection of In-Service Bridges" (FHWA-NHI-130101).**
  - Web-based course that presents engineering concepts, as well as inspection procedures and information about bridge types, bridge components, and bridge materials.
  - Intended audience: New inspectors with little or no practical bridge inspection experience.
  - This is a prerequisite for the 130055 and 130056 courses.
- **"Bridge Inspection Refresher Training" (FHWA-NHI-130053C and FHWA-NHI-130053).**
  - Three-day course that 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.
  - Intended audience: Certified bridge inspectors that wish to maintain their certification.
  - The prerequisite for this course is to successfully complete FHWA's 130055 (or 130055S) Safety Inspection of In-Service Bridges, or 130056 (or 130056S) Safety Inspection of In-Service Bridges for Professional Engineers, or a similar FHWA-approved bridge inspection course equivalent.
  - This course fulfills the bridge inspection refresher training requirement in the NBIS (per 23 CFR 650.309 (h)(1)(ii)).
  - FHWA-NHI-130053C is based on *Coding Guide* content and FHWA-NHI-130053 is based on *SNBI* content.
- **"Underwater Bridge Inspection" (FHWA-NHI-130091).**
  - Four-day course providing an overview of diving operations and for managing underwater bridge inspections.
  - Intended audience: Inspector divers performing or overseeing underwater type inspections. This course fulfills the requirement of the National Bridge Inspection Standards, which require underwater bridge inspection training for all underwater bridge inspection divers conducting underwater inspections.
- **"Bridge Inspection Techniques for Nonredundant Steel Tension Members" (FHWA-NHI-130078).**
  - Three-day course that provides an understanding of nonredundant steel tension members (NSTMs), NSTM identification, failure mechanics and fatigue and fracture in metal.
  - Intended audience: Bridge inspectors who inspect structures with NSTMs. An emphasis is placed on inspection procedures and reporting of common NSTMs and nondestructive evaluation (NDE) methods most often associated with steel highway bridges.
  - This course fulfills the requirement of the NBIS for those conducting NSTM inspections to be trained in all aspects of NSTM inspections to relate conditions observed on a bridge to established criteria (per 23 CFR 650.309 (h)(1)(iv)).

- **"Bridge Inspection Nondestructive Evaluation Seminar (BINS)" (FHWA-NHI-130099A).**
    - Two-day course that provides bridge inspectors and managers the ability to learn about the latest in commercially available nondestructive tools and systems for use on bridges.
    - The seminar is presented through a series of slides, instructional videos, and video demonstrations showing basic operation of the equipment.
    - The training has been fully developed in conjunction with the FHWA's NDE Validation Center and is delivered by qualified instructors experienced in using NDE equipment on bridges.
    - Intended audience: Federal, state, and local highway bridge inspectors, bridge management staff, and consultants. Individuals involved in material testing, as well as transportation structure design and construction, will likely find the information useful to ensure quality.
- **"Stream Stability and Scour at Highway Bridges" (FHWA-NHI-135046).**
    - Three-day course that 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.
    - Intended audience: Federal, State, Tribal, and local highway hydraulic, structural, and geotechnical engineers as well as bridge inspectors responsible for maintaining the integrity of highway bridges against possible hydraulic-related problems. Consultants who perform bridge engineering work.
- **"Stream Stability and Scour at Highway Bridges for Bridge Inspectors" (FHWA-NHI-135047).**
    - One-day course that is an abbreviated presentation of 135046 Stream Stability and Scour at Highway Bridges. The course provides an understanding of and assistance in detecting hydraulic-related problems at highway bridges. The course emphasizes inspection guidelines to complete the hydraulic and scour-related coding requirements of the National Bridge Inspection Standards (NBIS).
    - Intended audience: Practicing bridge inspectors.
- **"Inspection and Maintenance of Ancillary Highway Structures" (FHWA-NHI-130087).**
    - One-day course that provides training in the inspection and maintenance of ancillary structures, such as structural supports for highway signs, luminaries, and traffic signals.
    - Intended audience: Inspectors and engineers who inspect and maintain light and sign support structures.

Current Reference Material

- NBIS. *Code of Federal Regulations.* 23 Highways Part 650, Subpart C – National Bridge Inspection Standards, 2022.
- AASHTO. *LRFD Bridge Design Specifications, 8th Edition*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2017.
- FHWA. *Specifications for the National Bridge Inventory.* Washington, D.C.: United States Department of Transportation, 2022.
- AASHTO. *The Manual for Bridge Evaluation, Third Edition*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2018.
- AASHTO. *The Manual for Bridge Evaluation, 2019 Interim Revisions (to 2018 Third Edition).* Washington, D.C.: American Association of State Highway and Transportation Officials, 2018.
- AASHTO. *The Manual for Bridge Evaluation, 2020 Interim Revisions (to 2018 Third Edition)*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2020.
- AASHTO. *Manual for Bridge Element Inspection, Second Edition*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2019.

Information pertaining to the NBIS Final Rule and the *SNBI* can be found at the following link: https://www.fhwa.dot.gov/bridge/nbis2022.cfm.

Section 1.2 Qualifications and Responsibilities of the Bridge Inspection Program Organization

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 section presents the qualifications for bridge inspection personnel and some of the responsibilities of the inspection team leader and program manager.

1.2.2 Qualifications of Personnel Involved in Bridge Inspection Activities

The NBIS is 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*. These are minimum standards. State or local highway agencies can implement additional requirements at their discretion.

Program Manager

The program manager oversees the organizational unit responsible for bridge inspection, reporting, and inventory and has the overall responsibility to ensure the program conforms with the requirements of this subpart. The program manager provides overall leadership and is available to inspection team leaders to provide guidance. (23 CFR 650.305). This oversight encompasses the entire state. The minimum qualifications from 23 CFR 650.309 (a) are as follows:

1. Be a registered Professional Engineer, or have 10 years of bridge inspection experience;
2. Complete an FHWA-approved comprehensive bridge inspection training course as described in paragraph (h) of this section (of the CFR) and score 70 percent or greater on an end-of- course assessment (completion of FHWA-approved comprehensive bridge inspection training under FHWA regulations in this subpart in effect before June 6, 2022, satisfies the intent of this requirement);
3. Complete a cumulative total of 18 hours of FHWA-approved bridge inspection refresher training over each 60 month period;
4. Maintain documentation supporting the satisfaction of the preceding requirements; and
5. Satisfy the requirements of this paragraph (a) within 24 months from June 6, 2022, if serving as a program manager who was qualified under prior FHWA regulations in this subpart.

Team Leader

The team leader is the on-site, nationally certified bridge inspector in charge of an inspection team and responsible for planning, preparing, performing, and reporting on bridge field inspections (23 CFR 650.305). The team leader coordinates 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, NSTM, underwater, and special inspection (23 CFR 650.313 (j)). There are four ways to qualify as a team leader which are all combinations of experience, education, and certifications (23 CFR 650.309 (b)). Team leaders are also required to successfully complete an FHWA approved comprehensive bridge inspection training course and subsequent refresher trainings. There are additional qualifications for team leaders on NSTM inspections per 23 CFR 650.309 (c), which include completing an FHWA-approved training course on the inspection of NSTMs. The team leader must satisfy these requirements within 24 months from June 6, 2022, if serving as a team leader who was qualified under prior FHWA regulations (23 CFR 650.309 (b)(5)).

Bridge Inspector (Non-Team Leader)

There are no specific NBIS requirements for bridge inspector assistants (non-team leaders). Some state DOTs have requirements to be a certified bridge inspector. 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. Bridge inspectors may also assist the team leader in completing report documentation.

Underwater Bridge Inspection Diver

The underwater bridge inspection diver is the individual that performs that inspection of the underwater portion of the bridge (23 CFR 650.305). Per 23 CFR 650.309 (e), an Underwater Bridge Inspection Diver must complete FHWA-approved underwater bridge inspection training and score 70 percent or greater on an end-of-course assessment. The underwater bridge inspection diver must meet these requirements before June 6, 2022.

Damage, Special, and Service Inspections

Like other types of inspections, Damage, Special, and Service Inspections should be performed by competent personnel. Qualifications for those performing Damage and Special Inspections must be established and documented by State transportation departments, Federal agencies, and Tribal governments (23 CFR 309 (f)). Agencies are also responsible for establishing documented personnel qualifications for Service Inspections, which apply when risk-based routine inspection intervals exceed 48 months (23 CFR 309 (g)).

1.2.3 Responsibilities of the Inspection Team Leader and Program Manager

There are five basic functions of a bridge safety inspection program:

- 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 inspection team 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 may be violated.

The program manager's role should be to manage the bridge inspection program agency-wide to include:

- Develop and implement bridge inspection policies and procedures.
- Bridge inspections and reporting.
- Preparation and maintenance of the inventory.
- Quality assurance and quality control program.
- Developing and maintaining qualified staff.
- Provide leadership and guidance to inspection team leaders.
- Documenting and overseeing any NBIS requirements that have been delegated to other individuals or entities.
- Report critical findings.

The team leader's role should be:

- Planning, preparing, performing, and reporting on field inspection of the bridge.
- Documentation of bridge conditions and deficiencies, and preparation of the final report.
- Alert supervisors or program managers of any findings that impact the integrity of the structure to ensure the safety of the traveling public.

Protect Public Investment

Another responsibility is to protect public investment in bridges. Bridge inspection programs are funded by public tax dollars. Therefore, all parties involved with bridge inspection are financially responsible to the public.

Inspectors should be on guard for minor problems that can be corrected before they lead to costly major repairs. Also, inspectors should be able to recognize bridge elements that necessitate 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 nation's demands. 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 program manager's role should be:

- To recommend upgrades to design standards to promote longevity of bridge performance, such as the implementation of high-performance materials and enhanced bridge joints.
- Facilitate discussion with subject matter experts such as load rating engineers, geotechnical engineers, and hydraulics engineers to aid in the decision-making process. Particularly involve the hydraulics and geotechnical engineers with regards to observed scour.

The inspection team's role should be:

- To continually be mindful of minor problems that could become costly repairs.
- To recognize bridge components that should be repaired in order to maintain bridge safety and avoid the need for costly replacement.
- To make recommendations to the Program Manager and Load Rating Engineer with regards to the restriction or closure of a bridge if necessary.

Provide Bridge Inspection Program Support

Ensuring compliance with the Code of Federal Regulations is an important function of a bridge inspection program and therefore its team members.

Subpart C of the National Bridge Inspection Standards (NBIS) of the *Code of Federal Regulations*, 23 Highways Part 650 sets the national minimum standards for the proper safety inspection and evaluation of all highway bridges and provides the requirements for bridge inspection programs.

Maintain Accurate Bridge Records

There are four major reasons why accurate bridge records are necessary:

1. A structure history file facilitates the identification and/or monitoring of deficiencies. As an example, reference the inspection results below for two bridge abutments that were measured for tilt during several inspection cycles.

Table 1.2.1 Bridge Abutment Movement Per Given Year

Year	Abutment A	Abutment B
2019	4-3/16”	3-1/2”
2017	4-3/16”	2-1/4”
2015	4-1/8	1-1/8”
2013	4”	1”

   Looking at the measurements from 2019 may 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. However, 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.
2. To identify and assess bridge deficiencies and repair needs. To readily determine, from the records, what repairs are necessary as well as an estimate of quantities. Maintain bridge inspection reports, with notations of any action taken to address the findings of such inspections, to address maintenance, preservation, or repair needs.
3. 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.
4. To maintain a good bridge condition, including sufficient load carrying capacity to guarantee public safety and facilitate the routing of overweight/over-height vehicles.

To ensure accurate bridge records, proper record keeping should be maintained. Quality assurance and control systems should be developed to review bridge data and evaluate the quality of bridge inspections, refer to Section 1.3. Bridge files should be prepared as described in the AASHTO *MBE*. Inspectors should record the findings and results of bridge inspections on standard agency forms.

Fulfill Legal Responsibilities

A bridge inspection report is a legal document. Inspectors should make descriptions specific, detailed, quantitative (where possible), and complete. Vague adjectives, without concise descriptions to back them up, should not be used.

Some examples of inspection finding descriptions:

Poor 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”.

Poor description: “Deck in poor condition”.

Good description: “Deck in poor condition with spalls covering 50 percent of the top surface area of the deck as indicated on field sketch, see Figure 42”.

Poor description: “The bridge is dangerous”.

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

Phrases such as “no other apparent defects” or “no other defects observed” should be included in any visual assessment.

Inspectors should not alter original inspection notes unless there is approval from the inspector who wrote the notes.

A bridge inspection report implies that the inspection was performed in accordance with the NBIS, unless specifically stated otherwise in the report. The proper equipment, methods, and qualified personnel should be used. If the inspection is a special, scour monitoring, service, or interim inspection, it should be clearly explained in the report.

1.2.4 Liabilities

In the event of negligence in carrying out the basic responsibilities described above, individuals, including department heads, engineers, and inspectors, are 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 deck, superstructure, substructure, or culvert elements.
- Failed joints.
- Hazards to the traveling public.
- Improper or deficient bridge 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 or a vague report may have negative consequences for the highway agency involved in lawsuits involving bridges. The report is often scrutinized to determine if conditions are documented thoroughly and for the “proper” reasons. Therefore, inspectors should be as objective and complete as possible. If something could not be inspected, this should be stated along with the reason it was not inspected.

Example of Liabilities

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 ft to the ground. Ten years before the accident, the consulting firm had noted severe deterioration of the deck and had recommended tests to determine the need for replacement. Two years before 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 percent of the ensuing settlement.

Section 1.3 Quality Control and Quality Assurance

1.3.1 Introduction

Title 23, *Code of Federal Regulations (CFR)*, Part 650, Subpart C, Section 313 (p), 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 and QA program to assist bridge owners in developing their QC and QA programs. Refer to Chapter 18 for detailed information regarding the framework recommendations.

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 necessary 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, but it also affects public safety.

QC and QA programs are 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. Personnel completing QC and QA reviews are to be different than the personnel that completed the original work. The feedback is then used to enhance the inspection program through improved inspection processes and procedures, training, and quality of the inspection report.

1.3.2 Quality Control

Quality Control is the establishment and enforcement of procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level. The quality of the inspection and the ensuing reports rests primarily with the inspection team members and the team leaders. Their knowledge and professionalism should result in the development of a quality product. Experienced and qualified personnel, potentially at the senior level within the section, shall review the report and inspection data for accuracy and completeness before finalizing the inspection.

Organizational structure can vary based on the owning entity performing or administering bridge inspections. If an inspection program is decentralized, the state program manager is still ultimately responsible for QC, but the manner in which the program is carried out may differ according to each organization's policy. Results of the QC review should be documented, including tracking and completion of actions identified (23 CFR 650.313(p)).

1.3.3 Quality Assurance

Quality Assurance is the use of sampling and other measures to assure the adequacy of QC 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, QA inspections are to be performed by another inspection team or an independent team comprised of qualified personnel.- Results of the QA review should be documented, including tracking and completion of actions identified (23 CFR 650.313(p)).

PART I – BRIDGE SAFETY INSPECTION PROGRAMS

CHAPTER 2 TABLE OF CONTENTS

CHAPTER 2 LIST OF FIGURES

This page intentionally left blank.

Chapter 2 Inspection Fundamentals

Section 2.1 Planning and Performing the Inspection

2.1.1 Introduction

Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

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 nine basic types of inspection:

- Initial.
- Routine .
- Underwater.
- Nonredundant Steel Tension Member (NSTM).
- Damage.
- In-Depth.
- Special .
- Scour Monitoring.
- Service.

These inspection types are presented in the NBIS and the *Specifications for the National Bridge Inventory (SNBI)*, with additional information in Article 4.2 of the AASHTO *Manual for Bridge Evaluation (MBE)*. Although this manual is organized for "routine" inspections, it applies to any inspection type. The amount of time and effort necessary for performing each duty varies with the type of inspection performed.

2.1.2 Types of Bridge Inspection

Various inspection types may be employed over the life of a bridge to reflect the intensity of inspection necessary at the time of inspection. The nine types of inspections identified in the Federal Regulations are described below and enable a Bridge Owner to establish appropriate inspection levels consistent with the inspection interval and the type of structure and details.

Initial

An Initial inspection is the first inspection of a new, replaced, or rehabilitated bridge. This inspection serves to record required bridge inventory data, establish baseline conditions, and establish the intervals for other inspection types (23 CFR 650.305). This inspection type is typically accompanied by both a load rating and an initial bridge scour evaluation as necessary. Other inspection types such as Underwater and NSTM, if necessary, are typically completed concurrently to thoroughly establish baseline, but can be completed soon after if the need is identified during the Initial inspection.

Routine

Routine inspections are regularly scheduled comprehensive inspections consisting of observations and measurements needed to determine the physical and functional condition of the bridge. Routine inspections identify any changes from the "Initial" or previously recorded conditions and ensure that the structure continues to satisfy present service conditions (23 CFR 650.305). These inspections are typically performed from the deck and ground or water level, or from permanent inspection structures. Special access equipment, including rigging, bridge inspection cranes, and drones, are necessary for inspections in circumstances where their use provide the only practical means of accessing and/or determining the condition of the bridge (23 CFR 650.313 (a)).

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. Inspection of underwater portions of the substructure is generally limited to observations during low-flow periods and/or probing for signs of scour and undermining. If an area of the structure demands additional observation, such as a member adversely affecting the load-carrying capacity or if undermining is found, an In-Depth or Underwater inspection may be scheduled.

The NBIS establish regular, reduced, and extended Routine inspection intervals under Methods 1 and 2, which must be followed (23 CFR 650.311 (a)). In many cases States or other owners have additional requirements.

Damage

A Damage inspection is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions (23 CFR 650.305). A visual inspection is performed for all members that were affected in the event. The scope of the inspection is extensive enough to determine the need for emergency load restrictions or closure of the bridge to traffic. Clear, thorough information should be gathered so that the damage can be assessed to the level necessary to develop an effective repair. Nondestructive evaluation (NDE) methods may be necessary to fully assess damage and long-term needs.

In-Depth

An In-Depth inspection is a close-up, detailed inspection of one or more bridge members located above or below the water, using visual or nondestructive evaluation techniques as required to identify any deficiencies not readily detectable using Routine inspection procedures. Hands-on inspection may be necessary at some locations (23 CFR 650.305). This type of inspection can be scheduled independently of a Routine inspection, though may occur more or less frequently than Routine inspections, as outlined in bridge specific inspection procedures (23 CFR 650.305).

Nonredundant Steel Tension Member

A Nonredundant Steel Tension Member (NSTM) inspection is a hands-on inspection of all NSTMs on the bridge. NSTMs are primary steel members fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse (23 CFR 650.305). The NSTM inspection uses visual methods that may be supplemented by nondestructive testing. A detailed visual hands-on inspection is the primary method of detecting cracks and critical areas should be cleaned so that the inspection is effective at detecting cracks. Additional lighting and magnification could be necessary. Other nondestructive methods may be necessary to fully ascertain presence and extent of cracks. Where the fracture toughness of the steel is not documented, tests may be necessary to determine the threat of brittle fracture at low temperatures.

According to the NBIS, NSTMs are to be inspected at regular intervals not to exceed the interval established using one of two risk-based methods. The first method determines inspection intervals by a simplified assessment of risk to classify bridges into one of three risk categories. The categories include regular, reduced, and extended intervals, with intervals not to exceed 24, 12, and 48 months respectively (23 CFR 650.311(c)(1)). A second method may result in the same inspection intervals, but is determined by a more rigorous assessment of risk per 23 CFR 650.311(c)(2).

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 or by wading or probing, and generally requiring diving or other appropriate techniques (23 CFR 650.305). Underwater inspections are an integral part of a total bridge inspection plan, to determine channel conditions including the severity and extent of scour or undermining. Any plans of action for scour critical bridges that include additional inspection needs should be followed.

Underwater inspection intervals may be adjusted based on structure type, design, materials, age, condition ratings, scour, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle/vessel impact damage, loads and safe load capacity, and other known deficiencies. Regular, reduced, and extended intervals are outlined in Methods 1 and 2 in 23 CFR 650.311 (b).

Special

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, or to monitor special details or unusual characteristics of a bridge that does not necessarily have defects (23 CFR 650.305). Examples may include foundation settlement or scour, fatigue damage, or the public's use of a load posted bridge. These inspections are not as comprehensive as a Routine inspection. They are often scheduled between Routine inspections as an inspection of the focus area demanding additional monitoring.

Scour Monitoring

Per the *SNBI*, a Scour Monitoring inspection is an inspection performed during or after a triggering storm event as required by a Scour Plan of Action (POA), by personnel with qualifications required by the agency. Scour Monitoring inspections are performed on an as-needed basis, with no regular interval. Documentation completed during the Scour Monitoring inspection should be retained in the bridge file as part of the flood history for the bridge.

Service

A Service inspection is an inspection used to identify major deficiencies and safety issues, performed by personnel with general knowledge of bridge maintenance or bridge inspection (23 CFR 650.305). This is an inspection performed midway between Routine inspections, with extended intervals approved greater than 48 months.

2.1.3 Duties of the Bridge Inspection Team

This section presents the duties of the bridge inspection team. It also describes how the inspection team can prepare for the inspection and some of the major inspection procedures. For some duties, the inspection program manager may be involved.

Note that the Bridge Inspection Team referenced in this manual includes the Program Manager, Team Leader, and Inspector. There are several basic duties of the bridge inspection team:

- Planning the inspection.
- Preparing for the inspection.
- Performing the inspection.
- Communicating the need for immediate follow-up for critical findings.
- Preparing the report.
- Identifying items for repairs and maintenance.
- Recommending load ratings, if necessary.
- Establishing the next inspection intervals (per 23 CFR 650.311 (f)).

Agencies must review the inspection interval criteria after each inspection to ensure the proper interval is assigned to the bridge. They must establish the next inspection interval for each inspection type based on results of the inspection and requirements of 23 CFR 650.311.

2.1.4 Planning the Inspection

Planning is necessary for a safe, efficient, cost-effective inspection effort that results in a thorough and complete inspection of in-service bridges. Basic activities may 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, NSTM, Underwater, and Special inspections (as outlined in 650.313 (h)) (23 CFR 650.313(j)). Although not required by the NBIS, it is a good practice to provide a team leader for Damage, Scour Monitoring, and Service inspections. Though complex feature inspections are not considered a separate inspection type, a team leader must be on-site during all inspection activities, including mechanical and electrical.
- Evaluation of necessary activities to ascertain needs for access equipment; nondestructive evaluation; traffic control, including the use of flaggers; utilities; confined spaces; permits; railroad coordination; and hazardous materials such as pigeon droppings, lead paint and asbestos removal, etc.
- Establishing a schedule that includes the inspection duration.
- Establishing an efficient routing between inspection sites.

2.1.5 Preparing for Inspection

Preparation measures necessary before the inspection include organizing the proper personnel, 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 can include:

- Reviewing the bridge file.
- Identifying the components and elements.
- Developing an inspection sequence for each bridge.
- Preparing and organizing notes, forms, and sketches.
- Arranging for temporary traffic control.
- Arranging staging areas and access locations.
- Reviewing safety precautions.
- Organizing tools and equipment.
- Arranging for subcontracting special activities.
- Accounting for other special considerations.

Review the Bridge File

An initial 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.
- Accident records.
- Load rating records.
- Load posting.
- Permit loads.
- Flood, scour, and channel cross-section data.
- Traffic data.
- Inspection history.
- Inspection procedures.
- NBI data summary sheets.

Each of these sections of the bridge file is presented in detail in Chapter 18.

Identify Components and Elements

Bridges are comprised of four major components, namely the deck, superstructure, substructure, and culvert, if applicable. It is imperative for an inspector to correctly categorize members of each component. More information regarding bridge components can be found in Chapter 3. In addition to the deck, superstructure, substructure, and culvert, the FHWA *SNBI* includes joints, bearings, and roadside hardware which includes bridge railing and guardrail transitions as separate components. Joints and roadside hardware are discussed further in Chapter 8. Bearings are discussed further in Chapter 13.

Another important activity in preparing for the inspection is to establish, or identify the previously established, structure orientation, as well as a system for identifying the various components and elements of the bridge (see Figure 2.1.1).

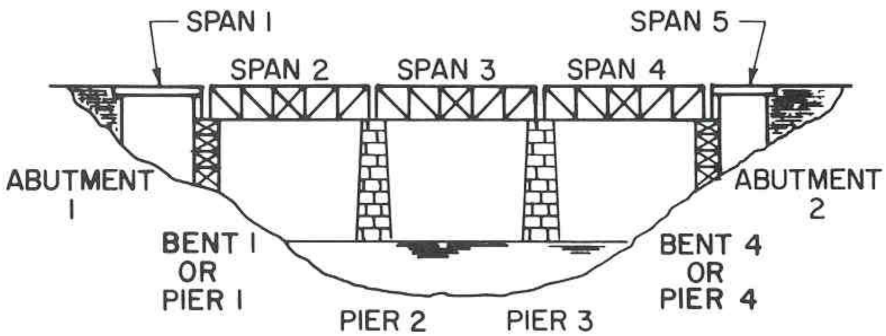


Figure 2.1.1 Sample Bridge Numbering Sequence

The numbering system presented in this section is one possible system, but agencies may establish their own systems.

Positioning 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, or agency standards (e.g., south to north, west to east). Latitude and longitude coordinates are identified for the bridge, as well as a Linear Reference System (LRS) mile point, if available.

Regardless of the identification system used for the bridge, the following information should be included for the deck, superstructure, and substructure.

Deck

Items commonly associated with the deck are the deck sections, joints, bridge railing and transitions, sidewalks, and lighting. Inspectors should identify these members consecutively, from the beginning to the end of the bridge.

Superstructure

Items commonly associated with the superstructure include the spans, beams, and the panel points, in the case of a truss, arch, or frame. The spans should be numbered consecutively, with Span 1 at the beginning of the bridge. Multiple beams are typically numbered consecutively from left to right facing in the route direction. Similar to spans, floorbeams should also be numbered consecutively from the beginning of the bridge, with the first floorbeam labeled the same as the first panel point. This coordinates the floorbeam and the bay numbers such that a given floorbeam number is situated at the end of its corresponding bay.

For trusses, the panels should be numbered similarly to the floorbeams. Both the upstream and downstream trusses should be labeled. Points in the same vertical line should have the same number. If there is no lower panel point in a particular vertical line, the numbers of the lower chord should 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 (e.g., U9′). However, this numbering system is typically not recommended for field inspection since it is more difficult to follow in the field.

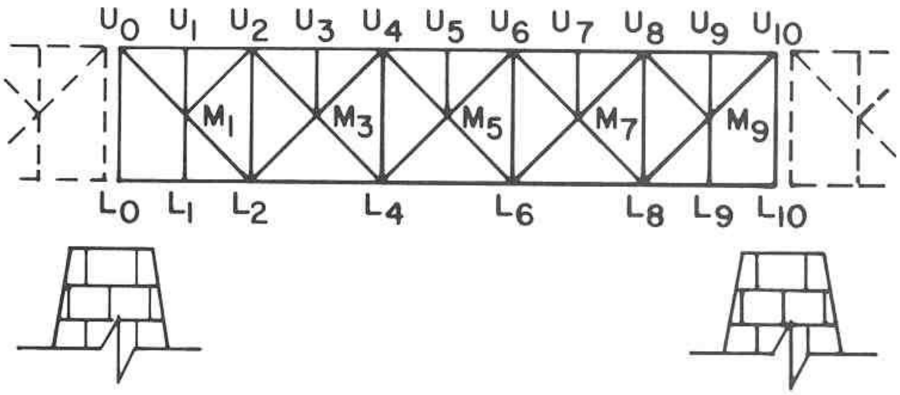


Figure 2.1.2 Sample Truss Numbering Scheme

Substructure

Items commonly associated with the substructure are the abutments and the piers or bents. Abutment 1 should be at the beginning of the bridge, and Abutment 2 should be at the end. Number the piers consecutively, with Pier 1 closest to the beginning of the bridge. Alternatively, the substructure units may be numbered consecutively without noting abutments or piers.

Develop Inspection Sequence

An inspection sequence typically follows the load path and begins with the deck and superstructure elements and proceeds to the substructure. However, there are many factors that should 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.
- Accessibility.
- Traffic conditions.
- Weather conditions.
- Special considerations.

Developing an inspection sequence is important to ensure a safe, complete, and thorough inspection of the bridge. A sample inspection sequence for a common bridge type is presented in the following steps.

1. Identify any general global issues. These include, but are not limited to, the following:
   - Overall bridge alignment: vertical and horizontal.
   - Deflections.
   - Bridge settlement.
   - Channel alignment.
   - Necessary construction or maintenance work.

2. Inspect the approach roadway. Items to review include:
   - Bridge approaches.
   - Roadside hardware.
   - Approach settlement.

3. Inspect the deck and adjacent areas, including:
   - Bridge deck: top and bottom.
   - Bridge joints.
   - Sidewalks.
   - Barriers, railings, other traffic control devices.
   - Drainage.
   - Signing.
   - Lighting/electrical.

4. Inspect the superstructure. This includes items such as:
   1. Primary load-carrying members.
   2. Secondary members, other bracing.
   3. Utilities and attachments.
   4. Anchorages.

5. Inspect the bearings.

6. Inspect the substructure, including any of the following that may apply.
   - Abutments.
   - Piers, or bents.
   - Footings.
   - Piles.
   - Curtain walls.
   - Slope protection.

7. Inspect applicable channel and waterway items, such as:
   - Scour.
   - Channel profile.
   - Streambed.
   - Embankment and protection.
   - Hydraulic opening and control devices.
   - Channel cross sections.
   - Substructure protection systems.
   - Navigational lights and aids.

Prepare and Organize Notes, Forms, and Sketches

Preparing notes, forms, and sketches before the on-site inspection reduces work in the field. Copies of the agency's standard inspection form, electronic or hard copy, should be obtained for use in recordkeeping and as a checklist to ensure that the condition of all elements is noted. It is also important to ensure all electronic inspection data collection software and equipment is in working condition before the inspection.

Any desired paper copies of sketches should be created from previous inspection reports so that deficiencies previously documented can simply be updated. Preparing extra copies provides a contingency for sheets that may be lost or damaged in the field. If sketches are electronic and would be updated via a tablet or laptop in the field, the inspector should do as much preparation as possible before the field inspection. Templates for new sketches including anticipated measurements and existing deterioration should be created.

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 streamlined 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. Sketches and drawings are particularly important for NSTMs. Nonredundant steel tension members, as well as any fatigue sensitive details, should be clearly identified in the NSTM inspection procedures.

Arrange for Temporary Traffic Control

Bridge inspection, like construction and maintenance activities on bridges, often presents motorists with unexpected and unusual situations (see Figure 2.1.3).

Most State agencies have adopted the Federal *Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD)*. Some State and local jurisdictions, however, issue their own manuals. When working in an area exposed to traffic, the governing standards should be checked and followed. Additional advanced coordination with State or City traffic operations centers may be necessary for higher traffic volume roadways. Standards generally prescribe the minimum methods for several applications and the proper use of standard traffic control devices, such as channelizers, signs, flashing arrow boards, and truck mounted attenuators (TMAs) in high speed areas. A police detail, flaggers, or a combination, may be necessary as well. Adjustments may be necessary based on actual conditions after the inspection starts.

Figure 2.1.3 Temporary Traffic Control Operation

Principles and methods that 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.
- Traffic should be routed through work areas with geometrics and traffic control devices comparable to those employed for other highway situations.
- Traffic and pedestrian movement should be inhibited as little as practicable.
- Approaching motorists should be guided in a clear and positive manner throughout the bridge inspection site.
- On long duration inspections, inspectors should perform Routine inspection of temporary traffic control devices.
- Personnel responsible for the performance of temporary traffic control operations should be adequately trained.

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 necessitate conducting the inspection operation with less than optimum efficiency. Though 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 traffic volume (i.e., at night or during the weekend).

Special Considerations

Time Needs

The total time necessary 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 separately (office preparation, travel, on-site, report preparation) benefits future planning and preparation efforts. 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 needing traffic restrictions may be limited to certain hours of the day, such as 10:00 AM to 2:00 PM. Some days, such as holidays, may be restricted for inspection work entirely.

Set-up Time

Set-up time, both before and during the inspection, should be considered. For example, rigging efforts may necessitate several days before the inspectors arrive on the site. Also, other equipment, such as compressors and cleaning equipment may necessitate daily set-up time. Adequate time should be provided in the schedule for set-up and take-down time needs. Also, the time to install and remove temporary traffic control devices should be considered.

Access

When preparing for an inspection, inspectors should consider access needs and necessary equipment such as boats, ladders, and under bridge inspection vehicles (UBIV). Bridge members may be very similar to each other, but may demand different amounts of time to gain access. 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 be opened to gain access to a portion of the bridge. Access hatches may be locked, so coordination with the bridge owner or local maintenance crews may be necessary. When preparing to inspect movable bridges, the inspector should coordinate with the appropriate personnel for opening and closing to assess proper mechanical operation.

Weather

Adverse weather conditions may not halt an inspection entirely but may play a significant role in the inspection process. During inclement weather, inspectors should avoid climbing on the bridge structure, using equipment that may be struck by lightning, and performing other bridge inspection activities that may affect traffic. An increased awareness of safety hazards is necessary and keeping notes dry can be difficult. It should also be noted that some electronic equipment may not work properly in cold weather. During seasons of poor weather, a less aggressive schedule should be adopted than during the good weather months.

Safety Precautions

Even though the inspection should be completed in a timely and efficient manner, the importance of taking safety precautions cannot be overlooked. General safety recommendations for inspection and any agency or bridge specific safety precautions and Personal Protective Equipment (PPE), such as for hazardous material and confined space entry, should be reviewed. Confined space entry methods are to be conducted in accordance with the more restrictive of OSHA, employer, or

Bridge Owner requirements. For additional information about bridge inspection safety precautions, refer to Section 2.2.

Permits

There are several instances for which a permit should be obtained to gain access for a bridge inspection. When inspecting a bridge crossing a railroad, obtain an access permit before proceeding with the field inspection. Coordination with the railroad to obtain flagging personnel will be necessary in addition to obtaining a permit. Also obtain a permit when inspecting bridges passing over navigable waterways. Environmental permits and permits to work around endangered species may be needed for some bridges and bridge sites. Some bridge owners also require a permit or other advance notice to close traffic lanes. Coordination for any of these permits can take up to six months or more to procure; therefore, contact with the owning railroad, environmental agency, Coast Guard, and/or the Bridge Owner should be made well in advance of the scheduled inspection.

Tools

To perform a complete and accurate inspection, the proper tools and equipment should be used. Bridge location and type are two main factors in determining necessary tools and equipment. Refer to Section 2.4 for a complete list of inspection tools and equipment.

Subcontracted Special Activities

Inspectors should consider additional time demands and coordination necessary when special activities are scheduled. Activities that may involve a subcontractor include the following:

- Maintenance and protection of traffic (M.P.T.).
- Special access, including rigging, inspection vehicle(s), or a combination of the two.
- Coordination with various railroads, including obtaining flagging services.
- Nondestructive evaluation/testing.

2.1.6 Performing the Inspection

This section refers to the general activities necessary for on-site completion of the bridge and waterway inspections, if present. Activities necessary for other inspection types should be specifically identified if discussed. Detailed information for inspecting specific materials and members, and for recording deficiencies and other observations, is contained in subsequent chapters.

Perform inspections in accordance with the NBIS and *MBE*.

Basic activities include:

- Visual examination of all bridge components and elements.
- Physical examination of bridge components and elements with observed defects.
- Evaluation of bridge components and elements.
- 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, as well as following proper inspection procedures. Inspection team members, air temperature, weather conditions, and time, should also be recorded.

The procedures used to inspect a bridge depend largely on the bridge type, the materials used, and the general condition of the bridge. Therefore, inspectors should be familiar with the basic inspection procedures for a wide variety of bridges.

Forethought used to prepare for the inspection should be combined with thorough and complete recordkeeping. Careful and attentive observations should be made for each deficiency recorded. A very careful inspection is worth no more than the records kept during that inspection.

For an Initial inspection, when design or as-built plans are available, the inspector should compare the dimensions of the bridge with those presented on the plans. For subsequent inspections, dimensions should be verified only as necessary to ensure accuracy if an item in the inspection report appears incorrect.

On larger bridges, placing numbers or letters on the bridge by using builder's crayon (also called keel) or paint to identify and code components and elements of the structure should be considered. It may be appropriate to use markings on smaller bridges as well, especially if they can be used to identify the bridge itself or specific members of the bridge in the future. These marks help the inspector keep track of their location and guard against overlooking any portion of the structure. Some bridge owners, however, do not let marking placement to occur directly on the structure, so ensure that it is permitted before the inspection.

The general approach roadway alignment should be noted, and the inspector should sight along the railing and edge of the deck or girder to detect any misalignment, deflection, or settlement. When inspecting the underside, any possible causes for such issues should be noted.

Approaches and Decks

The approach pavement should be checked for unevenness, settlement, or roughness. The inspector should also check the condition of the shoulders, slopes, and drainage. Adjacent roadside hardware, signs (load restrictions), and lighting should be verified that they are present, and their conditions should be noted.

The deck and any sidewalks should be examined for various deficiencies, noting size, type, extent, and location of each deficiency. The inspector should reference the location using the centerline or curb line, the span number, and the distance from a specific pier or joint.

The expansion joints should be examined for sufficient clearance and for adequate seal. Any roadway debris restricting movement should be noted. The inspector should 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. Any vertical misalignment between the near and far side of the joint should also be recorded.

Superstructures

The superstructure should be inspected 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 typically visual, which may necessitate some cleaning to view all surfaces properly such as the removal of rust scale, dirt, leaves, animal waste, and debris. The most common forms of primary load-carrying members are:

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

Bearings

If design or as-built plans are available, the dimensions of the bearings should be compared with those presented on the plans. The bearings should be inspected thoroughly since they provide the critical link between the superstructure and the substructure. The primary method of bearing inspection is a visual inspection, which necessitates removing rust scale, dirt, leaves, animal waste, and debris to enable close observation and evaluation of the bearings. The inspector should record the difference between the bearing movement and a fixed reference line, noting the direction of movement, the air or bearing material temperature, and the general weather conditions at the time of the inspection. Any debris or pack rust restricting movement should be noted.

Substructures

The substructure, which supports the superstructure, consists of abutments, piers, and bents. If design or as-built plans are available, the dimensions of the substructure units should be compared with those presented on the plans. Since the primary method of bridge inspection is visual, inspectors should remove the dirt, leaves, animal waste, and debris to enable close observation and evaluation. The substructure units should be checked for settlement by sighting along the superstructure and noting any tilting or cracking of vertical faces. In conjunction with the scour inspection of the waterway, the inspectors should check the substructure units for undermining, noting both its extent and location.

Culverts

Culverts should be inspected regularly to identify any potential safety problems and maintenance needs. The culvert should be examined for various deficiencies, noting size, type, extent, and location of each deficiency. Inspectors should reference the location using the centerline and from the ends of the culvert. In addition to the inspection of the culvert and its components, inspectors should look for high-water marks, bulging or distortion in flexible culverts, scour, and settlement of the roadway.

Waterways

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

A historical record of the channel alignment, profile, and cross-sections should be maintained. Inspectors should record and compare current measures to initial (base line) measures, noting any meandering of the channel both upstream and downstream. Any substructure units (location or skew) that may adversely affect normal or flood flows under the bridge should be reported.

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. Inspectors should determine the existence and extent of scour at the substructure units. Any evidence of undermining should be noted and probed if possible. The extent of scour and undermining should be compared with the tolerable amount as determined by analysis.

The embankment erosion both upstream and downstream of the bridge, as well as any debris and excessive vegetation should be noted. Inspectors should record their type, size, extent, and location. Inspectors should also note the high-water mark, referencing it to a fixed elevation such as the bottom of the superstructure.

Inspection of Bridge Members

There are several general terms used to describe bridge deficiencies (see Appendix C of the *SNBI*):

- Corrosion - section loss.
- Cracking - breakage without complete separation.
- Connection - loose or missing fasteners, broken welds, pack rust with or without distortion.
- Distortion - misshapen from its original state.
- Settlement - vertical displacement of the structure into supporting soil; exists within or exceeds tolerable limits.
- Scour - removal of streambed material; existing within or exceeding tolerable limits.
- Efflorescence - leeching of material from cement paste to the surface of concrete.
- Mortar breakdown - partial to full-depth voids in masonry mortar.
- Patched areas - sound or unsound patches in concrete or masonry.

- Displacement - shifting slightly or significantly out of alignment.
- Delamination, spalling - separation and removal of a portion of the material (generally concrete).
- Exposed rebar – usually in the presence of a spall; present with or without section loss.
- Exposed prestressing – usually in the presence of a spall; present with or without section loss.
- Splitting - masonry or timber separating into parts (see also checks, shakes).
- Decay - section loss of a timber member.
- Abrasion, wear - wearing away of material from friction.
- Slippage - relative movement of connected parts.
- Overstress - deformation due to overload.
- Collision damage - damage caused when a bridge is struck by vehicles or vessels.
- Deterioration - defect for “other materials”; breakdown or section loss of material.

Though collision damage or overload may be the cause of deficiencies, speculation should be avoided and the inspector should document all issues. Refer to Chapter 7 for a more detailed list and description of types and causes of deterioration for specific materials. As described in Chapter 7, each material is subject to unique deficiencies; therefore, it is important to be familiar with the different inspection methods used with each material.

For the inspection to be substantiated, inspectors should document and record all inspection findings, which are generally referred to as “condition remarks”, on the inspection form or in the inspection report.

Concrete

When inspecting concrete structures, all visible cracks should be noted, recording their type, width, length, and location. Establishing the severity of cracking when inspecting prestressed concrete should demand more scrutiny as compared to concrete structures with mild reinforcing. The inspector should record any rust or efflorescence stains. Concrete scaling can occur on any exposed face of the concrete surface, so its area, location, depth, and general characteristics should be recorded. Concrete surfaces should be inspected for delamination or hollow zones, which are areas of incipient spalling, using tools such as a hammer or a chain drag. Any delamination should be carefully documented using sketches showing the location and pertinent dimensions.

Unlike delamination, spalling is readily visible. Any spalled areas should be documented using sketches or photos, noting the depth of the spalling, the presence of exposed reinforcing steel, and any deterioration or section loss that may be present on the exposed reinforcement.

Refer to Chapter 9 for more detailed information on the characteristics and inspection of concrete superstructures.

Metal: Steel, Iron, and Others

When inspecting metal structures, the inspector should determine the extent and severity of corrosion, carefully measuring the amount of cross section remaining. All cracks should be noted, recording their length, size, and location. Distorted or damaged members should be documented, noting the type of damage and amount of distortion.

Loose rivets or bolts can be detected by striking them with a hammer and holding a thumb on the opposite end of the rivet or bolt at the same time. Movement can be felt if it is loose. In addition, the inspector should note any missing rivets or bolts.

Any locked or "frozen" pins, hangers, or expansion devices should be noted. 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 tend to lean towards the fixed end of the bridge. In hot weather, they tend to lean away from the fixed end. A locked bearing is generally caused by heavy rust on the bearing assembly.

The inspector should also know the steel type prior to inspection in order to properly assess for specific defects. For instance, AASHTO M244 Grade 100 (commonly known as T-1) steel is known to be susceptible to cracking and has poor weldability (FHWA Memo 2021).

Refer to Chapter 10 for more detailed information on the characteristics and inspection of steel superstructures.

Timber

When inspecting timber structures, the inspector should determine the extent and severity of decay, weathering, and wear, being specific about dimensions, depths, and locations. The timber can be sounded and probed to detect hidden deterioration due to decay, insects, or marine borers.

Any large cracks, splits, or crushed areas should be noted. Any fire damage should be noted, recording the measurements of the remaining sound material. Inspectors should document any exposed untreated portions of the wood, indicating the type, size, and location.

Refer to Chapter 11 for more detailed information on the characteristics and inspection of timber superstructures.

Masonry

The examination of stone masonry and mortar is similar to that of concrete. Inspectors should carefully inspect the joints for cracks and other forms of mortar deterioration. Masonry arches or masonry-faced concrete arches should be checked for bulging, mortar cracks, vegetation, water seepage through cracks, loose, or missing stones or blocks, weathering, and spalled or split blocks and stones.

Refer to Chapter 11 for more detailed information on the characteristics and inspection of masonry structures.

Fiber Reinforced Polymer

When inspecting Fiber Reinforced Polymers (FRP), any blistering, voids, delamination, discoloration, wrinkling, fiber exposure and any scratches should be noted. Inspectors should document visible cracks, recording their width, length, and location.

Refer to Chapter 11 for more detailed information on the characteristics and inspection of FRP structures.

Underwater, NSTM, In-Depth, and Complex Feature Inspection Procedures

Some bridges are complex or otherwise demand specialized inspection procedures. For the inspection procedures listed below, refer to Chapter 17 for advanced inspection methods for concrete, steel, timber, and fiber reinforced polymers. In accordance with the NBIS, NSTM, Underwater, In-Depth, and complex feature inspections require documented inspection procedures (CFR 650.313 (g)).

Inspection procedures are developed to identify:

- Equipment needs.
- Personal needs and qualifications.
- Access requirements.
- Scheduling considerations.
- Coordination with agencies and/or partners.
- Risk factors.
- Those portions of the bridge to be inspected.
- The inspection methods and techniques to be utilized.
- Inspection interval.
- Documentation requirements.
- Reporting and follow-up processes.
-

Underwater

The Underwater inspection is important for recognizing any problems that cannot be identified in a Routine inspection. Underwater inspections necessitate trained underwater bridge inspection divers. The divers check the substructure for undermining, deterioration, and accumulated debris, as well as any other issues.

Refer to Chapter 16 for Underwater inspection methods.

Nonredundant Steel Tension Members (NSTM)

Bridge members that are identified to be nonredundant steel tension members or have fatigue-prone details should be closely inspected and documented for any fatigue cracks, out-of-plane bending, or other distortion. Areas of emphasis can include connections, steel members with section loss, fatigue-prone details, previous crack-arrest holes, and retrofits. These inspections should be hands-on, within arm's reach, so that even small cracks can be identified.

Refer to Chapter 7 for inspection methods for nonredundant steel tension members.

In-Depth

An In-Depth inspection is defined in the Code of Federal Regulations as a close-up detailed inspection of one or more bridge members located above or below water, using visual or nondestructive evaluation techniques as required to identify any deficiencies not readily detectable using routine inspection procedures. Hands-on inspection may be necessary at some locations. In-depth inspections may occur more or less frequently than routine inspections, as outlined in bridge specific inspection procedures (23 CFR 650.305). Agencies must document the criteria to determine the level and interval for this inspection type in its bridge inspection policies and procedures (23 CFR 650.311 (d)). Agencies must also identify the location of bridge members that need an In-Depth inspection and document in the bridge files. A Team Leader must be present during each In-Depth inspection.

Complex Features

Complex features are bridge components or members with advanced or unique structural members or operational characteristics, construction methods, and/or requiring specific inspection procedures (23 CFR 650.305). Bridges such as cable-supported bridges, movable bridges, and floating bridges have complex features that demand additional planning and on-site inspection. Complex features include mechanical, electrical, or hydraulic portions that may necessitate additional qualified personnel to assist in inspection.

Refer to Chapter 5 and Chapter 12 for more detailed information on the characteristics and inspection of complex features.

2.1.7 Identifying Items for Preservation and Routine Maintenance

Another common duty is to identify work recommendations for bridge preservation or repair. Inspectors should recommend work items that promote public safety and maximize useful bridge life. 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 may include deck or bridge washing, flushing the scuppers and down spouts, lubricating the bearings, and painting the structure. Inspectors should carefully consider the benefits to be derived from completing the work recommendation and the consequences if the work is not completed.

When summarizing inspection data and making recommendations, it is necessary to create categories that prioritize the repairs that should be performed. Also, inspectors should check the previous report recommendations to see what work was recommended and the priority of such items. The recommendations can be included in an inspection report format and/or entered into the bridge database for scheduling repair or rehabilitation and for historical purposes. Not all bridge owners intend for their inspectors to provide specific recommendations, instead simply report findings to other qualified personnel. If work was scheduled to be completed before the next inspection, it should be noted if the work was completed and the necessity for any follow-up work.

The following repair classifications are suggested:

- Critical repair.
- Priority repair.
- Routine repair.

A critical repair usually results from a critical finding (see Sec 2.1.8), which refers to damage, deterioration, or a defect that requires immediate action to ensure public safety. This includes the possible closure of the structure or the affected area, for safety reasons until interim remedial measures can be implemented.

A priority repair refers to the conditions for which further investigation, design, and execution of interim or long-term repairs should be undertaken on a priority basis, including precedence over other scheduled work. These repairs should improve the durability and functionality of the structure or member, reducing future maintenance costs. Deterioration or damage that impacts the ability of the member to carry its design load may also be classified as a priority repair. It is suggested to schedule repair for items within this type of repair classification within two years.

A routine repair refers to the conditions requiring further investigation or remedial work that can be undertaken as part of a scheduled maintenance program, other scheduled project, or routine machine facility maintenance, depending on the action necessary. Items that are identified in the preventive maintenance program should be incorporated within this category. If there is not an upcoming project or scheduled maintenance, monitor the condition to ensure that the repair should not be completed with greater urgency.

Bridge owners are free to develop different repair classifications if they wish.

2.1.8 Critical Findings

Critical findings are any structural or safety-related deficiency that requires immediate action to ensure public safety (23 CFR 650.305). When a critical finding is discovered, the inspector should immediately communicate and document the critical finding according to agency procedures.

The NBIS regulation requires agencies to establish an agency wide procedure to assure that critical findings are addressed in a timely manner (23 CFR 650.313). Additionally, the NBIS requires that FHWA be notified of all critical findings and actions taken, underway, or planned to resolve critical findings. The duty of the inspection team is to follow agency-wide procedures for the follow-up on critical findings.

It is the responsibility of Bridge Owners to implement procedures for addressing critical findings, including:

- Immediate critical finding 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.
- Verification of a completed action and documentation (close-out) within the bridge management system.
- Provisions for identifying other bridges with similar structural details for follow-up, or Special, inspections.

Refer to Chapter 3 for a detailed description of critical findings and the methods necessary to address any critical findings discovered.

Section 2.2 Safety Fundamentals for Bridge Inspectors

2.2.1 Importance of Bridge Inspection Safety

Though 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 demands 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 should be concerned about safety.

Key motivations for bridge inspection safety may include but are not limited to:

- Injury and pain.
- Family hardship.
- Equipment damage.
- Lost production.
- Medical expenses.

2.2.2 Safety Responsibilities

The employer is responsible for providing a safe work environment, including, but not limited to:

- Clear safety regulations and procedures.
- Safety training.
- Proper tools and equipment, including PPE.

The team leader is responsible for maintaining a safe working environment, including, but not limited to:

- Supervision of established job procedures.
- Ensuring proper application of safety procedures.
- Ensuring proper use of equipment.
- Enforcement of safety regulations.

Bridge inspectors are ultimately responsible for their own safety. The bridge inspector's responsibilities include, but are not limited to:

- Recognition of physical limitations - Inspectors should recognize their limitations and communicate them to supervisors and inspection team members.
- Knowledge of rules and duties of job - Inspectors should verify that they understand a particular task and that they are qualified to perform that task. If a procedure appears to be unsafe, it should be questioned and a safer procedure should be developed constructively.
- Safety of fellow workers - Inspectors should not act in a manner that endangers fellow inspectors. Co-workers should be warned 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. Any injury should be promptly reported in order to assure coverage, if necessary, under workmen's compensation or other insurance.

2.2.3 Personal Protective Equipment

It is important to dress properly for the job. Inspectors should wear field clothes that are properly sized and appropriate for the climate. For general inspection activities, boots with traction lug soles should be worn. For climbing sections of a bridge, boots with a steel shank (with non-slip soles without heavy lugs), as well as gloves should be worn. Wearing a tool pouch enables the inspector to carry tools and notes with hands free for climbing and other inspection activities.

Safety equipment, commonly referred to as Personal Protective Equipment (PPE), is designed to prevent injury. Equipment should be correctly used in order for it to provide protection. Agency or OSHA regulations should be consulted for approved types and appropriate usage (OSHA 1926.954). The following are some common pieces of PPE.

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. The main concerns are:

- Deteriorated portions of bridge members 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 members. When inspections involve climbing or access equipment, the inspector is frequently dodging various bridge members. These bridge members can be sharp edged and are unyielding. If the inspector makes a mistake in judgement during a maneuver and impacts the structure, a hard hat may prevent serious injury or a potential fall. In addition, if inspection is to occur extensively around power lines, hard hats rated for electrical hazards should be worn for protection.

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

Figure 2.2.1 Inspectors Wearing Hard Hats

Reflective Safety Apparel

When performing activities near traffic, the inspector is required by OSHA or agency policies to wear a safety vest, shirt, jacket, and/or pants (OSHA 1926.95). The inspector must verify that their apparel conforms to current OSHA and MUTCD standards and State requirements (see Figure 2.2.2). 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.

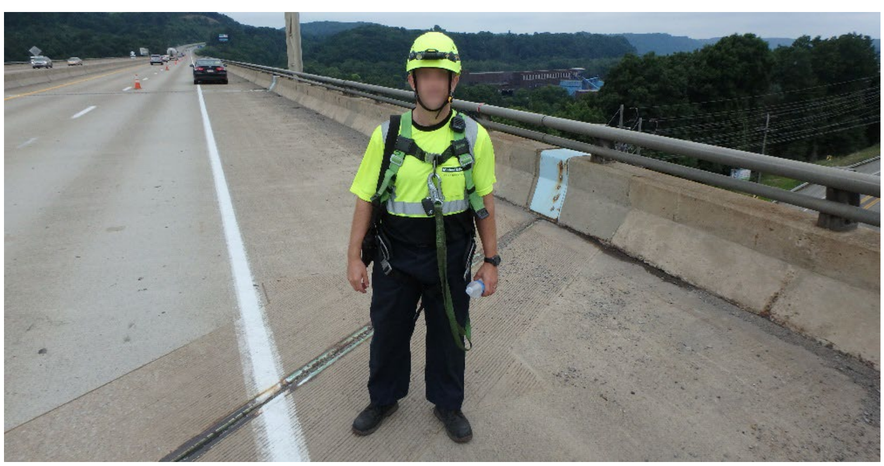


Figure 2.2.2 Inspector Wearing Reflective Safety Shirt

Protective Eyewear

Eye protection is necessary when the inspector is exposed to flying particles (see Figure 2.2.3). 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).

Eye protection should be worn during activities such as:

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

Gloves

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 may be 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

Proper Footwear

It is important for inspectors to wear appropriate footwear when inspecting a bridge. Generally, work boots are required by OSHA or agency policies, and boots with a safety toe are preferred for maximum protection (OSHA 1926.95). If inspecting around electrical hazards, proper footwear with rubber insulation should be worn. If the inspection is anticipated to be in shallow water, the inspector should use waders. Waders can be knee-height, hip waders, or chest waders, which can be used for deeper water.

After a rainy day, the inspector should ensure their boots are free of mud, and should use extreme caution in areas where debris accumulation may cause a slippery surface.

Personal Flotation Device

Inspectors should wear a personal flotation device or life jacket when working over water or in a boat (see Figure 2.2.4). If an accident occurs, even good swimmers may drown if burdened with inspection equipment. Also, if knocked unconscious or injured due to a fall, a floatation device can keep the inspector afloat. Inspectors should also consider wearing 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 weigh them down causing a precarious situation.

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).

Figure 2.2.5 Inspector Wearing a Respirator and Safety Glasses

Conditions necessitating a respirator include:

- Sand blasting.
- Painting.
- Exposure to dust from pigeon droppings (exposure to pigeon droppings may result in histoplasmosis, a potentially serious illness).
- Work in closed or constricted areas.
- Hammering, scraping or wire brushing steel members with lead-based paints.

It is a good practice to also wear safety glasses/goggles if wearing a respirator. Flying airborne containments can be harmful to the eyes.

Health Monitors

When confined areas are inspected, safety procedures issued by OSHA and any additional agency-specific requirements need to be followed (OSHA 1910.120(b)(4)(ii)(E)). Pre-entry air tests include testing for oxygen with an approved oxygen testing device and testing for other gases, such as carbon monoxide, hydrogen sulfide, methane, natural gas, and combustible vapors (see Figure 2.2.6).

Figure 2.2.6   Multi-gas Meter

Gas testing demands special observation. Some gases float to the ceiling (e.g., methane, carbon monoxide) and some sink into depressions (e.g., carbon dioxide). Hydrogen sulfide gas dissolves in water, with the potential for large volumes to be released when it is disturbed.

Safety Harness and Lanyard

The safety harness and lanyard are the inspector's lifeline in the event of a fall (see Figure 2.2.7). Verify that equipment satisfies agency and OSHA requirements (OSHA 1926.502).

Figure 2.2.7   Inspector with Safety Harness with a Lanyard

To reduce the possibility of injury, the maximum lanyard length limits a fall to a maximum of 6 ft per OSHA regulations (OSHA 1926.104(d)). Further protection can be achieved using a shock absorber between the lanyard and the safety harness. The shock absorber typically reduces g-forces through the controlled extension of nylon webbing, which is pre-folded and sewn in union. 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. The second lanyard should be used to enable safe movement around obstacles connecting the second lanyard before disconnecting the first lanyard in order to safely move along the structure.

Suspension trauma straps are attached to the harness and provided in case of a fall. These straps are intended to relieve the pressure on the inspector's legs when hanging in a harness. They can be deployed manually, creating a sling to stand or sit on instead of relying on the leg straps which cut off circulation and can cause a life-threatening situation.

Inspectors should not tie off to scaffolding or a supporting cable. One of the reasons for tying off is to limit fall distance in case the rigging or scaffold fails. When working from a UBIV or bucket truck, inspectors should tie off to the structure if possible. The structure is generally the most stable point of support. Extreme caution should be exercised not to enable the equipment to be moved out from under someone tied to the bridge. If the access equipment is being moved frequently, it is best to tie off to the inspection bucket.

2.2.4 Causes of Accidents

General Causes

Accidents are usually caused by human error or equipment failure. Part of safety awareness is 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 or training. 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 may include the following:

- Loss of focus - distraction, carelessness, inattentive when performing routine tasks.
- Personal limitations - lack of knowledge or skill, exceeding physical capabilities.
- Physical impairment - previous injury, illness, side effect of medication, alcohol, or drugs.
- Lack of awareness - distraction from safety 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 into several categories: General Precautions, Climbing Safety, Confined Spaces, Vegetation, Wildlife, Night Work, Working Around Water, and Working Around Traffic.

General Precautions

Mental Attitude

- Emotional distress should be avoided - Dangerous tasks should not be performed when emotionally upset. In activities such as climbing, emotional distress may increase chances of falling.
- Awareness of surroundings - Inspectors should be aware of dangers associated with inspection location. If not fully alert, inspectors should not be inspecting.
- Limitations should be realized - An inspector should be confident that the job can be performed safely. If there is a feature that cannot safely be inspected with the equipment available, it should not be inspected.

General Recommendations

Some general recommendations for safe inspections are as follows:

- Keeping well rested and alert - Working conditions encountered during an inspection are varied and can change rapidly demanding the inspector to be fit and attentive.
- Maintaining proper mental and physical condition - Inspection tasks necessitate a multitude of motor skills. To perform at tolerable levels, the inspector is to be physically fit and free from mental distractions.
- Using proper tools - Inspectors should not 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 - Inspectors should establish methods early in the job and utilize them so everyone knows what to expect of one another.
- Safety rules and regulations should be followed - Inspectors should adhere to the safety rules and regulations established by the OSHA, the agency, and their employers.
- Common sense and good judgment should be used - Inspectors should not engage in horseplay, and should not take short cuts or foolish chances.
- Intoxicants or drugs should not be used - Intoxicants and drugs may impair judgment, reflexes, and coordination.
- Medication - Potential side effects should be discussed with a physician or pharmacist.
- Electricity - This is a potential killer. Inspectors should assume cables and wires to be energized even if they appear to be only telephone cables. The conditions encountered on many bridges are conducive to electric shock. These conditions can include steel members, humidity, perspiration, and damp clothing. Transmission lines should be identified on a structure before the inspection. Inspectors should maintain the required stand-off distance from any energized line or consider having power lines de-energized. In rural areas, electric fences should be avoided since they can be a hazard. Inspectors should have

awareness that fiberglass posts eliminate the need for the distinctive porcelain insulation, which once identified electric fences.

- Inspection over traffic - It is typically best to avoid working above traffic. If it cannot be avoided, inspectors should tie off equipment, such as hand tools and clip boards.
- Entering dark areas - Inspectors should use a flashlight or headlamp to illuminate dark areas before entering as a precaution against falls, snakebites, cornered animals, and stinging insects.
- Vagrant people - Caution should be exercised when approaching a bridge where homeless people are present. Inspectors should try to explain to them that an inspection of the bridge is taking place. Inspectors should leave the bridge site immediately if there are any illegal activities or perceived danger.

Working in Teams

It is highly recommended to work in groups of two or more. Any action should not be taken without someone else there to help in case of an accident. Someone else should know where you are. If someone seems to be missing, that person should be found immediately.

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

As a general precaution, an inspector should realize any physical limitations, as climbing is very strenuous, and an inspector should be in good shape to perform (see Figure 2.2.8).

Proper preparation is key for a safe climbing inspection. There are two primary areas of preparation necessary: organization and inspection equipment.

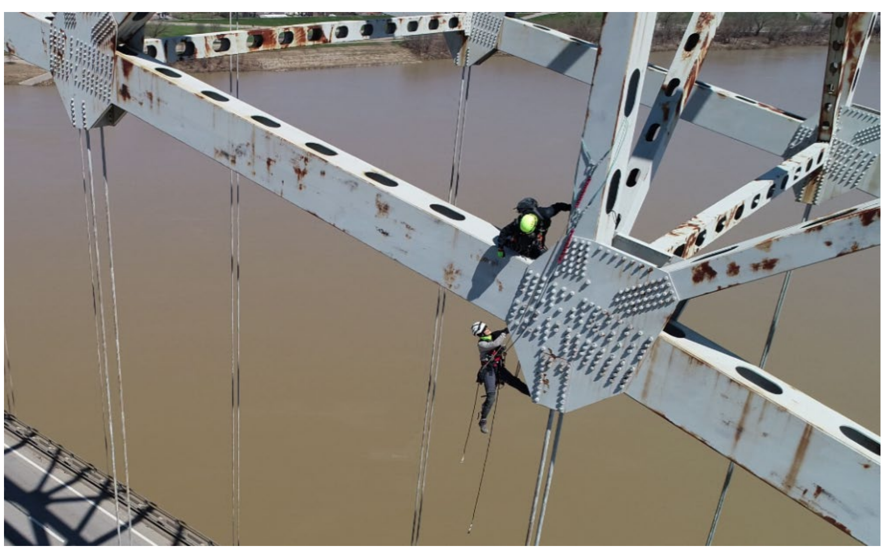


Figure 2.2.8 Inspection Involving Extensive Climbing

Organization

A good inspection procedure should incorporate a climbing strategy that minimizes climbing time. For example, beginning the day with an inspection of a truss span from one bent and finishing at the next bent by lunch time eliminates unproductive climbing across the span.

The inspection procedure should have an inspection plan, so the team knows where to go, what to do, and what tools are necessary 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.

Inspection Equipment

The inspection team should 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. Tools and equipment should be adequately tethered to the inspector when working at heights. Even lightweight tools when dropped from a significant height can cause injury or death to inspection team members, motorists, or pedestrians below.

The use of ladders often presents fall hazards. Inspectors should refer to and follow OSHA for rules applicable to stairways and ladders (OSHA 1926.502). In order to use a ladder properly, the following should be considered:

- Only a stable and level surface should be used, otherwise the ladder should be secured to prevent displacement.
- Proper ladder length for the job.
- 4 to 1 (vertical to horizontal) tilt with blocked and secured bottom.
- An assistant to anchor or hold the ladder to assure stability when climbing for ladders over 25 ft and making sure the top is tied off.
- Inspecting the ladder, before 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 within the rails. Inspectors should always maintain a 3-point (two hands and a foot, or two ft and a hand) contact on the ladder when climbing. Using a hand line to lift equipment or tools.

Access equipment and vehicles should always be used in accordance with the manufacturer's recommendations and safety warnings. Inspection equipment, access equipment, and access vehicles are explained in detail in Section 2.5.

Scaffolding may be the chosen method of access to some bridges. Inspectors should refer to and follow the owner's safety requirements for rules applicable to scaffolding. Inspectors may check scaffolding for the height and load capacity necessary to support the inspection team and their equipment. Scaffolding load tests can be performed on the ground with planned equipment and personnel. A daily inspection for cracks, loose connections, and buckled or weak areas of the scaffolding should be performed before use.

Single planks should never be used as a work platform on scaffolding. Two or more planks should be used and securely cleated in union. Inspectors should securely attach plank ends to their supports. Planks should be inspected for knots, splits, cracks, and deterioration before use.

Use of platform trucks, bucket trucks, and UBIVs may be necessary to access members during an inspection. Inspectors should confirm that the vehicles are in safe operating condition. Such equipment should only be used when placed on a firm flat surface or at a slope not exceeding the manufacturer's recommendations. Extreme caution should be exercised when operating near traffic.

Permanent inspection access devices, such as inspection catwalks, are useful to gain access to many portions of the bridge (see Figure 2.2.9). Inspectors should be on guard for misalignment and deterioration of members, such as flooring, hand-hold rods, and cables. Although these are a permanent part of the bridge and generally have handrails, it is still recommended to tie off to the structure when possible.

Figure 2.2.9 Inspection Catwalk

Inspectors should be familiar with proper rigging techniques. The support cables should be at least one-half inch in diameter. The working platform or "stage" should be at least 20 inches wide. A line or tie-off cable separate from the primary rigging should be used.

Inspectors should use common sense with regard to rigging and should not blindly trust the people arranging the rigging. Mistakes by riggers can cause life threatening accidents. If a method is unsafe or doubtful, it should be questioned and changed if necessary. Inspectors should not rely on ropes or planks left on the bridge by previously completed work. They may be rotted or not properly attached.

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. There is additional concern if one end of a structure or member may be blocked. Confined space entry for private sector workplaces is regulated by OSHA and requires proper training, equipment, and permitting (OSHA 1910.146). However, there are no state or federal safety rules for confined space entry for public sector workplaces.

There are four major concerns when inspecting a confined space:

- Lack of oxygen - an oxygen content above 19 percent is necessary 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, inspectors should use the safety standards prescribed by OSHA and any additional agency or employer requirements (OSHA 1910.146). 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 before entry.
- During occupancy - Regardless of activity, continuously ventilate the space with outside air. 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 necessary to remain outside of dark or confined areas with means to communicate with inspectors.

Vegetation

Inspectors should be aware of vegetation around substructures. Poison ivy, oak, and sumac are examples of vegetation that can cause skin irritations if they come in contact with skin. Thorn bushes or vines can scratch exposed skin, tear clothing or even perforate rubber boots. 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. Vegetation growing up the side of a substructure unit should be removed to enable visual inspection.

Wildlife

Dangerous wildlife can be present in the area around or under a bridge. Poisonous snakes are just one example of wildlife that can pose a threat. Inspectors should document if any wildlife is encountered for planning of future inspections.

Night Work

When working at night, it is important to be properly dressed, so the inspectors are visible to passing motorists. This can be accomplished by wearing a safety vest or shirt that has both bright colors and reflectivity. The use of proper temporary traffic control also helps notify motorists that there are workers ahead. Drowsiness can be a problem, especially when switching to the night shift after typically working daylight hours.

Working Around Water

Wading

When wading in water, it is important to be aware of any scour holes and be careful not to 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 weigh them down, creating a precarious situation. It is also important to wear a life vest when wading to help prevent the inspector from being pulled down if the waders were to fill up with water. It may be beneficial for the inspector to carry and use a probing rod to find scour holes and soft stream bed material. Inspectors should 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 they plan to walk.

Storms may generate high flows in culverts very quickly. This creates a dangerous situation for the inspectors. It is not uncommon for culverts to carry peak flow long before a storm reaches the culvert site. Inspectors should be cautious whenever storms appear imminent.

Quicksand Conditions at the Outlet

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

Underwater

When performing an Underwater inspection, particularly in low visibility and/or high current situations, inspectors should use extreme care and be sure to watch for drift and debris at any height in the water. Refer to Chapter 16 for additional safety concerns.

Boats/Skiff

When an inspection takes place over water and a drowning hazard exists that cannot be mitigated by other means such as fall protection, a safe work environment necessitates additional precautions. For their personal protection, the inspector wears a personal flotation device. The operation also includes rescue capability, such as a boat/skiff with a designated rescue person. Inspectors should be sure the boat is equipped with a life ring and radio communication with the inspection crew. In the event that an inspector falls into the water, the boat or designated rescue person can assist them quickly. This is especially important if the individual has been rendered unconscious.

Working Around Traffic

Even with the proper traffic control in place, inspectors should still exercise caution and be aware of the dangers of working adjacent to traffic. It is a good practice, when possible, to face traffic when inspecting the bridge deck. Inspectors should always exercise care if crossing live traffic is necessary. Be aware that many motorists do not concentrate on driving, even in the presence of inspectors at a bridge. Many drivers are talking on their phones, texting, or looking at the internet instead of watching out for bridge inspectors. Traffic should not be obstructed during bad weather. Inspectors should avoid the inspection of the top of concrete decks during or just after it rains due to poor visibility and decreased stopping distances.

Section 2.3 Temporary Traffic Control

2.3.1 Introduction

Bridge inspection usually only demands temporary traffic control procedures for a relatively short-term closure. Bridge inspection, like construction and maintenance activities on bridges, often presents motorists with unexpected and unusual situations. Most State agencies have adopted the *MUTCD*. Some States and local jurisdictions, however, issue their own standard manuals or drawings. Inspectors should coordinate with the appropriate agency.

When working in an area exposed to traffic, inspectors should check and follow the existing agency standards. These standards prescribe the minimum procedures for typical applications and the proper use of standard temporary traffic control devices such as channelizers, signs, flashing arrow-boards and TMAs (see Figure 2.3.1). Sometimes after initial installation, temporary traffic control may be revised to provide adequate protection to motorists, pedestrians, or inspectors. It is good practice for the team leader to drive through the traffic control set-up to ensure adherence to standards. This is especially true if the traffic control is to be left in place for an extended period of time.

Figure 2.3.1 Temporary Traffic Control Operation

2.3.2 Philosophy and Fundamental Principles

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

Inspection time should be minimized to reduce exposure to potential hazards without compromising the thoroughness of the inspection. Principles and procedures that 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 part of every inspection project, from the planning stage to performance of the inspection. Inspectors should keep in mind the safety of the motorist, pedestrian, and inspection team member.

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. Clear communication should be made to inform the driver 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 typically 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

Traffic movement should be inhibited as little as practical. Temporary traffic control in work sites should be designed on the assumption that motorists only reduce their speeds if they clearly perceive a need to do so. Reducing the speed limit should be avoided 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. Frequent and abrupt changes in geometrics, such as lane narrowing, dropped lanes, or main roadway transitions that can cause rapid maneuvers should be avoided.

Provisions for the safe operation of work vehicles should be made, 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.

Provide a Clearly Marked Path

A good traffic control plan provides safe and efficient movement of motorists, pedestrians, and other system users and the protection of bridge inspectors in work zones. Adequate warning, delineation, and channelization should provide positive guidance in advance of and through the work area. Use proper signing and other devices that are effective under varying conditions of light and weather.

2.3.3 Inspector Safety Practices

Work Zone

Moving traffic may represent an 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. Inspectors should keep in mind that some agencies require advance notice and approval before a traffic lane can be closed. This advanced notice gives the agency the opportunity to notify and educate the traveling public.

The work zone should be clearly marked 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 should be as compact as possible, but wide enough and long enough to permit access to the area to be inspected and provide for safe movement of workers and equipment. The end of the work zone should be clearly signed as a courtesy to the motorist.

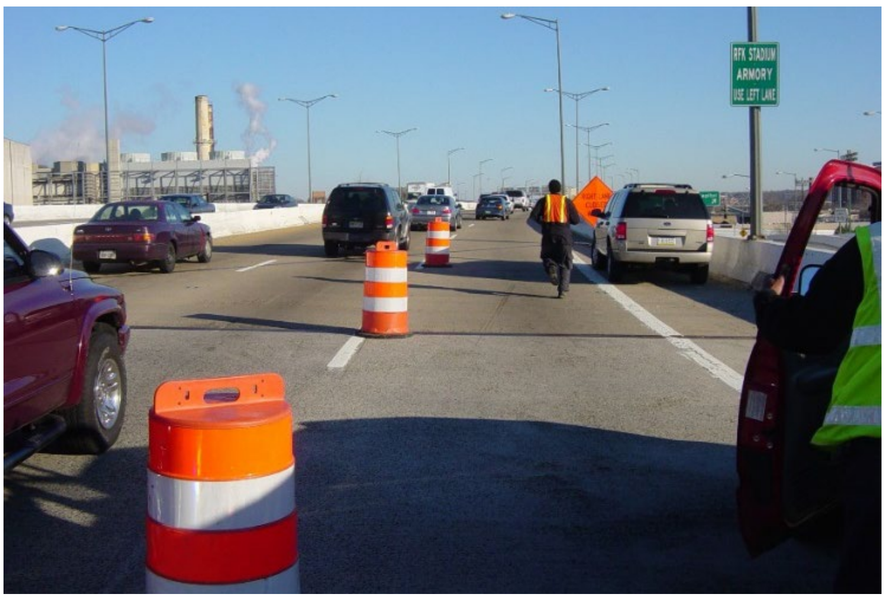


Figure 2.3.2 Work Zone

Vehicles and Equipment

Inspection vehicles and equipment should be made visible to the motorists and work within traffic control limits as appropriate (see Figure 2.3.3).

Roof mounted flashing lights or flashers should be used on vehicles entering and exiting the work zone to distinguish them from other motorists' vehicles. Also, extreme caution should be used when moving inspection vehicles in and out of the work zone. Motorists should be provided ample time to react to the inspection vehicle's movements.

Figure 2.3.3 Inspection Vehicle with Traffic Control

Workers

Approved safety apparel and hard hats should be worn for visibility and identification. Inspectors wearing proper safety attire present a professional appearance to the public but most importantly, PPE enables them to safely perform their activities. Also, it is important for the inspector to stay within the work zone for their own safety.

2.3.4 Temporary Traffic Control Assistance

Reference the State traffic control requirements for typical signage, channelizing devices, lighting devices, and flagging requirements. The following areas are considerations specific to bridge inspection.

Flaggers

Flagger control is usually used for bridge inspection, where the one-lane section is short enough so that each end is visible from the other end. Traffic may be controlled by means of a flagger at each end of the section. One of the two should be designated as the chief flagger to coordinate movement. They can communicate with each other verbally using personal communication devices 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. A flagger's sole duty is to maintain the flow of traffic and not engage in any other aspects of the bridge inspection when flagging.

Where traffic in both directions use a single lane for a limited distance, provisions should be made 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 should 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 the ends of the structure. Control points should be chosen at each end of the route to permit easy passing of opposing lines of vehicles.

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

- Flag transfer.
- Pilot car.
- Automated flagger assistance devices.
- Temporary traffic signals.
- Stop or yield control.

Truck Mounted Attenuators

Shadow vehicles with truck mounted attenuators (TMAs) assist in the prevention of vehicles entering the work zone if the motorist drifts into the lane closure. These are commonly utilized during bridge inspection. Each agency has its own specific requirements, but a shadow vehicle with TMA is generally employed any time a shoulder or travel lane is occupied by workers or equipment. Shadow vehicles have an independent operator and 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 2.3.4). This protects the errant motorist, as well as the inspectors in the work zone.

Figure 2.3.4 Shadow Vehicle with Truck Mounted Attenuator

Police Assistance

On some inspection sites, 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 bridge 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 volume traffic roads. Their specialized training provides for a safer work environment for the bridge inspectors.

Section 2.4 Inspection Equipment

2.4.1 Introduction

Proper inspection equipment plays a key role in a proper inspection and 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 designed for the job. 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 should be familiar with every piece of equipment and how to use and operate it properly and safely. Note that inspection equipment used during the inspection is to be recorded under *SNBI* Item B.IE.12 "Inspection Equipment".

2.4.2 Necessary Equipment

Several factors play a role in what type of equipment is necessary for an inspection. Bridge location and type are two of the main factors in determining equipment needs. If the bridge is over water, certain pieces of equipment such as life jackets and boats are necessary. Also, if the bridge is made of timber, then specific pieces of equipment like timber boring tools and picks are necessary, 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 necessary equipment. Bridge Inspection vehicles are typically fully stocked with various types of equipment. Inspection teams typically verify that the tools or equipment necessary for the day are in the vehicle before leaving the office.

2.4.3 Standard Tools

Standard tools that an inspector uses at the bridge site can be grouped into seven basic categories:

- Cleaning (see Figure 2.4.1).
- Inspection (see Figure 2.4.2).
- Visual aid (see Figure 2.4.3).
- Measuring (see Figure 2.4.4).
- Documentation.
- Access.
- Miscellaneous equipment.

Figure 2.4.1 Tools for Cleaning

Figure 2.4.2 Tools for Inspection

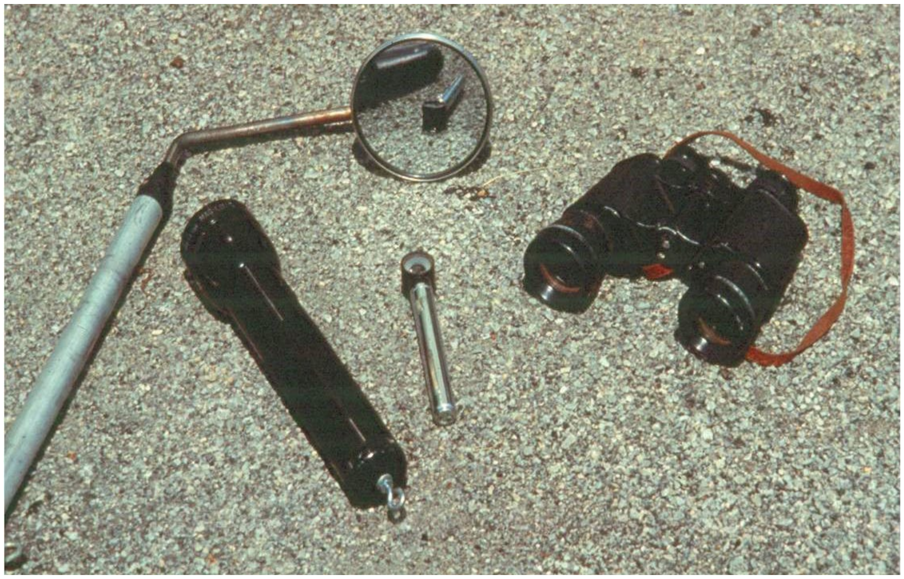


Figure 2.4.3 Tools for Visual Aid

Figure 2.4.4 Tools for Measuring

Cleaning

Common tools used for cleaning include, but are not limited to:

- 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.

Inspection

- Common tools used for inspection include, but are not limited to: Pocket knife - used for general duty.
- Ice pick - used for surface examination of timber members.
- Cordless drill (or hand brace and bits) - used for boring suspect areas of timber members.
- Timber boring tools - used for internal examination of timber members.
- Chipping hammer (16-ounce geologist's pick) - used for loosening dirt and rust scale, sounding concrete, removing rust, and checking for sheared or loose fasteners.
- Rotary percussion tool - used for sounding concrete in areas difficult to access by chipping hammer.
- 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.

Visual Aid

Common tools used as visual aids include, but are not limited to:

- Binoculars - used to preview areas before 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).

Measuring

Common tools used for measuring include, but are not limited to:

- Pocket tape (six-foot rule) - used to measure deficiencies and member and joint dimensions.
- 25 foot and 100-foot tape - used for measuring member dimensions.
- Scour pole - used to measure water depths during wading.
- Calipers - used for measuring the thickness of a member beyond an exposed edge.
- Crack comparator card - used to measure crack widths.
- 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.

Documentation

Common tools used for documentation include, but are not limited to:

- Inspection forms, clipboard, and pencil - used for record keeping.
- Computers or Tablets - used for record keeping.
- Notebooks - used for additional record keeping for complex structures.
- Straight edge - used for drawing readable sketches.
- Digital camera - used to document inspection findings and provide digital images of deficiencies that can be downloaded and e-mailed for instant assessment.
- Chalk, keel, 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).
- Masonry survey nails, sometimes referred to as Parker-Kalon nails (P-K), used for establishing a reference point necessary for movement documentation of substructures and large cracks.

Most bridge inspectors use pencil and paper to record inspection findings. 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 bridge management software and the inspection is updated. Some agencies are using Electronic Data Collection and appropriate computer applications to create or mark-up existing bridge inspection information.

Hardware

Data recording hardware can include desktop computers, notebook computers, tablets (see Figure 2.4.5), or smartphones. Some versions of these devices have been made to be more rugged and even “wearable” for use in the field.

Figure 2.4.5 Tablet Used to Collect Inspection Data

Applications

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

Access

Common tools used for access include, but are not limited to:

- 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 Section 2.5.

Miscellaneous Equipment

- "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.
- Coveralls - used to protect clothing and skin against sharp edges during inspection.
- Life jacket - used for safety over water.
- Cell phone - used to call in emergencies.
- Toilet paper - used for other "emergencies".

2.4.4 Special Equipment

For the Routine inspection of a common bridge, special equipment is usually not necessary. However, with some structures, special inspection activities necessitate special tools.

Survey Equipment

Special circumstances may demand the use of GPS or a transit, a level, an incremental rod, or other survey equipment. This equipment can be used to establish an exact location of an area of interest relative to the rest of the structure, as well as an established reference point.

Nondestructive Evaluation Equipment

Nondestructive evaluation (NDE) is the in-place examination of a material for structural integrity without damaging the material. NDE equipment enables the inspector to "see" inside a bridge member and assess deficiencies that may not be on the surface or 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 methods and equipment, refer to Chapter 17.

Underwater Inspection Equipment

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, or a folding rule.

When the waterway is deep, an Underwater inspection is performed by trained divers. This necessitates special diving equipment that may include a working platform, underwater camera(s), depth-finder, fathometer, air supply systems, radio communication, sonar, and sounding equipment. Refer to Chapter 16 for a more detailed description of Underwater inspection equipment.

Other Special Equipment

An inspection may demand special equipment to prepare the bridge before 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.

Section 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 can include ladders, rigging, and scaffolds. Common access vehicles may include lifts, bucket trucks, and UBIVs. In most cases, using a lift or bucket truck should be less time consuming than using a ladder or rigging to inspect a structure. The time saved, however, is typically offset by the higher costs associated with operating access vehicles and possible lane closures.

Occasionally the need for specialized access equipment is not discovered until the bridge is being inspected. In this case, highlight this fact in the notes so that appropriate equipment can be scheduled if necessary. Inspectors should not hide the fact that a particular bridge member was not inspected.

Safety

Before the bridge inspection begins, an equipment inspection should be performed. As a minimum, access equipment should be inspected as per the manufacturer's recommendations. Using faulty equipment can lead to serious accidents and even death. Inspectors should check the equipment and verify that it is in good working condition with no defects or problems. If rigging or scaffolding is being used, it should be checked to ensure that it was installed properly and that the cables and planks are secured tightly. OSHA-approved safety harnesses with shock absorbing lanyards should be used when using access equipment (OSHA 1926.502).

If the inspector is not familiar with the inspection vehicle being used, then the time necessary should be taken to become accustomed to its operation. In some cases, formal operator training may be necessary or required by the owner or manufacturer. When operating any inspection vehicle, all parties involved should 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. The inspector should always be alert to their position. The boom should not be extended out into unsafe areas such as unprotected traffic lanes or near electrical lines. OSHA-approved safety harnesses with shock absorbing lanyards should be used when using access vehicles (OSHA 1926.502).

2.5.2 Types of Access Equipment and Access Vehicles

Access equipment and vehicles position the inspector close enough to the bridge member so that a "hands-on" inspection can be performed. The following are some of the most common forms of access equipment and access vehicles used in bridge inspection.

Ladders

Ladders are generally 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. When set up at the proper angle (1 horizontal to 4 vertical), the inspector is able to reach out horizontally to grasp the rung and keep their feet at the base of the ladder (see Figure 2.5.1).

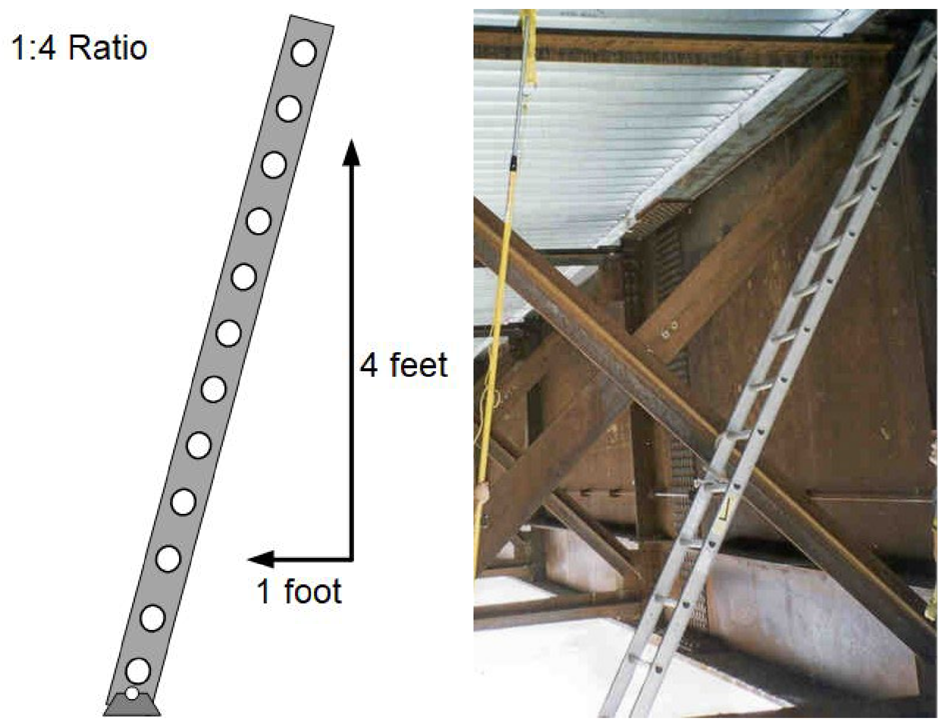


Figure 2.5.1 Ladder with the Proper 1H to 4V Ratio

Ladders may also be 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). When using a hook-ladder, the inspector is tied off to a separate safety line, independent of the ladder.

Figure 2.5.2 Inspector Using a Hook-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 necessary (e.g., NDE of pins). Rigging is often used when ladders or other access equipment cannot reach a given location (see Figure 2.5.3 and Figure 2.5.4). Rigging can be a good choice for a load-posted bridge that does not have the capacity to support an inspection vehicle.

Figure 2.5.3 Rigging for Substructure Inspection

Figure 2.5.4 Rigging for Superstructure Inspection

Rigging does not typically 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.

Scaffolds

Scaffolds can provide an efficient access alternative for structures that are less than 40 ft high and over level ground with little or no traffic nearby (see Figure 2.5.5). OSHA has specific requirements when working on scaffolding (OSHA 1926.452). Scaffolds may take longer to set up than it takes to inspect the bridge; therefore, scaffolding is not typically used for bridge inspection.

Figure 2.5.5 Scaffold

Lift

A lift is a vehicle with a platform or a bucket capable of holding one or more inspectors. The platform is attached to a hydraulic boom that is mounted on a carriage (see Figure 2.5.6). An inspector "drives" the carriage using controls in the platform. This type of vehicle is usually not licensed for use on highways. However, some lifts are nimble and can operate on a variety of terrains. Although four-wheel drive models are available, lifts are limited to use on fairly level terrain. Lifts come in various sizes with vertical reaches ranging from 40 ft to over 170 ft.

Figure 2.5.6 Lift

A track-mounted lift provides access to areas with rough terrain that a conventional bucket truck would not be able to navigate (see Figure 2.5.7 and Figure 2.5.8). By utilizing rubber tracks, track-mounted lifts can be operated in water, climb 35-degree slopes, traverse 25-degree side slopes, and navigate wet and muddy terrain.

Figure 2.5.7 Track-mounted Lift in a Stream

Figure 2.5.8 Track-mounted Lift on a Slope

Scissor Lift

Scissor lifts may be used for bridge inspections with low clearance between the bridge and roadway underneath (see Figure 2.5.9). Scissor lifts have a typical maximum vertical reach of 20 ft. These lifts are designed for use on relatively level ground.

Figure 2.5.9 Scissor Lift

Bucket Truck

A bucket truck is similar to a lift. However, a bucket truck can be driven on a highway, and the inspector controls bucket movement (see Figure 2.5.10). As with the lift, a bucket truck should be used on fairly level terrain. Bucket trucks have several features and variations:

- Lift capability - varies 25 to 50 ft.
- 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 ft.
- 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 necessitate outriggers for stable operations cannot be moved during the inspection unless the outriggers have wheels.

Figure 2.5.10 Bucket Truck

Under Bridge Inspection Vehicle

An under bridge inspection vehicle (UBIV) is a specialized bucket or platform truck with an articulated boom designed to reach under the superstructure when parked on the bridge deck (see Figure 2.5.11 and Figure 2.5.12). Usually, the third boom has the capacity for extending and retracting, providing for greater reach under a structure. Some of the larger vehicles have four booms, providing an even greater reach. UBIVs currently have the capability to extend up to 75 ft horizontally beneath the bridge.

Figure 2.5.11 Under Bridge Inspection Vehicle with Bucket

Figure 2.5.12 Under Bridge Inspection Vehicle with Platform

Boats or Barges

A boat or barge may be necessary to gain access to structures over or adjacent to water. A boat can be used for inspection (see Figure 2.5.13), as well as providing access to areas for taking photographs or provide a staging area for divers. A safety boat is also necessary when performing an inspection over water.

Figure 2.5.13 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 lift or mobilization for Underwater inspections.

Bucket Boats

A bucket boat is another option to perform inspections over water, as an alternative to a UBIV. Bucket boats can provide access from the water as opposed to needing traffic control on the deck of the structure. They are typically self-propelled and self-leveling and unlike a barge, a bucket boat can be equipped with typical bridge inspection equipment.

Crawler

Crawlers are mobile inspection platforms or cages that “climb” steel cables or truss members (see Figure 2.5.14). They are typically well suited for the inspection of high piers and other long vertical faces of bridge members.

Figure 2.5.14 Crawler

Floats

A float is a wood plank work platform hung by ropes (see Figure 2.5.15). Floats are generally used for access in situations where the inspector may be at one location for a relatively long period of time.

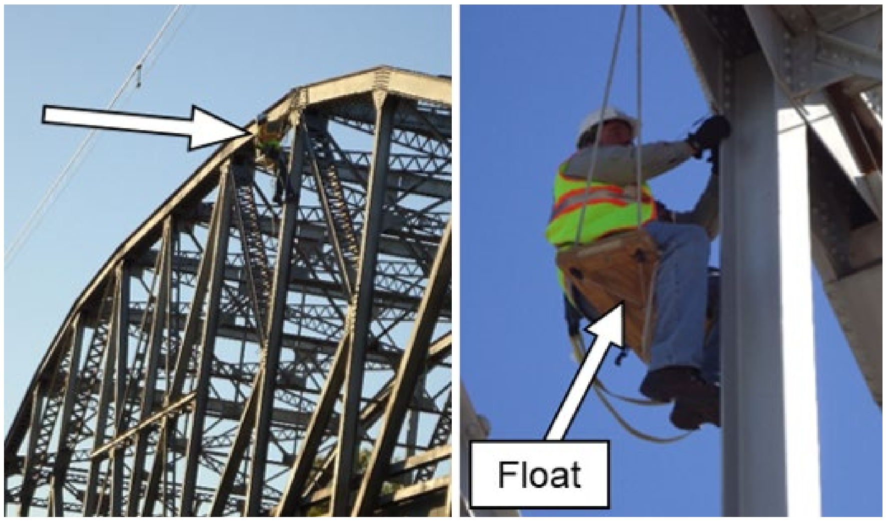


Figure 2.5.15 Inspector Using Float

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 quickly with block and tackle devices to provide access to many areas in a short amount of time. Rappelling is a similar access method to the Bosun chair but utilizes different equipment and techniques (see Figure 2.5.16). Both methods demand the use of independent safety lines.

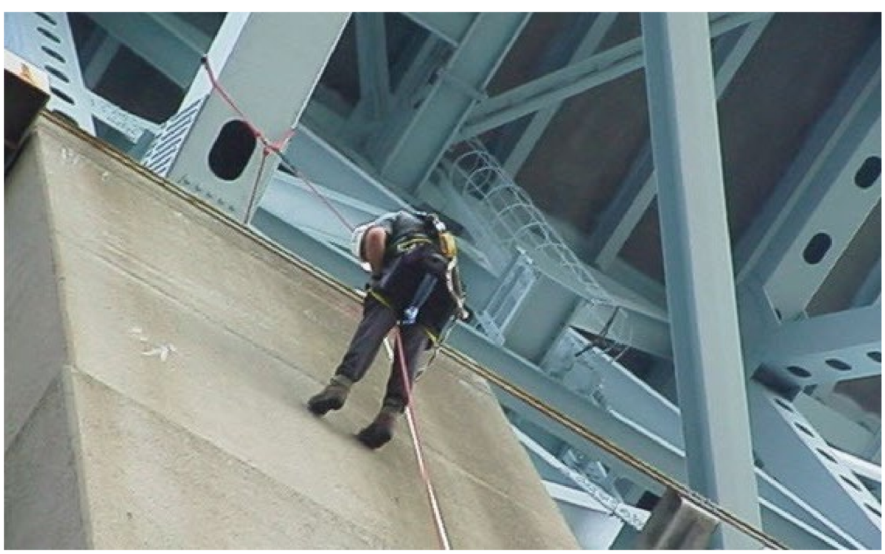


Figure 2.5.16 Inspector Rappelling Substructure Unit

Rope Access

On structures, where other methods of access are not practical, inspectors climb on the bridge members to gain access (see Figure 2.5.17). 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.

As rope access became more widely used in the 1990's, the Society of Professional Rope Access Technicians (SPRAT) was established to develop a set of safe standards for modern rope access systems. In addition to developing standards, SPRAT provides education and administers certification.

For more information on rope access, please reference *Certification Requirements for Rope Access Work* here: https://sprat.org/wp-content/uploads/2019/09/Certification_Requirements_19A.pdf

Figure 2.5.17 Rope Access

Unmanned Aerial System

Unmanned Aerial Systems (UAS), commonly referred to as drones, are useful in aiding inspection access but are not intended to replace inspectors. Using a UAS does not fulfill the hands-on access requirements for an NSTM inspection. UAS is discussed in more detail in Chapter 17.

Remotely Operated Vehicle

A remotely operated vehicle (ROV) acts as an extension of a video camera for Underwater inspection. The camera is mounted on a surface-controlled propulsion system. ROVs are discussed in more detail in Chapter 16.

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. All permanent inspection structures should be verified to be in good condition prior to use. The following are some examples of permanent inspection structures.

Catwalks

A catwalk is an inspection platform typically running parallel to the superstructure members (see Figure 2.5.18). 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 2.5.18 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 units (see Figure 2.5.19). Having the platform perpendicular to the girders provides a wider range of movement and enables the inspectors to see more of the superstructure units.

Figure 2.5.19 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 Figure 2.5.20 and Figure 2.5.21). Handrails are typically provided to assist the inspector when utilizing rope access on the bridge and give the inspector a place to secure their lanyard and safety harness. Handrails should also be inspected and any deficiencies should be noted so they may be addressed to ensure safety in future inspections.

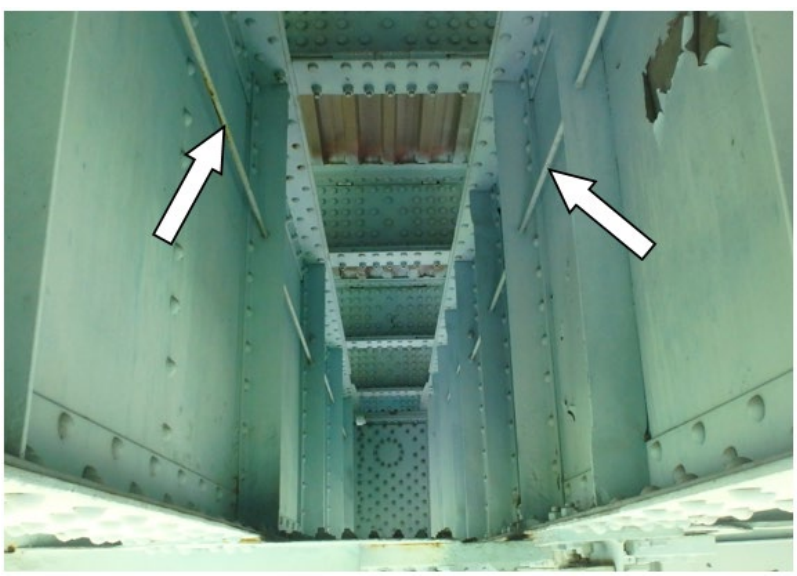


Figure 2.5.20 Handrails on Floorbeams

Figure 2.5.21 Handrail on Suspension Bridge

2.5.3 Efficiency

In most cases, even the most cumbersome inspection access equipment or vehicles will most likely be quicker than using a ladder, rigging, or rope access techniques to inspect a structure. The time saved, however, should offset the higher costs associated with obtaining and operating an access vehicle.

In assessing the time-saving effectiveness of inspection access equipment or vehicles, the following questions should be answered:

- Can the bridge be safely inspected by other reasonable methods?
- What types of access vehicles 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 need an independent operator or driver other than the inspector?
- Could the use of the access vehicle necessitate 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 using standard access equipment and the most appropriate decision can be made concerning access to various parts of the bridge.

PART I - BRIDGE SAFETY INSPECTION PROGRAM

CHAPTER 3 TABLE OF CONTENTS

CHAPTER 3 LIST OF FIGURES

CHAPTER 3 LIST OF TABLES

Chapter 3 Introduction to Inspection Records and Reporting

Section 3.1 Structure Inventory

3.1.1 Introduction

A thorough bridge inspection reporting system is essential to document bridge conditions and to protect public safety and investment in bridge structures. Therefore, it is 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 of the National Bridge Inspection Standards (NBIS), it is necessary to employ a uniform bridge inspection reporting system (23 CFR 650.315). A uniform reporting system is essential to evaluate the condition of a structure correctly and efficiently. It is a valuable aid in establishing maintenance and replacement priorities, and in determining structure capacity and the cost of maintaining the nation's bridges. The importance of a reporting system cannot be overemphasized, and the success of any bridge inspection program is dependent upon its reporting system. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

3.1.2 FHWA Structure Inventory

The FHWA *Specifications for the National Bridge Inventory (SNBI)* is used for defining the bridge inventory items to be collected, including information on the overall condition of the deck, superstructure, substructure, culvert, and channel. The data is reported to FHWA in accordance with the *SNBI*. Each Agency is responsible for having the capability to obtain, store, and report certain information about bridges, for collection by FHWA annually and as requested (23 CFR 650.15(a)).

The NBI data summary sheet is a tabulation of reported NBI data, and is useful for inspectors and others for viewing a summary of this information. States typically customize this summary for their own use.

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

Data Entry Requirements

For all inspection types, the NBIS requires entry of the NBI inventory data into the Agency's inventory within 3 months after the month when the field portion of the inspection is completed (23 CFR 650.315(b)).

For new bridges, existing bridge modifications that alter previously recorded data, and changes in load restriction or closure status, the NBIS requires entry of the NBI inventory data into the Agency's inventory within 3 months after the month of opening to traffic or the month of the change in load restriction or closure status of the bridge is implemented (23 CFR 650.315(c) and (d)).

Each Agency must establish and document a process that ensures the time constraint requirements in the NBIS. (23 CFR 650.315(e)).

Some states 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 Figure 3.1.1 and Figure 3.1.2) or wearable computers (see Figure 3.1.3 and Figure 3.1.4). Inspectors can record data and inspection findings, and also electronic files as necessary for reference.

Figure 3.1.1 Portable Computer with Inspection Forms

Figure 3.1.2 Inspector Using Portable Computer

Figure 3.1.3 Wearable Computer with Case

Figure 3.1.4 Inspector Using Wearable Computer

The data format is listed in the *SNBI*. It is important to note that several items listed in the *SNBI* may need to be updated by either the field or office personnel responsible for bridge inspections or both. The bridge inspection team 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 should remain unchanged. The inspection team is responsible for verifying that inventoried items are consistent with observations at the bridge site.

3.1.3 Inventory Data Items

Inventory data is defined as all data reported to the NBI in accordance with the *SNBI* (23 CFR 650.305). Inventory data items are broken into the following categories in the *SNBI*:

- Bridge Identification – Identifies the structure using location codes and descriptions.
- Bridge Material and Type – Categorizes the structure based on the material, design and construction, the number of spans, and wearing surface.
- Bridge Geometry – Includes pertinent structural and clearance dimensions.
- Features – Identifies highway and other features carried by, below, and above bridges.
- Loads, 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.
- Inspections – Includes latest inspection dates, designated interval, and critical features requiring special inspections or special emphasis during inspection including Underwater and Nonredundant Steel Tension Member (NSTM) inspection.
- Bridge Condition – provides information about the state of the bridge or culvert and any adjacent waterways. They include component condition ratings and element conditions.

All inventory data items are explained in the *SNBI*. Although many of the items are provided from the Initial inspection or previous reports, the inspection team is responsible for verifying and updating the data as necessary.

All bridge inventory data items are an important part of an owner's Bridge Management System (BMS). Bridge Owners use the items, such as bridge identification, bridge material and geometry, as well as the features on or under the bridge to help plan inspection, maintenance, and reconstruction of their bridges, as well as classify and categorize their bridges.

3.1.4 Bridge Material and Type Items

Bridge Material and Type items are defined in Section 2 of the FHWA *SNBI*. These data items are grouped into the following subsections:

- Subsection 2.1: Span Material and Type.
- Subsection 2.2: Substructure Material and Type.
- Subsection 2.3: Roadside Hardware.

Data items under span material and type identify the bridge configuration based on material(s), type(s), and continuity. Span material and type items are considered part of the span data set and include the following:

- Span Configuration Designation.
- Number of Spans.
- Number of Beam Lines.
- Span Material.
- Span Continuity.
- Span Type.
- Span Protective System.
- Deck Interaction.
- Deck Material and Type.
- Wearing Surface.
- Deck Protective System.
- Deck Reinforcing Protective System.
- Deck Stay-In-Place Forms.

These data items are reported for each span configuration present in the bridge. A span configuration characterizes all spans of similar material, type, and continuity. Spans of similar configuration need not be contiguous to be reported in the same data set. When applicable, additional data sets are reported. Deck Interaction, Deck Material and Type, Wearing Surface, Deck Protective System, Deck Reinforcing Protective System, and Stay-In-Place Forms are only not reported for bridges and culverts under fill. Typically, once a bridge is initially inventoried, these data items will not change.

Data items under substructure material and type identify the material(s), type(s), and configuration of the bridge's substructure and foundation. Data items in this subsection include the following:

- Substructure Configuration Designation.
- Number of Substructure Units.
- Substructure Material.
- Substructure Type.
- Substructure Protective System.
- Foundation Type.
- Foundation Protective System.

A substructure configuration characterizes all substructure units that have the same material, type, and foundation type. When applicable, one or more substructure sets are reported for the bridge. Substructures of similar configuration need not be adjacent to be reported in the same data set. The data items listed above are not reported when Span Type data item is a pipe.

Roadside hardware is discussed in greater detail in Chapters 4 and 8.

3.1.5 Appraisal Data Items

Appraisal data items are used to provide information on potential vulnerabilities for the bridge. Appraisal data items include:

- Approach Roadway Alignment.
- Overtopping Likelihood.
- Scour Vulnerability.
- Scour Plan of Action.
- Seismic Vulnerability.

Refer to the *SNBI* for all item coding information.

Section 3.2 Component Condition Ratings

3.2.1 Introduction

Condition ratings are used to identify the existing field conditions as compared to the as-built condition. Component condition ratings are typically coded by the inspection team leader, but in some cases can be determined after discussion or analysis with other pertinent engineers or experts. Condition ratings consider both the severity and the extent of any deterioration, defects, or damage, including the location(s) and whether structural capacity is affected. Component condition rating data items (*SNBI* Subsection 7.1) include:

- Deck – Summarizes the physical condition rating of the deck, as determined from the inspection of all deck surfaces. The condition of the surface/protective systems, joints, expansion devices, curbs, sidewalks, parapets, fascia beams, bridge rail and scuppers are not included in the rating, but the condition should be noted in the inspection form. Decks that are integral with the superstructure should be rated as a deck only, but can also influence the superstructure rating.
- Superstructure – Summarizes the physical condition of all the structural members of the superstructure. The condition of the bearings, joints, paint system, etc. should not be included in the rating except for extreme bearing defects. The condition should be noted in the inspection form. Superstructures that are integral with the deck should be rated as a superstructure only and not influence the deck rating.
- Substructure – Summarizes the physical condition of piers, bents, abutments, piles, footings, or other members directly supporting the superstructure.
- Culvert – Summarizes the condition and performance of the culvert, including footings, piles, and other foundation members. Evaluates the alignment, settlement, joints, structural condition, scour and undermining, and any other factors associated with a culvert.
- Bridge Railing – Summarizes the physical condition of all bridge railings including parapets, structure mounted, and median barriers. Assessment includes the portions of the railings, posts, blocking, and curbs that are part of the bridge railing system. This component is part of roadside hardware.

- Bridge Railing Transition – Summarizes the condition of the transition between the bridge railing and the approach guardrail. Assessment of this component includes the portions of the railings, posts, blocking, and curbs that are part of the bridge railing transition. This component is part of roadside hardware.
- Bridge Bearings – Summarizes the condition of all types and shapes of bridge bearings. In cases where bridge bearings are not visible, a condition rating of this component can be judged based on alignment and grade across the joint along with other indirect factors indicating bearing performance.
- Bridge Joints – Summarizes the condition of all types and shapes of bridge deck joints. In cases where the joint is not visible, the joint condition can be assessed based on other indirect factors indicative of the condition.
- Channel and Channel Protection – Summarizes 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.
- Scour – Reports the scour condition that represents the observed or measured scour at a bridge site.
- NSTM Inspection – Represents the condition rating of any NSTM(s) to be inspected in an NSTM Inspection, incorporated into the superstructure or substructure condition rating.
- Underwater Inspection – Represents the condition rating of any underwater members to be inspected in an Underwater Inspection, incorporated into the substructure condition rating.

To ensure a comprehensive inspection and as a part of the requirements of recordkeeping and documentation, an inspector is responsible for recording the location, type, size, quantity, and severity of deterioration and deficiencies for all areas of a given component.

3.2.2 Evaluation of Bridge Condition Components

NBI component condition ratings for deck, superstructure, substructure, culverts, joints, bearings, roadside hardware, and channel and channel protection components serve to rate the structural condition and capacity of the component, as well as the functionality, in comparison to its original condition. Component condition ratings range from 0 to 9, shown below.

- Ratings 7-9 – Good condition.
- Ratings 5-6 – Fair condition.
- Ratings 0-4 – Poor condition.

The following descriptors are generally used in establishing a component's condition rating:

- Good – Component has inherent defects or some minor defects.
- Fair – Component has widespread minor defects and/or some moderate defects, but strength and performance of the component are not affected.
- Poor – Structural capacity of components is affected. Condition typically necessitates monitoring, load restrictions and/or corrective actions in order to keep the bridge open.

These classifications are coded in the Bridge Condition Classification item to aid with national performance measures. The Lowest Condition Rating Code item represents the lowest rating of the major components. Both of these *SNBI* items are automatically calculated based on the lowest condition rating code of the deck, superstructure, substructure, and culvert ratings.

Refer to Chapter 18 for further detail on component condition rating systems. Refer to the *SNBI* for specific guidance with regard to component condition ratings.

Accurate assignment of condition ratings is typically dependent upon the bridge inspector's ability to identify the bridge components and their associated members. Bridge components are the major parts comprising a bridge, and include the deck, superstructure, substructure, joints, bearings, and roadside hardware. Bridge elements are individual members that comprise the bridge components. In some cases, a component is comprised of only one primary member or element, such as a deck or culvert.

When assigning a culvert condition rating, all areas of the culvert and the possible effects on the overall structure should be investigated. The inspection team should consider 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. Chapter 15 addresses the individual characteristics of various culverts. The overall component condition rating considers all of the parts and elements that make up a culvert. The rating summarizes the condition and includes evaluation of alignment, settlement, joints, structural condition, scour, and other items associated with culverts. Headwalls and integral wingwalls to the first construction or expansion joint are included in the evaluation.

For structures positioned over waterways, a condition rating is provided for both the channel and channel protection according to the *SNBI*. A channel condition rating is assigned based on the waterway conditions and a channel protection condition rating is assigned based on the effectiveness and condition of channel protection devices and structures.

These items describe 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. Channel stability is very important, as channel instability and scour are leading causes of bridge failures. Channel cross-section measurements, underwater inspections, and observations of channel characteristics are essential and are discussed in subsequent chapters.

The inspection team should look for 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 in the inspection report, but not included in the component condition rating of the superstructure and substructure.

Section 3.3 Element Level Evaluation

3.3.1 Introduction

Although component condition rating and reporting, as described in the *SNBI*, provides a consistent method for evaluation and reporting, the data by itself is typically not comprehensive enough to support bridge preservation performance-based decision support.

In developing a system for standardized data collection, FHWA needed to look at the structure of component condition data where each bridge is divided into only the major parts for condition assessment. A system was developed which included a standardized description of bridge elements at a greater level of detail.

The National Bridge Element and Bridge Management Element system provides multiple distress paths for each defined condition state. This enables 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. The 2011 version was replaced in 2013 with the AASHTO *Manual for Bridge Element Inspection (MBEI), First Edition 2013*. In 2015, *Interims for the Manual* were released. The document was updated again in 2019 as *MBEI, Second Edition*.

In 2022, the NBIS (23 CFR 650.315) was updated to require that bridge inventory data must include element level bridge inspection data for bridges on the National Highway System (NHS). The NBIS also incorporated the MBEI and SNBI by reference (23 CFR 650.317) and established the AASHTO *MBEI* as a guide for identifying and quantifying bridge elements for reporting to FHWA. The *SNBI* is used in conjunction to assign condition codes for bridge elements.

3.3.2 Element Level Rating Terminology

The AASHTO *MBEI, Second Edition, 2019* (see Figure 3.3.1) provides a description of structural elements that are commonly used in highway bridge construction and encountered on bridge safety inspections.

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 and protective coatings.

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 generally 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 is used to indicate good condition and Condition State 4 is used for severe conditions.

Environments are typically 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.

Specific defect descriptions are incorporated into the National Bridge Element or Bridge Management Element condition state definitions. Defect quantities are reported to document the extent of an element exhibiting a particular defect. This defect data inherits the same units of measure as the NBE or BME to which they are assigned.

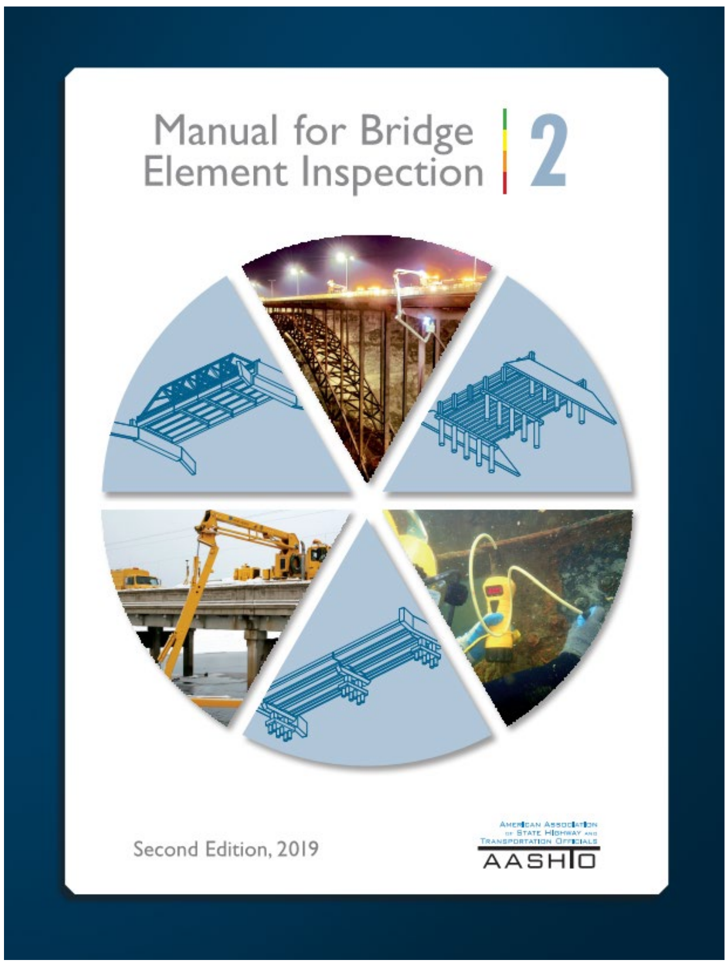


Figure 3.3.1 AASHTO Manual for Bridge Element Inspection

3.3.3 Bridge Element Identification

In the development of National Bridge Elements, it was emphasized that the specification be generic and have flexibility, as different agencies have varying maintenance practices, funding mechanisms, policy concerns and terminology. However, since the physical components of bridges and deterioration processes are not unique, agencies should 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 AASHTO *MBEI* allows an agency to add custom agency developed elements or modify recommended Bridge Management Elements.

One primary use of definitions is to establish a useful inventory. In the field, each element is clearly identified, measured, and counted economically. It is also important to quantitatively describe element attributes, such as size, material, condition, and serviceability. 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.

National Bridge Elements

National Bridge Elements describe primary load carrying members, including:

- Decks.
- Slabs.
- Superstructures.
- Substructure.
- Culverts.
- Railings.
- Bearings.

See Table 3.3.1 for a table of AASHTO National Bridge Elements to be reported per the *SNBI*. NBEs are reported as NBI data with Element Identification items for element number and total quantity in the *SNBI*.

Table 3.3.1 National Bridge Elements Reported to FHWA

Element	Units	Element Number					
		Steel	Concrete		Timber	Masonry	Other
			P.S.	R.C.			
Deck/Slab							
Deck	SF		13	12	31		60
Open Grid Deck	SF	28					
Concrete Filled Grid Deck	SF	29					
Corrugated/Orthotropic Deck	SF	30					
Slab	SF			38	54		65
Top Flange	SF		15	16			
Superstructure							
Closed Web/Box Girder	LF	102	104	105			106
Girder/Beam	LF	107	109	110	111		112
Stringer	LF	113	115	116	117		118
Truss	LF	120			135		136
Arch	LF	141	143	144	146	145	142
Main Cable	LF	147					
Secondary Cable	EA	148					149
Floor Beam	LF	152	154	155	156		157
Pin, Pin and Hanger Assembly	EA	161					
Gusset Plate	EA	162					
Substructure							
Column	EA	202	204	205	206		203
Column Tower (Trestle)	LF	207			208		
Pier Wall	LF			210	212	213	211
Abutment	LF	219		215	216	217	218
Pile Cap/Footing	LF			220			
Pile	EA	225	226	227	228		229
Pier Cap	LF	231	233	234	235		236
Culvert							
Culvert	LF	240	245	241	242	244	243
Bridge Rail							
Bridge Rail	LF	330*		331	332	334	333
Bearing							
Elastomeric	EA	310					
Movable (roller, sliding, etc.)	EA	311					
Enclosed/Concealed	EA	312					
Fixed	EA	313					
Pot	EA	314					
Disk	EA	315					
Other	EA	316					

*Element 330 - Metal Bridge Rail may include steel or aluminum rails.

Bridge Management Elements

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.
- Wearing Surfaces.
- Protective Systems.

See Table 3.3.2 for a table of AASHTO Bridge Management Elements to be reported per the *SNBI*. BMEs are also reported as NBI data with Element Identification items for element number, parent number (for wearing surfaces and protective systems), and total quantity in the *SNBI*.

Table 3.3.2 Bridge Management Elements Reported to FHWA

Element	Units	Element Number
Joint		
Strip Seal	LF	300
Pourable	LF	301
Compression	LF	302
Assembly with Seal (Modular)	LF	303
Open	LF	304
Assembly without Seal	LF	305
Other	LF	306
Wearing Surfaces and Protective Coatings		
Wearing Surfaces	SF	510
Steel Protective Coating	SF	515
Concrete Protective Coating	SF	521

Agency Developed Elements

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

3.3.4 Condition States

The four condition states are as follows:

- Good (Condition State 1) - No deterioration to minor deterioration.
- Fair (Condition State 2) - Minor to Moderate deterioration.
- Poor (Condition State 3) - Moderate to Severe deterioration.
- Severe (Condition State 4) - Beyond the limits established in Condition State 3 or warrants a structural review to determine strength or serviceability of the element or bridge.

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.

The assignment of quantities to a condition state is determined from the element definitions and element commentary for National Bridge Elements. Condition state definitions serve as recommendations to the bridge inspection team for categorization of the severity of the deficiency. Element commentary represents additional considerations for the inspection team during the collection of data. From this information, the inspection team can complete the element level evaluation. Condition states are reported as NBI data with Element Condition *SNBI* items that quantify the elements in each condition state.

3.3.5 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 is stored so that the analysis may be updated or improved at a later time. Bridge elements should be usable to support bridge management decisions. Condition state data provides quantitative data about the physical condition and performance of bridge elements. Element level evaluations can track the effectiveness of actions, by showing the various condition states and how they may change over time, especially after the bridge element is repaired, replaced, or no action is chosen. Element and condition state information may also be useful to help determine maintenance needs, treatment selections, or estimate budgeting and funding allocation for programming or long-range planning.

Section 3.4 Critical Findings

3.4.1 Introduction

A critical finding is a structural or safety related deficiency that requires immediate action to ensure public safety (23 CFR 650.305).

A structure related deficiency of this category would likely include those causing loss of adequate load capacity or stability, potentially leading to partial or total collapse of the structure. Critical findings may also include non-structural deficiencies which jeopardize the safety of motorists or pedestrians.

3.4.2 Procedures

The NBIS regulations direct each Agency to establish a procedure to assure that critical findings are addressed in a timely manner (23 CFR 650.313(q)). Although specific procedures vary among agencies, general steps should be taken to assure that critical findings are identified and resolved as quickly and efficiently as possible.

Per the NBIS Section 650.313 (q)(1)(i), critical finding procedures must define critical findings considering the location and the redundancy of the member affected and the extent and consequence of a deficiency. At a minimum, include findings which warrant:

- Full or partial closure of any bridge.
- An NSTM to be rated in serious or worse condition, as defined in the NBI by the NSTM Inspection item, coded (3) or less.
- A deck, superstructure, substructure, or culvert component to be rated in critical (2) or worse condition.
- The channel condition or scour condition to be rated in critical (2) or worse.
- Immediate load restriction or posting, or immediate repair work to a bridge, including shoring, in order to remain open.

In accordance with NBIS regulations, the agency is also required to inform the FHWA of all critical findings. FHWA must be notified within 24 hours upon discovery of the following critical findings that occur on NHS bridges:

- Full or partial bridge closure.
- NSTM Inspection Item coded 3 or less.

All critical findings must be reported monthly or as requested in written status reports to FHWA until resolved for all NHS or non-NHS bridges (23 CFR 650.313 (q)(2)(ii)).

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 or e-mail notification is required when the potential critical deficiency is identified. Public works officials or law enforcement may also be contacted as necessary.

In addition to a verbal notification, agencies may require immediate written notification of the potential critical finding. 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. In some cases, these actions may be permanent. These actions may include:

- Load posting.
- Traffic restrictions from the damaged area.
- Speed restrictions.
- Temporary lane closure.
- Temporary shoring/cribbing.
- Complete bridge closure.

The finding may need to be further assessed by other engineers or experts, and the severity determined along with a proposed repair strategy or plan of action.

In the past, states have employed one of two approaches to coding condition items when localized areas of severe deterioration are encountered. Some may 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., a 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).

Other states have rated an entire component based on 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 defect or damage is not improved following the critical finding follow-up process, the component rating would need to be lowered to account for the severity of the deterioration if structural capacity is affected. Since the component condition rating considers both the severity and extent of the deficiencies and any effect on structural capacity, the condition rating should initially be lowered to include any effects of the finding. If the critical finding is immediately rectified as intended by definition, then the rating should be readjusted upward as appropriate.

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 intolerable time frame 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 inspection team, inspection program manager, or bridge maintenance supervisor, follow established Agency procedures. Procedures should include immediate notification by inspectors to appropriate personnel so that expedited action can be taken as necessary to ensure public safety.

3.4.3 Examples of Critical Findings

An example of a follow-up to finding a critical finding issue may include a procedure in which the Agency promptly submits a copy of inspection reports or recommendations to the Division office for bridges that meet the following criteria:

- Primary structural member(s) with collision damage or deterioration affecting structural capacity.
- Nonredundant Steel Tension Members (NSTMs) with damage or deterioration affecting structural capacity.
- Substructure units with severe scour and undermining of foundation(s) causing instability.
- Superstructure or substructure condition ratings of 2 or less.

Many 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 guidance in determining whether or not a deficiency may be considered a critical finding.

Several examples of critical findings are listed below. These deficiencies represent excerpts obtained from several agencies' critical finding documentation. The examples may be critical findings based on either safety concerns or structural issues, or both.

- Decks spalls which could result in a loss of control of the vehicle (safety).
- Primary structural members with damage or deterioration (structural).
- Steel members with deteriorated areas that have failed in buckling, crippling, etc., or which makes failure likely in the near future (structural).
- Prestressed girder with spalling and broken strands or 100% deterioration at critical high stress areas (structural).
- Falling concrete or concrete that is delaminated or partially detached and anticipated to fall, presenting a hazard to under-passing motorists and/or pedestrians (safety).
- Expansion joints that are deteriorated, damaged, or loose which may present a hazard to passing traffic (safety).
- Rocker bearings that are critically tilted exceeding the tolerable amount of tilt or bearing on the outer one-quarter width of the rocker (structural).
- Bearing seats that are severely deteriorated or undermined (structural).
- Substructure units with severe scour and undermining of the foundation causing instability (structural).
- 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 (safety).
- Load posting or vertical clearance signs that are missing, damaged, improperly situated, or visually obstructed including relevant advance warning signs (safety).

3.4.4 Examples of Actions Taken

As previously mentioned, an Agency-wide procedure must be established to assure that critical findings are addressed and resolved in a timely manner. The appropriate actions to be used for repair or mitigation of the critical finding should be quickly identified and efficiently carried out. In accordance with the NBIS, the FHWA must be notified of the actions that have been taken to resolve critical findings (23 CFR 650.313(q)). It is required that bridge owners implement standard procedures for immediately addressing and resolving critical deficiencies.

Some important steps to remember when implementing these standard procedures are:

- Rapid evaluation of the deficiencies.
- Emergency notification of police and the public (if necessary).
- Rapid implementation of corrective or protective actions.
- A tracking system to ensure adequate and timely follow-up.
- Provisions for identifying other bridges with similar details for follow-up inspections.

After the Agency's plan to address the critical finding has been accepted, recommended repair work should then be performed, and a post-repair report should be generated documenting all necessary work done to resolve the critical finding and the date of completion. This information should be included in the monthly status report provided to FHWA. A follow-up inspection should also be conducted to assess the condition of the repairs. Depending on the complexity and nature of the repairs, an initial inspection may be necessary to update the NBI inventory items within 3 months of completion of the repair.

Section 3.5 Recordkeeping and Documentation

3.5.1 Introduction

It is of paramount importance that 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 information that may be important for repair, rehabilitation, or replacement of their assets.

3.5.2 Bridge Records

Bridge records, or files, are generally 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 bridge file is not only a resource to the bridge owner, but also a resource to the inspection team. The inspection team will likely gain valuable insight into the bridge by being familiarized with it before the inspection.

Refer to Chapter 18 for a list of recommended items to be included in the bridge file.

3.5.3 Methods of Inspection Documentation

Traditionally, the most commonly used method for recordkeeping is pencil and paper. The inspection team writes findings on forms, sketches, and notebooks. This method can be extremely flexible in that the inspection team can draw whatever configurations are necessary to best describe and document deficiencies.

Another method of recordkeeping is electronic data collection (see Figure 3.5.1).

Figure 3.5.1 Electronic Data Collection

This technology can provide a significant advantage in several areas. With all the bridge data available at the site, the inspection team 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 may eliminate errors that can occur when transferring the inspector's field notes to the computer back at the office. Electronic data collection can provide a logical and systematic sequence of inspection, ensuring that no portions of the bridge are overlooked. It also enables the inspection team to compare the current deficiencies with previous reports and note if any deterioration has gotten worse.

Section 3.6 Inspection Report Basics

3.6.1 Introduction

The bridge inspection report enables trained and experienced personnel to record objective observations of all parts of a bridge and to make logical deductions and conclusions from their observations.

The bridge inspection report represents a systematic documented summary of the current condition of all bridge components, members, and elements. Moreover, bridge reports often form the basis of quantifying the labor, 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.

3.6.2 Minimum Requirements of a Report

The bridge inspection report documents signs of distress and deterioration with sufficient precision so that future inspectors can readily make a comparison of condition. Even if no changes are evident, reports are still generated for each type of bridge inspection.

Bridge owners typically choose the format to be used when preparing a bridge inspection report. During an inspection, inspectors should gather enough information to ensure a comprehensive and complete report. An inspection report reflects the level of effort an inspector took to perform the inspection and the report is a record of both the bridge condition and the inspector's work. A complete inspection report contains several parts, depending on the type of inspection. Items to be included in each type of inspection report are outlined in detail in Chapter 18.

An inspection report generally includes the following information at a minimum:

- Documentation of Team Leader.
- Date of inspection.
- Summary of inventory items.
- Clear and concise descriptions of bridge condition.
- Inspection procedures.
- Evaluation of component ratings and/or element condition states.
- Photographs and sketches.
- Recommendations for items that necessitate review or action.

3.6.3 Reviewing Existing Inspection Information

It is important to understand the history of a bridge before performing a field inspection. In addition to a review of the bridge file and the planning activities presented in Chapter 2, the previous inspection reports should be reviewed to form an understanding of the needs for upcoming inspections.

Past inspection reports and the bridge file contain vital information such as:

- Inspection access requirements.
- Deterioration profiles.
- Condition ratings and/or element condition states.
- Pertinent rehabilitation or repair history.
- Areas to be monitored.
- Additional information requests.

Refer to Chapter 2 for detailed information on planning and performing the inspection.

PART II – BRIDGE, CULVERT, AND WATERWAY CHARACTERISTICS

CHAPTER 4 TABLE OF CONTENTS

CHAPTER 4 LIST OF FIGURES

Chapter 4 Basic Bridge, Culvert, and Waterway Characteristics

Section 4.1 Basic Bridge Characteristics

4.1.1 Introduction

It is important to be familiar with the terminology and elementary theory of bridge characteristics, mechanics, and materials. This section presents the terminology necessary for inspectors to properly identify and describe the individual members that comprise a bridge. The major components of a bridge are introduced, and the basic member shapes and connections of the bridge are described. Finally, the purpose and function of the major bridge components are described in detail. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

4.1.2 NBIS Bridge Length

23 CFR 650.305 gives the definition of a bridge as it applies to the NBIS regulations. Applicable sections of the Code of Federal Regulations (CFR) are provided at the end of this Manual. 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 ft between under copings 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.

The minimum length for a structure to be considered a bridge for National Bridge Inspection Standards purposes, is to be greater than 20 ft as shown in see Figure 4.1.1.

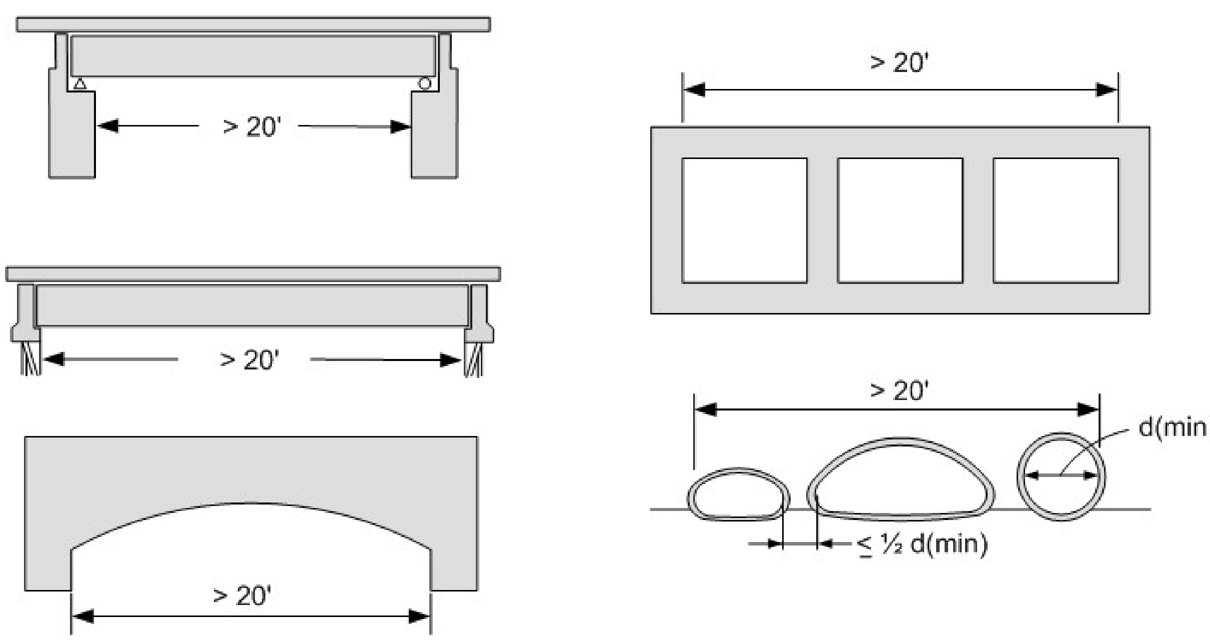


Figure 4.1.1 NBIS Bridge Length

A key stipulation regarding classifying pipe culverts as a qualifying NBIS bridge is the spacing of culverts (less than or equal to one half of the minimum diameter). This is often missed. It is important to note that this dimension is taken from inside face to inside face. The NBIS bridge length for each bridge in the NBI is recorded using the *SNBI* Item B.G.01.

4.1.3 Major Bridge Components

A thorough and complete bridge inspection is dependent upon the bridge inspector's ability to identify and understand the function of the major bridge components and their members. Each component is comprised of one or more members, discussed in detail in Chapter 4. Most bridges can be divided into several basic parts or components: (see Figure 4.1.2):

- Roadside hardware.
- Deck.
- Superstructure.
- Bearings.
- Substructure.
- Culvert.

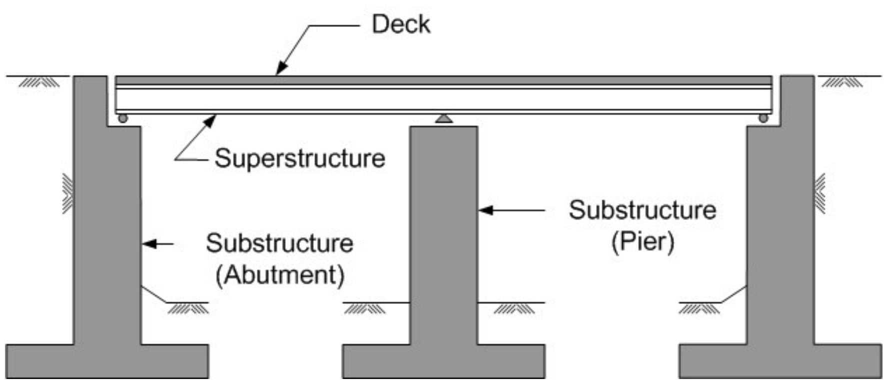


Figure 4.1.2 Major Bridge Components

4.1.4 Basic Member Shapes

The ability to recognize and identify basic member shapes necessitates an understanding of common materials such as concrete, steel, and timber shapes used in the construction of bridges. Iron is no longer a common bridge material, but was widely-used historically and therefore bridge inspectors may encounter iron members on older bridges. There are several other bridge materials that are less common, including aluminum, fiber reinforced polymer (FRP), masonry, and plastic. The Span Material is reported using the *Specifications for the National Bridge Inventory (SNBI)* Item B.SP.04.

Every bridge member is designed to carry a unique combination of tension, compression, shear, and torsion. These are considered the 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. Torsion forces are caused by a transverse force twisting about the member. As such, certain shapes and materials have distinct characteristics in resisting the applied loads. For a review of bridge loadings and member responses, refer to Chapter 6.

Concrete Shapes

Basic materials, properties, reinforcement, deficiency, protective systems, and examination of concrete are covered in detail in Chapter 7.

Figure 4.1.3 Unusual Concrete Shapes

Concrete is a unique material for bridge members because it can be formed into an infinite variety of shapes (see Figure 4.1.3). Concrete members are typically used to carry axial, bending, shear, and torsion loads. Since bending results in a combination of compressive and tensile stresses, concrete bending members are typically reinforced with 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 4.1.4):

- 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). Several of the designs are no longer being built today, but these structure types have been built in the past and still remain in service. Concrete members of this type are generally used for short and medium span bridges.

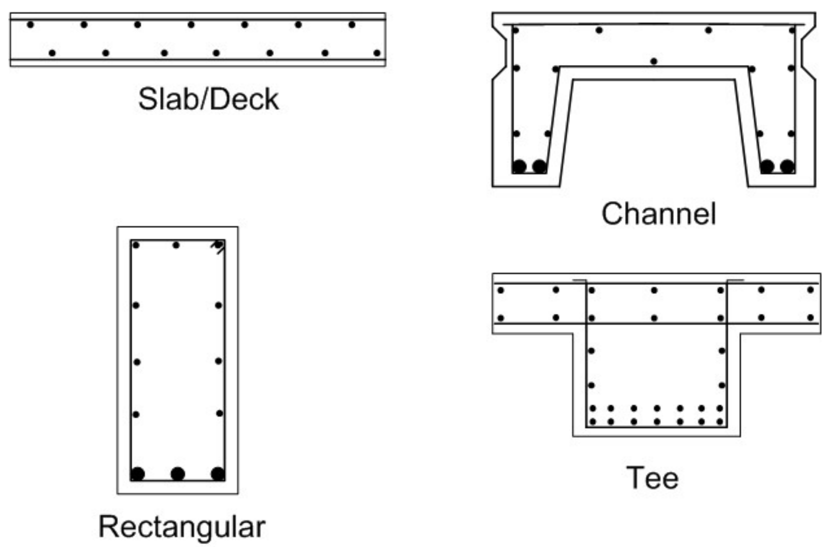


Figure 4.1.4 Reinforced Concrete Shapes

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 members are usually 12 to 24 inches thick.

Rectangular concrete shapes may be used for both superstructure and substructure bridge members. Concrete pier caps are commonly rectangular beams that support the superstructure. Although rare, rectangular concrete beams may be used in concrete through girder bridges.

Tee beams are generally limited to superstructure members. 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 members. These particular shapes 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. Adjacent channel beams are connected with bolts, shear keys or transverse post-tensioned rods. Refer to Chapter 9 for further information on these superstructure types.

Precast Flexural Shapes

The most common precast shapes include the following prestressed concrete members (see Figure 4.1.5):

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

These shapes are generally used for superstructure members.

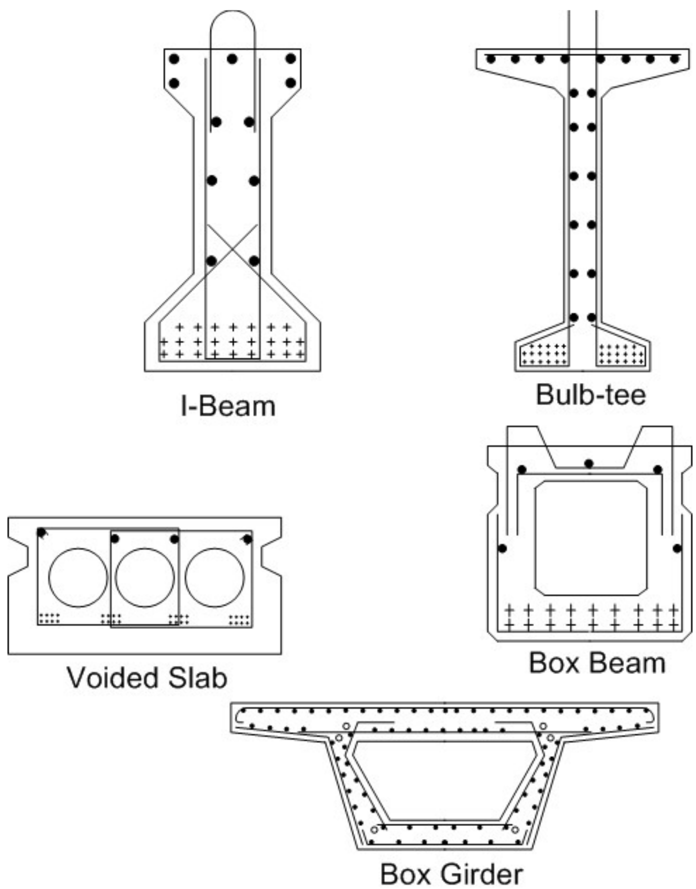


Figure 4.1.5 Prestressed Concrete Shapes with Typical Reinforcing Layouts

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 enable these members to carry greater loads. Therefore, they may be 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 the 1940s.

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 typically very high strength, resulting in longer spans.

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 ft.

Bulb-tee beams are distinguished by their "T" shapes, with a bulb-shaped section at the bottom of the vertical leg of the tee. This type of beam can be used for spans as long as 200 ft. Bulb-tee beams are coded as I-shaped adjacent or spread girders in the *SNBI* Span Type item.

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 span lengths that range 40 to 130 ft and spread box beams have span lengths that range up to 130 ft. Adjacent box beams may or may not have a deck but spread box beam superstructures will always have a deck.

Box girders, distinguished by their trapezoidal or rectangular box shapes, can function as both deck and superstructure. They are characterized by larger widths and depths than box beams. Box girders are typically used for long span or curved bridges and can be cast in place or precast and erected in segments, often referred to as segmental box girders. Span lengths can range from 130 to 1000 ft.

Voided slabs, distinguished by their rectangular shape and their interior circular voids, are generally precast units supported by the substructure. They are often confused with box beams. The interior voids reduce the dead load. Voided slabs have a typical maximum depth of less than 17 inches, use circular voids and can be used for spans up to 40 ft.

Double-tee beams are a less commonly used precast beam type, as it is more commonly seen in buildings. These are mostly used on shorter span bridges and can be a solution if superstructure dead load is a concern.

Axially Loaded Compression Shapes

Concrete loaded compression members are typically used in bridges in the form of columns or piles.

Tie bars decrease the unsupported length of compressed rebars and increase the axial capacity of concrete member without increasing the longitudinal rebars or concrete section.

Columns are straight members that can carry axial load, horizontal load, and bending forces, and can be used as substructure or superstructure members. Columns are commonly square, rectangular, or round. Columns may be cast in place or precast.

Piles are slender columns that support the substructure footing or partially form the substructure. Piles may be partially above ground (see Figure 4.1.6) but are completely buried when supporting a footing or other substructure unit. Concrete piles may be conventionally reinforced or prestressed.

Figure 4.1.6 Concrete Piles

Steel Shapes

Steel bridge members began to be used in the United States in the late 1800s 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. These advances yielded a steel material that surpassed iron in both strength and elasticity. Steel could carry heavier loads and withstand the shock and vibration of ever-increasing live loads. Since the early 1900s, the quality of steel has continued to improve. Stronger and more ductile ASTM A36, A572, A588, and, more recently, today's high-performance steels (HPS) offer an increase in corrosion/section loss resistance and strength/toughness increases compared to older "regular" steels such as ASTM A7.

Due to their strength and other characteristics, steel bridge members may be used to carry axial forces as well as bending and shear forces. Steel shapes are generally rolled or built-up.

Rolled Shapes

Rolled steel shapes commonly used on bridges include (see Figure 4.1.7):

- Rods.
- Bars and plates.
- Angles.
- Tees.
- Channels.
- S Beams (American standard "I" beams).
- W Beams (Wide flange "I" beams).
- H Shapes.

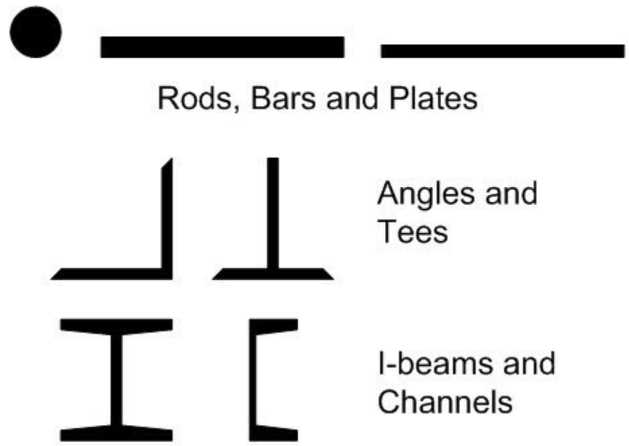


Figure 4.1.7 Common Rolled Steel Shapes

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. Pieces less than 8 inches wide are commonly called bars. Pieces greater than or equal to 8 inches wide are commonly called plates. Common examples of bars include lacing bars on a truss and steel eyebars. A common example of a plate is the gusset plate on a truss. Gusset plates are typically used to connect truss chords, verticals, and diagonal members. Lacing bars and gusset plates are described in more detail in Chapter 10. 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 ½” 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 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”.
- 2 L’s 5” x 3” x 3/8” x 1’-1”.

Angles range in size from 1” x 1” x 1/4” to 8” x 8” x 1-1/8”. 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 4.1.8).

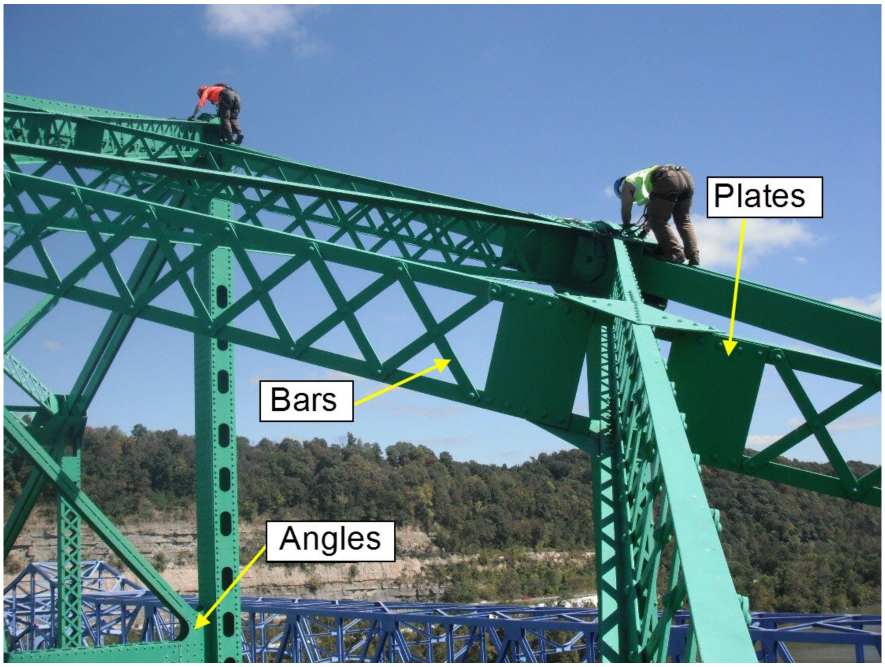


Figure 4.1.8 Bracing Members Made from Angles, Bars, and Plates

Channels are squared-off "C"-shaped members and are typically 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 necessary. From this information, the inspector can then determine the true channel designation through the use of reference books such as *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 that are "I"-shaped sections are typically 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.
- W = Wide flange.
- WF = Wide flange.
- CB = Carnegie.
- M = Miscellaneous.
- HP = H-pile.

To identify an I-beam, measurements of the depth, the flange width and thickness, and the web thickness (if possible) are necessary. With this information, the inspector can then determine the beam designation from reference books such the *AISC Manual of Steel Construction*. These beams typically 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 enable the bridge engineer to customize the members for their particular need. Built-up shapes are fabricated by riveting, bolting, or welding techniques.

The practice of riveting steel shapes began in the 1800s and continued through the 1950s. Typical riveted shapes include truss members, girders, columns, 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 (see Figure 4.1.9).

Riveted boxes are large rectangular shapes fabricated from plates, angles, or channels. These boxes are generally used for cross-girders, truss chord members, and substructure members (see Figure 4.1.10).

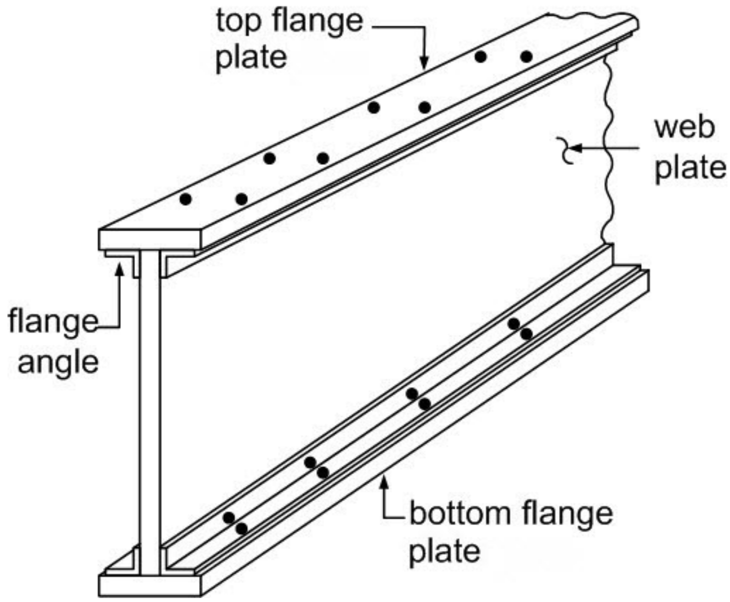


Figure 4.1.9 Riveted Plate Girder

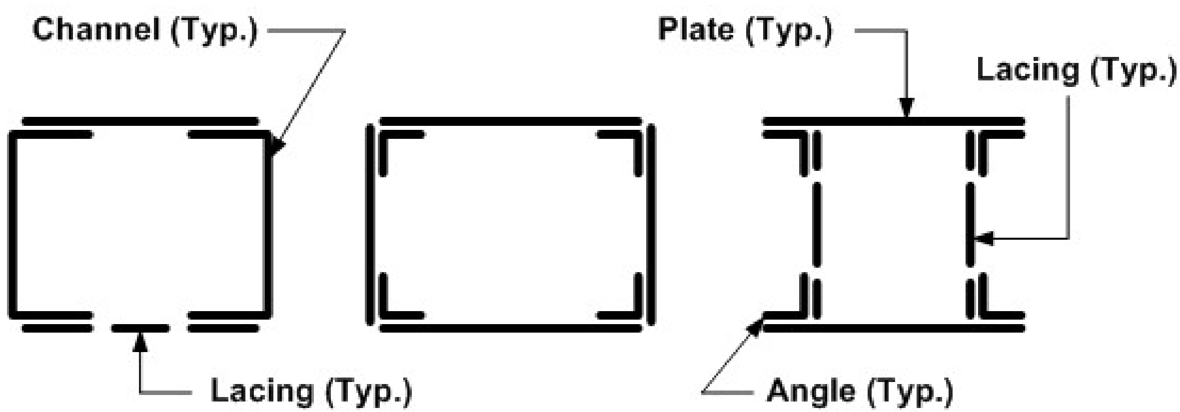


Figure 4.1.10 Riveted Box Shapes

As technology improved, riveting was replaced by high strength bolting and welding. Commonly used since the early 1960s, 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 4.1.11).

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 4.1.12).

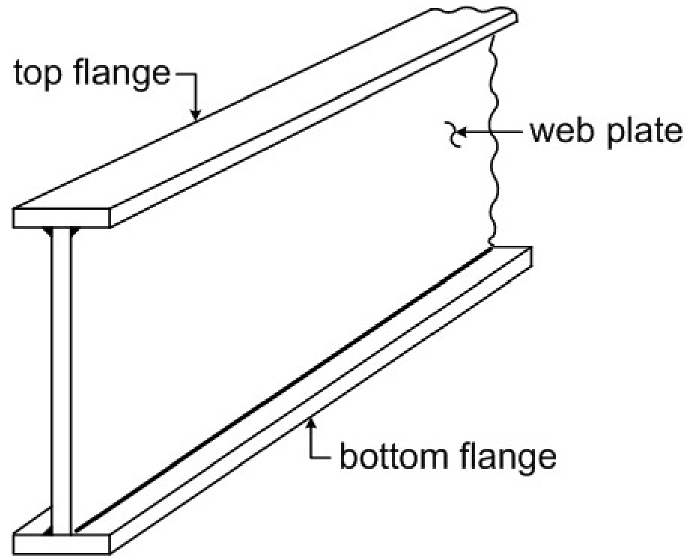


Figure 4.1.11 Welded I-Beam

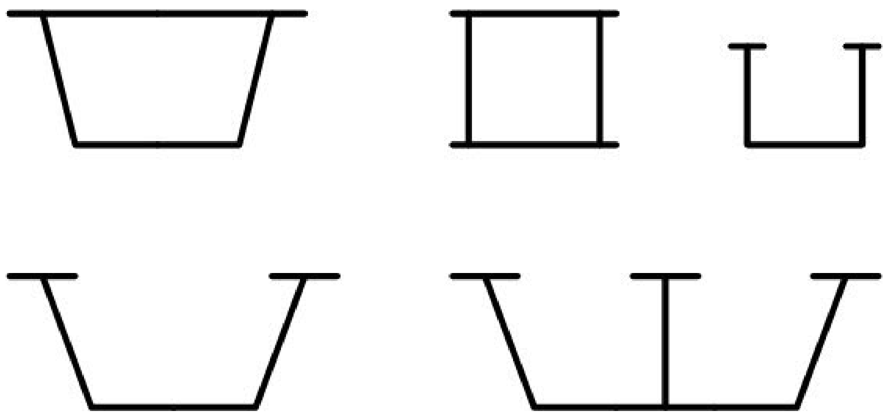


Figure 4.1.12 Welded Box Shapes

Cables

Steel cables (see Figure 4.1.13) are tension members and are generally used in suspension, arch, and cable-stayed bridges. They are typically used as main cables and hangers of these bridge types (see Figure 4.1.14 and Figure 4.1.15). Refer to Chapter 5 for a more detailed description of cable-supported bridges.

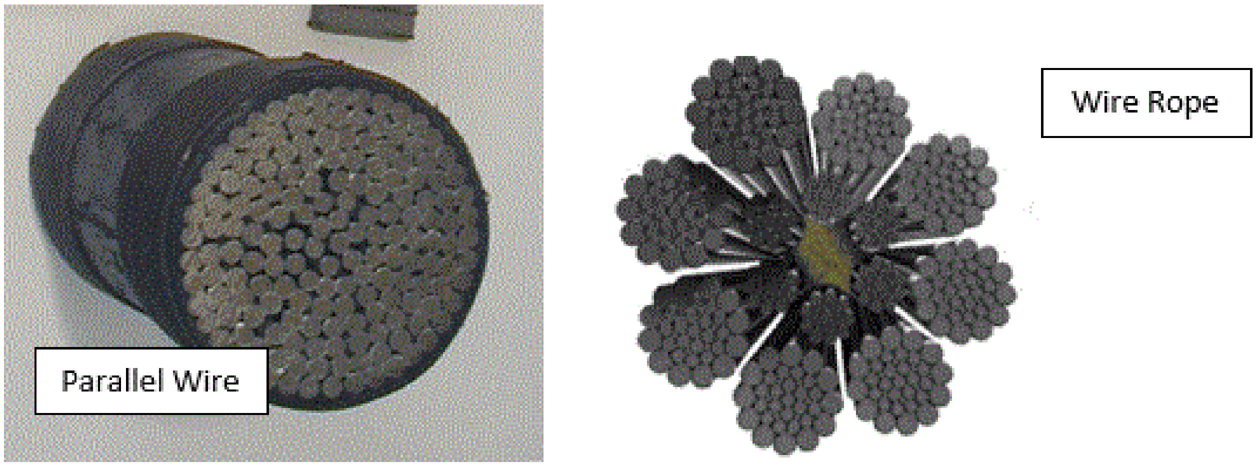


Figure 4.1.13 Cable Cross-Sections

Figure 4.1.14 Cable-Supported Bridge: Suspension Cables and Hangers

Figure 4.1.15 Cable-Supported Bridge: Cable-Stayed

Timber Shapes

Basic shapes, properties, gradings, deficiencies, protective systems, and examination of timber are covered in detail in Chapter 7.

Timber members are found in a variety of shapes (see Figure 4.1.16). The sizes of timber members are generally given in nominal dimensions (such as in Figure 4.1.16 and Figure 4.1.17). Nominal is an approximate or rough-cut dimension by which a material is generally called or sold in trade, but which differs from the actual dimension. In lumber trade, for example, a finished (dressed) 'two by four' piece is less than 2 inches thick and less than 4 inches wide. However, sawn timber members are generally seasoned and surfaced from the rough sawn condition, making the actual dimension approximately 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 milled and constructed into three basic shapes:

- Rectangular - planks, beams, columns, piles.
- Built-up shapes - beams, decks, slabs.
- Round - piles, columns.

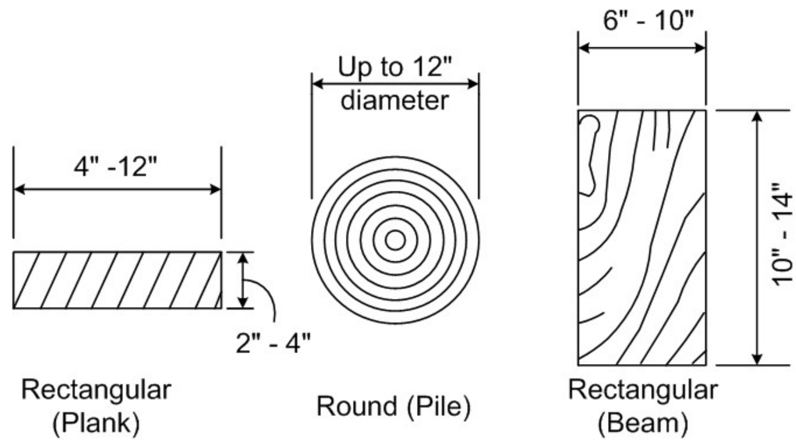


Figure 4.1.16 Timber Members with Typical Dimensions for Bridges

Planks

Planks are characterized by elongated, rectangular dimensions that are determined by the intended bridge features. Plank thickness is dependent upon the distance between the supporting points and the magnitude of the vehicle load. Planks are generally installed with the longer dimension horizontal. 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 4.1.16).

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 generally used in bridge superstructures and substructures to carry bending and axial loads.

Timbers can be solid sawn or built-up glued-laminated planks (see Figure 4.1.17). Glued-laminated timbers are advantageous in that they can be fabricated from smaller, more readily available pieces. Glued lamination also enables larger rectangular members to be formed without the presence of natural deficiencies such as knots. Glued-laminated timbers are typically manufactured from well-seasoned wood and display very little shrinkage after they are fabricated. Timber beams have practical span lengths that can range up to 40 ft and glue-laminated beams have practical span lengths that may range from 80 ft to 150 ft.

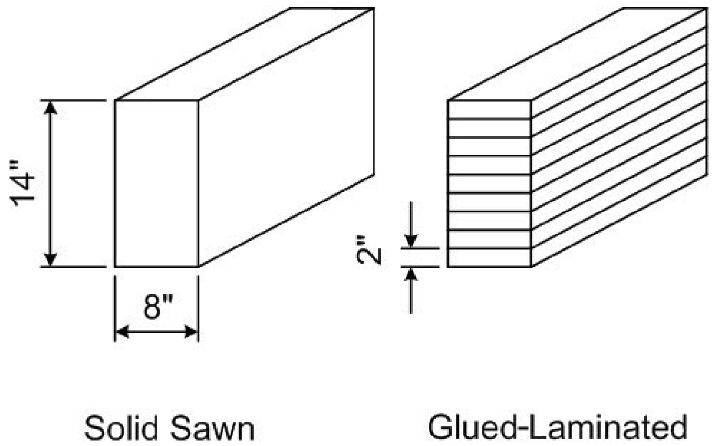


Figure 4.1.17 Timber Beams

Piles/Columns

Timber can also be used for piles or columns. Piles are typically round, (see Figure 4.1.16) slender columns that support the substructure footing or partially form the substructure. Piles may be partially above ground or completely buried.

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 enabled 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 1800s, 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.

4.1.5 Connections

Rolled and built-up steel shapes are typically used to make stringers, floorbeams, girders, trusses, frames, arches, and other bridge members. These members necessitate structural joints, or connections, to transfer loads between members. There are several different types of bridge member connections:

- Pin.
- Riveted.
- Bolted.
- Welded.
- Pin and hanger assemblies.
- Spliced.

Pin Connections

Pins are cylindrical bars produced by forging, casting, or cold-rolling. The pin sizes and configurations are as follows (see Figure 4.1.18):

- 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 members. Pin connections are commonly used in eyebar trusses, hinged arches, pin and hanger assemblies, and bearing supports (see Figure 4.1.19).

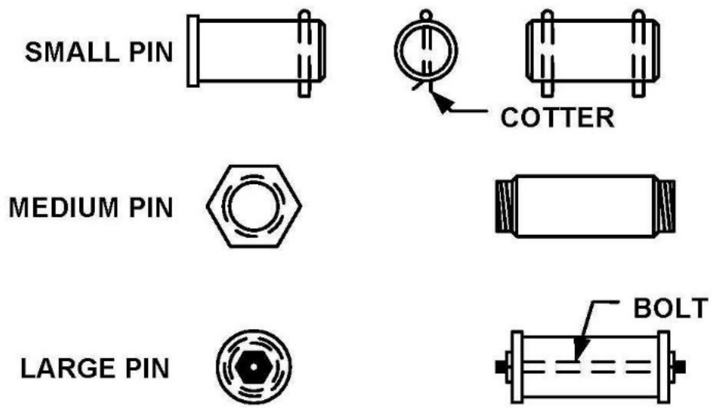


Figure 4.1.18 Sizes of Bridge Pins

Figure 4.1.19 Pin-Connected Eyebars

The major advantages of using pin connection details are the design simplicity and to facilitate rotation. The design simplicity provided by pin connections reduces the amount and complexity of design calculations. By providing 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 inability to properly inspect. Vibrations increase with pin connections because they enable 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 seizing or "freezing" due to corrosion. This results in changes in structural behavior and undesirable stresses when axially loaded members begin to 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 necessary (e.g., ultrasonic testing (UT) or pin removal). Refer to Chapter 17 for more information on UT and other forms of nondestructive evaluation.

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 1960s.

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 4.1.20).

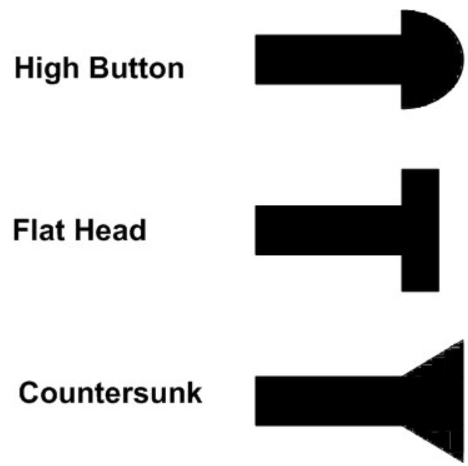


Figure 4.1.20 Types of Rivet Heads

There are two grades of rivets typically found on bridges:

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

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

An inspector can get a general indication whether a rivet is loose by checking for vibration in the rivet in question when an adjacent rivet head is struck with a hammer.. 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 1950s may have a combination of riveted (shop) and bolted (field) connections (see Figure 4.1.21).

Structural bolts consist of these basic material ASTM designations:

- F3125 Grade A325.
- F3125 Grade F1852.
- F3125 Grade A490.
- F3125 Grade F2280.

Figure 4.1.21 Shop Rivets and Field Bolts

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

Tightening high strength bolts puts them in tension, which clamps the member. 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.

For further information on the rivets and bolts listed above or any other material properties visit the American Society for Testing and Materials International website at: www.astm.org or the Research Council for Structural Bolting (RCSB) website at: www.boltcouncil.org.

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 in union, is as strong as the individual materials themselves.

Similar to mechanical fasteners, welds are generally used to make structural connections between members and also to connect members of a built-up member. Welds have also been used in the fabrication and erection of bridges as a way to temporarily combine pieces before field riveting, bolting, or welding. Small temporary erection welds, known as tack welds, can cause serious fatigue problems to certain bridge members (see Figure 4.1.22). Fatigue and fracture of steel bridge members are presented in detail in Chapter 7. Welding can also be used as a means of sealing joints and seams from moisture.

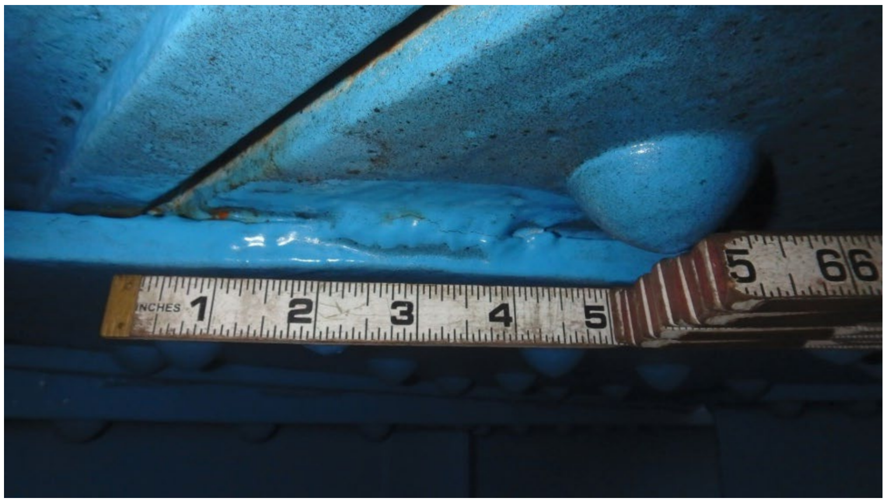


Figure 4.1.22 Tack Weld on a Riveted Built-up Truss Member

The first specification for using welds on bridges appeared in 1936 (FHWA-HRT-04-124). 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 in bridge design attributed to welding technology.

Welds should 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 members.

Pin and Hanger Assemblies

A pin and hanger assembly is a type of hinge consisting of two pins and two hangers (see Figure 4.1.24).

Figure 4.1.23 Pin and Hanger Assembly

Pin and hanger assemblies are typically 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 situated on both sides of the web. 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 simplify design calculations before computer programs were developed to aid design of continuous cantilevered/suspended bridges. Cantilevered/suspended beam seats serve the same purpose as pin and hanger assemblies (see Figure 4.1.24).

Pin and hanger assemblies and the bearings on the beam seat details should be carefully inspected for signs of wear and corrosion. A potential problem can occur if corrosion of the pin and hanger or bearing causes the assembly to "freeze," inhibiting free rotation. This condition does not enable the pin or bearing to rotate and results in additional stresses in the pin and hanger or bearing and adjacent members. The failure of a pin and hanger or beam seat assembly may cause a partial or complete failure of the bridge.

Figure 4.1.24 Cantilevered/Suspended Beam Seat

Splice Connections

A splice connection is the joining of two sections of the same member, 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 since lengths may be limited due to shipping (see Figure 4.1.25). Shop welded flange splices are common in large welded plate girders and long truss members.

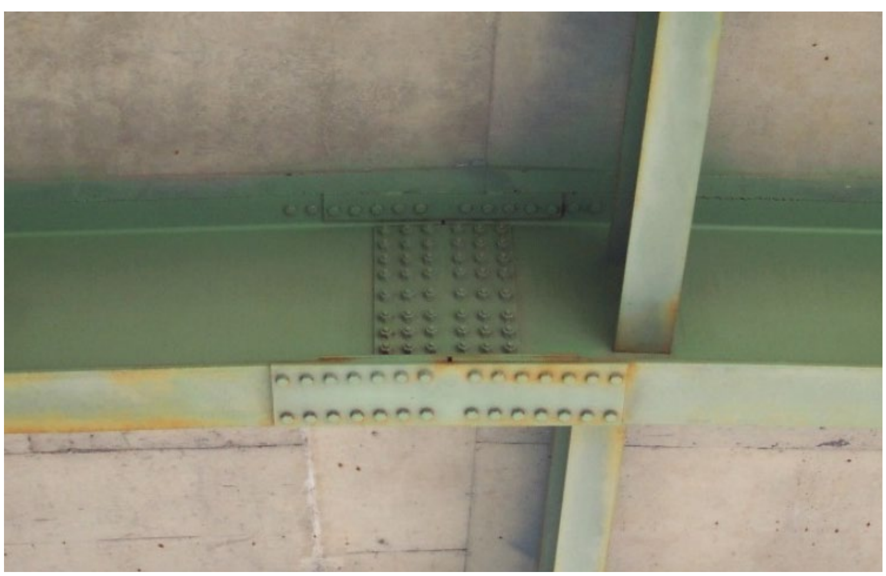


Figure 4.1.25 Bolted Field Splice

4.1.6 Roadside Hardware

The proper and effective use of roadside hardware aims to redirect errant vehicles and can minimize hazards for traffic on the bridge, on the highways, and waterways beneath the bridge.

Roadside hardware includes:

- Bridge railing - located on the bridge or crossing buried structures to guide, contain, and redirect errant vehicles.
- Bridge railing transitions - the transition from the bridge railing to the approach guardrail.

Figure 4.1.25 and Figure 4.1.25 provide examples of the bridge railing and bridge railing transition for typical structures.

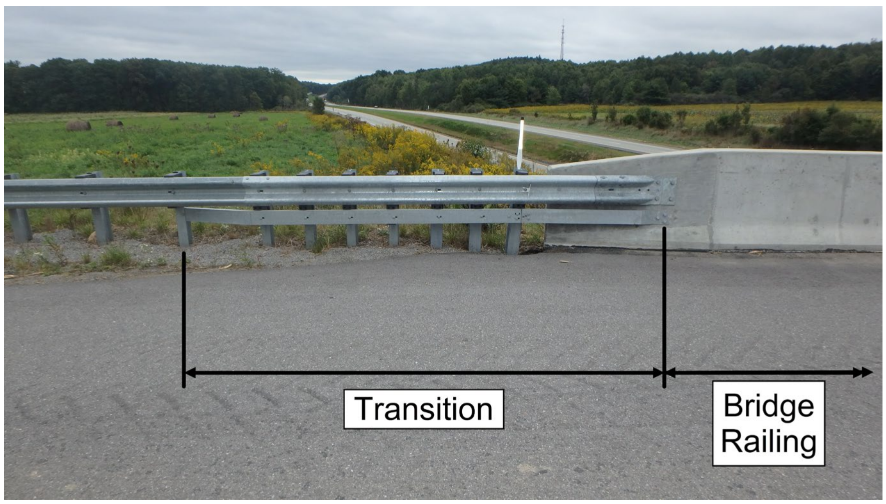


Figure 4.1.26 Roadside Hardware on a Bridge

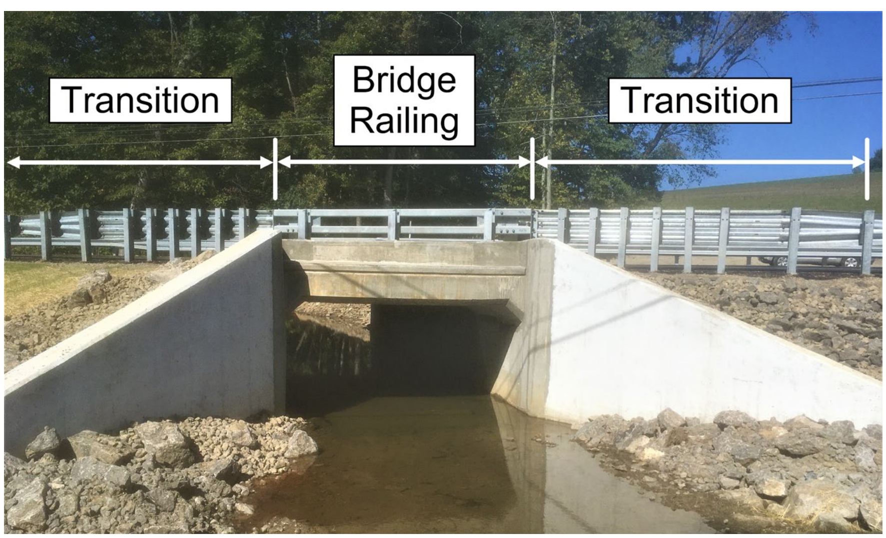


Figure 4.1.27 Roadside Hardware on a Culvert

Bridge railing and transitions may be made of several materials, including:

- Metal bridge railing.
- Reinforced concrete bridge railing (see Figure 4.1.25).
- Timber bridge railing.
- Masonry bridge railing.

Refer to Chapter 8 for detailed explanation on the inspection and evaluation of roadside hardware.

Figure 4.1.28 New Jersey Barrier

4.1.7 Decks

The deck is the member of a bridge to which the live load is directly applied. Refer to Chapter 8 for a detailed explanation on the inspection and evaluation of decks.

Deck Purpose

The deck is designed to transfer live loads and dead loads from the deck to the bridge component referred to as the superstructure (see Figure 4.1.29). 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 4.1.29 Underside View of a Bridge Deck

The deck is also designed to provide a smooth and safe riding surface for the traffic utilizing the bridge (see Figure 4.1.30).

Figure 4.1.30 Bridge Deck

Deck Types

Decks function in one of two ways:

- Composite decks - join with their supporting members and increase superstructure capacity (see Figure 4.1.31 and Figure 4.1.32).
- 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.

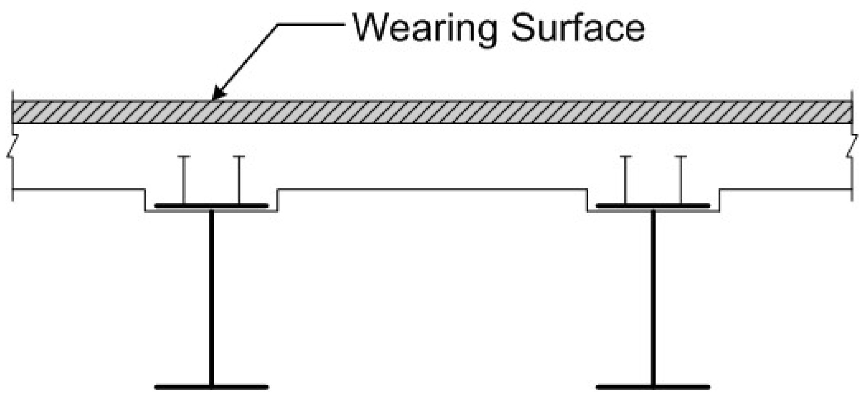


Figure 4.1.31 Composite Concrete Deck and Steel Beams with Shear Studs

Figure 4.1.32 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:

- Concrete.
- Steel.
- Timber.

Aluminum and Fiber Reinforced Polymer (FRP) have been used but are not as common.

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 with reinforcement to resist tensile stresses. Refer to Chapter 8 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.
- Precast conventionally reinforced.
- Precast prestressed – pre-tensioned or post-tensioned.
- Precast prestressed deck panels with cast-in-place topping.

Steel Decks

Steel decks are decks composed of solid steel plate or steel grids (see Figure 4.1.33). Refer to Chapter 8 for a detailed explanation on the inspection and evaluation of steel decks.

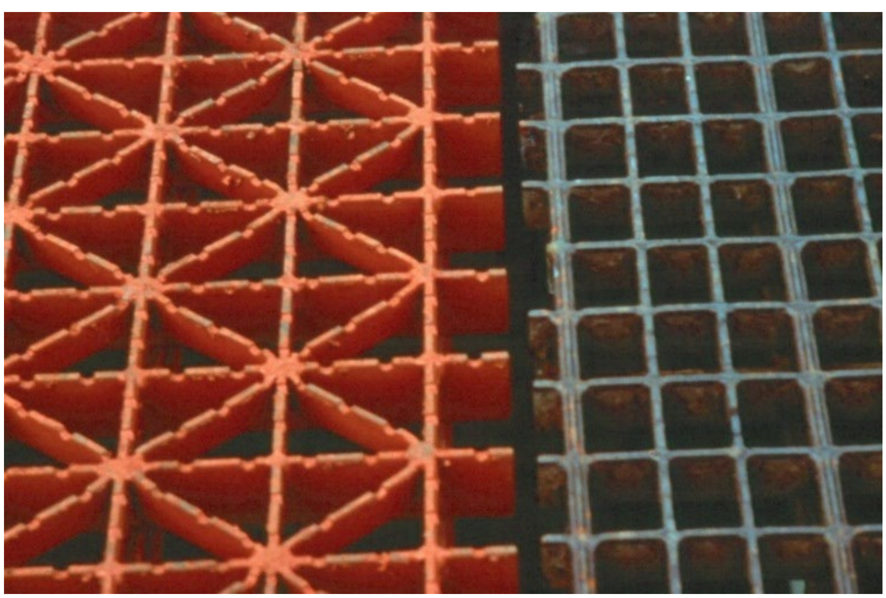


Figure 4.1.33 Steel Grid Deck

There are several types of steel decks:

- Open Grid.
- Concrete Filled Grid.
- Steel Plate.
- Orthotropic.
- Corrugated.

Timber Decks

Timber decks are often referred to as decking or timber flooring, and the term is limited to the roadway portion that receives vehicular loads. Refer to Chapter 8 for a detailed explanation on the inspection and evaluation of timber decks.

Four basic types of timber decks are:

- Plank or solid sawn (see Figure 4.1.34).
- Nail laminated.
- Glue laminated planks.
- Stress laminated.

Figure 4.1.34 Plank Deck

Fiber Reinforced Polymer (FRP) Decks

Fiber-reinforced polymer (FRP) bridge decking has been used to replace some existing highway bridge decks. Though FRP material is typically more expensive than conventional bridge materials such as concrete, it has several advantages. These include lighter weight for efficient transport, improved resistance to earthquakes, and easier installation. FRP bridge decking is also generally less affected by water and de-icing salts, which corrode steel and deteriorate concrete (see Figure 4.1.35). Refer to Chapter 8 for further information on the inspection and evaluation of FRP decks.

Figure 4.1.35 Fiber-Reinforced Polymer (FRP) Deck

The variation of FRP composite decks include:

- Aramid fiber.
- Carbon fiber.
- Glass fiber.

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:

- Bituminous.
- Concrete.
- Gravel.
- Polymers.
- Timber planks - running boards.

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, Ultra-High Performance Concrete (UHPC).
- Asphalt/Bituminous (see Figure 4.1.36).
- Polymers - epoxy, polyester, methyl methacrylate.

Figure 4.1.36 Asphalt Wearing Surface on a Concrete Deck

Steel decks may have wearing or riding surfaces that consist of:

- Serrated steel.
- Concrete.
- Asphalt or Bituminous.
- Polymers.

In abnormal cases, earth (i.e., gravel or soil) may be used as the bridge wearing surface.

SNBI Item B.SP.10, Wearing Surface reports the wearing surface material type.

Deck Appurtenances

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 Chapter 8 for detailed explanation on the inspection and evaluation of drainage systems.

A deck drainage system typically has the following members:

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

Deck grade and cross slope are important factors used to facilitate water removal from a bridge deck.

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 members forms a complete deck drainage system.

Substructure drainage enables the fill material behind an abutment or wingwall to drain any accumulated water. Substructure drainage is generally accomplished with weep holes, geotextile, or substructure drainpipes.

Sidewalks and Curbs

The function of sidewalks and curbs is to provide access to and maintain safety for pedestrians, inspectors, and maintenance personnel and to direct water to the drainage system. Curbs serve to lessen the chance of vehicles crossing onto the sidewalk and endangering pedestrians. Older standards include safety walks which are narrow access areas for inspectors and are typically only 12 inches wide. Wheels jumping the curb on these safety walks can lunge quickly in a horizontal direction and can cause a safety issue for motorists.

Signing

Signing provides different types of information to the motorist including bridge or roadway conditions that may be considered hazardous. Refer to Chapter 8 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 4.1.37).
- Speed limit.
- Vertical clearance.
- Lateral clearance.
- Narrow underpass.
- Informational and directional.
- Dynamic message signs.
- Object markers (see Figure 4.1.37).

Figure 4.1.37 Weight Limit Sign and Object Marker Sign

Lighting and Signals

Types of lighting and signals that may be encountered on a bridge include the following (see Figure 4.1.38):

- Highway lighting.
- Aesthetic lighting
- Traffic signals.
- Lane assignment indications.
- Aerial obstruction lights.
- Navigation lights.
- Signing lights.
- Illumination and drawbridge operation flashing lights.

Figure 4.1.38 Bridge with Multiple Types of Lighting

Refer to Chapter 8 for detailed explanation on the inspection and evaluation of lighting or signal systems.

4.1.8 Deck Joints

The primary function of a deck joint is to accommodate the expansion, contraction, and rotation of the superstructure. The joint should also provide a smooth transition from an approach roadway to a bridge deck, or between adjoining segments of bridge deck. Refer to Chapter 8 for detailed explanation on the inspection and evaluation of deck joints.

Examples of deck joints may include:

- Strip seal expansion joints (see Figure 4.1.25).
- Pourable joint seals.
- Compression joint seals (see Figure 4.1.25).
- Assembly joints with seal (modular, finger plate and sliding plate joints).
- Open expansion joints.
- Assembly joints without seals (finger plate and sliding plate joints) (see Figure 4.1.25).
- Other joints.

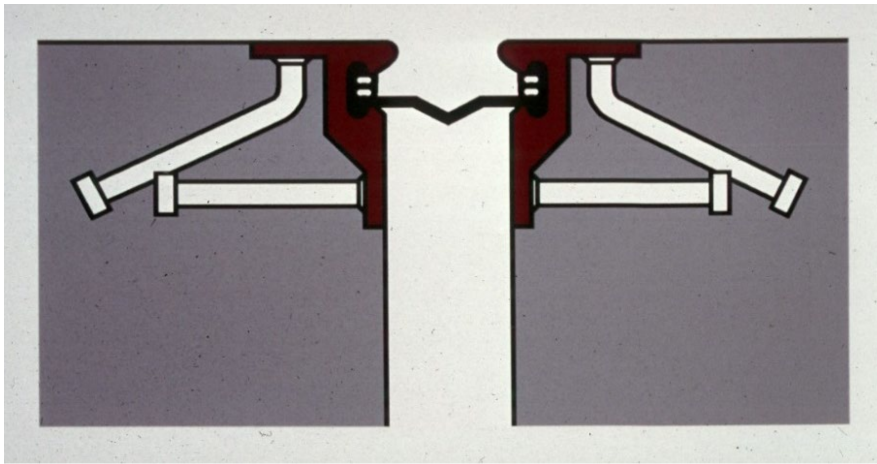


Figure 4.1.39 Strip Seal Expansion Joint

Figure 4.1.40 Top View of an Armored Compression Seal in Place

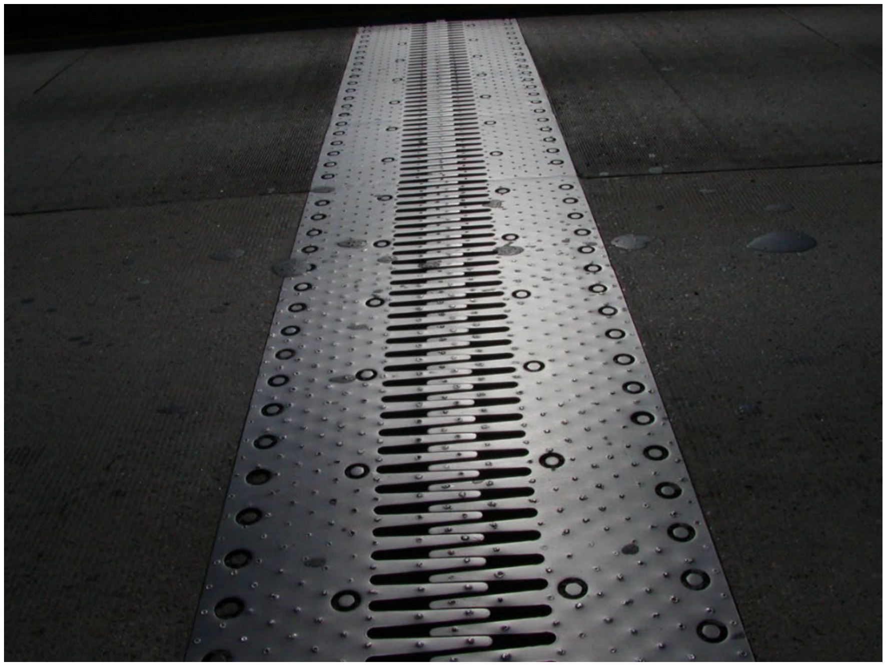


Figure 4.1.41 Top View of a Finger Plate Joint

4.1.9 Superstructures

Superstructure Purpose

The superstructure is designed 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 (see Figure 4.1.42) such as:

- Slabs.
- Multi-beams/girders.
- Girder-floorbeam-stringer systems.
- Trusses.
- Arches.
- Rigid frames.
- Cable-supported bridges.
- Movable bridges.
- Floating bridges.

The superstructure types are described in more detail in Chapters 9-12.

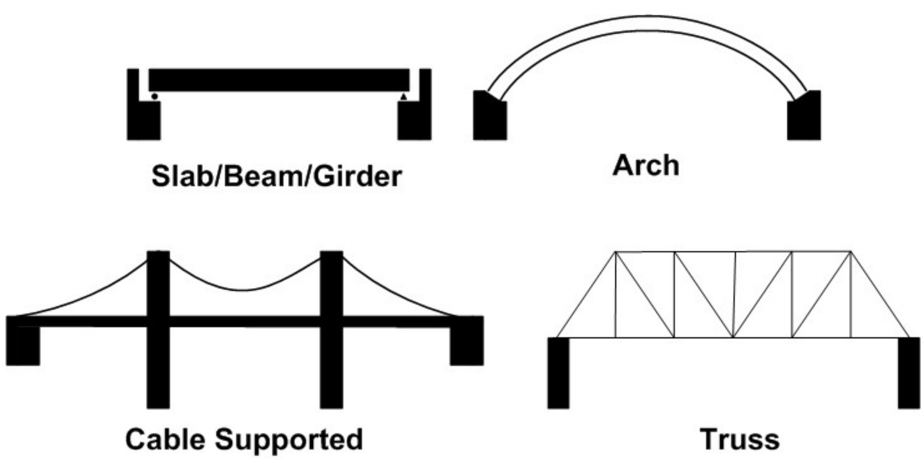


Figure 4.1.42 Basic Bridge Types

Slab Bridges

In slab bridges, loads from the slab are transmitted vertically to the substructure (see Figure 4.1.43).

Figure 4.1.43 Slab Bridge

Multi-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 may include:

- Beams (timber, concrete, or steel) (see Figure 4.1.44, Figure 4.1.45, Figure 4.1.46, Figure 4.1.47).
- Girders (concrete or steel) (see Figure 4.1.48, Figure 4.1.49).

Figure 4.1.44 Multi-Beam Timber Bridge

Figure 4.1.45 Multi-Beam Prestressed Concrete Bridge

Figure 4.1.46 Concrete Tee Beam Bridge

Figure 4.1.47 Adjacent Box Beam Bridge

Figure 4.1.48 Steel Curved Girder Bridge

Figure 4.1.49 Steel Box Girder Bridge

Girder-Floorbeam Systems

In the 1960s, the steel industry came up with a new design for steel bridges to be competitive in cost with prestressed concrete bridges. The new steel bridge utilized a floor system consisting of transverse floorbeams and possibly longitudinal stringers to support the deck. The girders support the floor system (see Figure 4.1.50). In most cases the girders are below the deck, but steel through girder bridges were constructed in the past, with the girders extending above the deck.

Figure 4.1.50 Girder-Floorbeam 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 Figure 4.1.51 and Figure 4.1.52).

Figure 4.1.51 Deck Truss Bridge

Figure 4.1.52 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 Figure 4.1.53 and Figure 4.1.54). Several types of arch span types are used depending on the bridge site, including open and closed spandrel arches, through and tied arch bridges, and arches under fill.

Figure 4.1.53 Deck Arch Bridge

Figure 4.1.54 Through Arch Bridge

Rigid Frames

Rigid frame superstructures are characterized by rigid (moment) connections between the horizontal girder and the legs. This connection provides the transfer of both axial forces and moments into vertical or sloping members, which may be classified as superstructure or substructure members 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. Steel rigid frames may be either K-shaped (see Figure 4.1.55) or delta-shaped, which uses a triangular configuration.

Figure 4.1.55 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. Typical cable-supported bridges are suspension or cable-stayed (see Figure 4.1.56 and Figure 4.1.57). Extradosed bridges are less common. Refer to Chapter 5 for a more detailed explanation on cable-supported bridges.

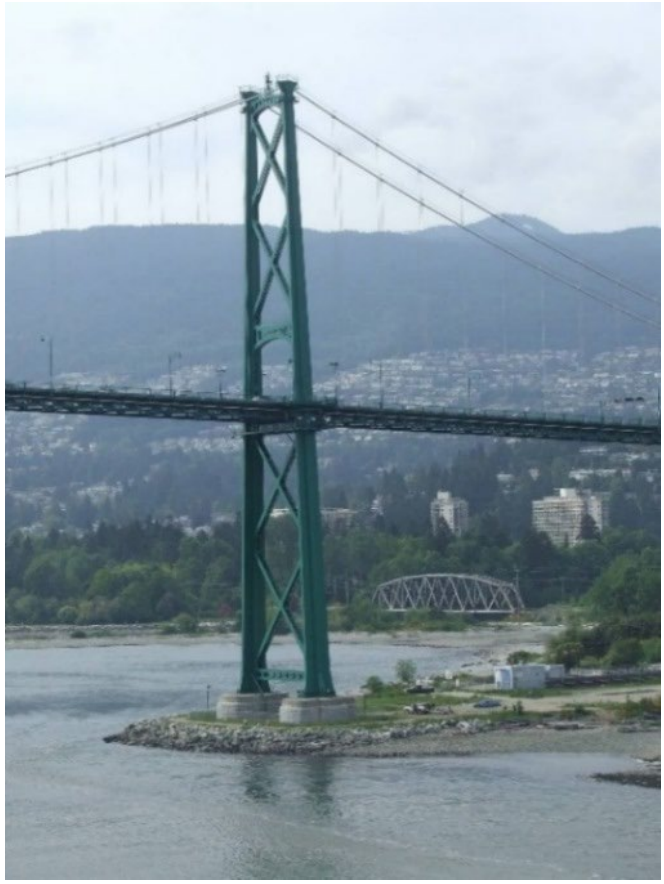


Figure 4.1.56 Suspension Bridge and Tower

Figure 4.1.57 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. A movable bridge is designed to provide the appropriate channel width and under clearance for passing water vessels when fully opened. Refer to Chapter 5 for a more detailed explanation on movable bridges.

Movable bridges can be classified into three general groups:

- Bascule (see Figure 4.1.58).
- Swing (see Figure 4.1.59).
- Lift (see Figure 4.1.60).

Figure 4.1.58 Bascule Bridge

Figure 4.1.59 Swing Bridge

Figure 4.1.60 Lift Bridge

Floating Bridges

Although uncommon, some states have bridges that are not supported by a substructure (see Figure 4.1.61). Instead, they are supported by water. The elevation of the bridge will likely change as the water level fluctuates.

Figure 4.1.61 Floating Bridge

Superstructure Materials

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

- Concrete.
- Steel.
- Timber.

Other materials that are less commonly used include:

- Aluminum.
- FRP.
- Iron.
- Masonry.
- Plastic.

Primary Members

Typical primary superstructure members that carry primary live load from vehicles typically consist of the following:

- Slabs.
- Girders (see Figure 4.1.62).
- Floorbeams (see Figure 4.1.62).
- Stringers (see Figure 4.1.62).
- Trusses.
- Spandrel girders.
- Arch ribs.
- Rib chord bracing.
- Spandrel columns or bents (see Figure 4.1.63).
- Frame girder.
- Frame leg and knee.
- Pin and hanger assemblies – pins and hanger plates.
- Cantilevered/suspended span beam seats.

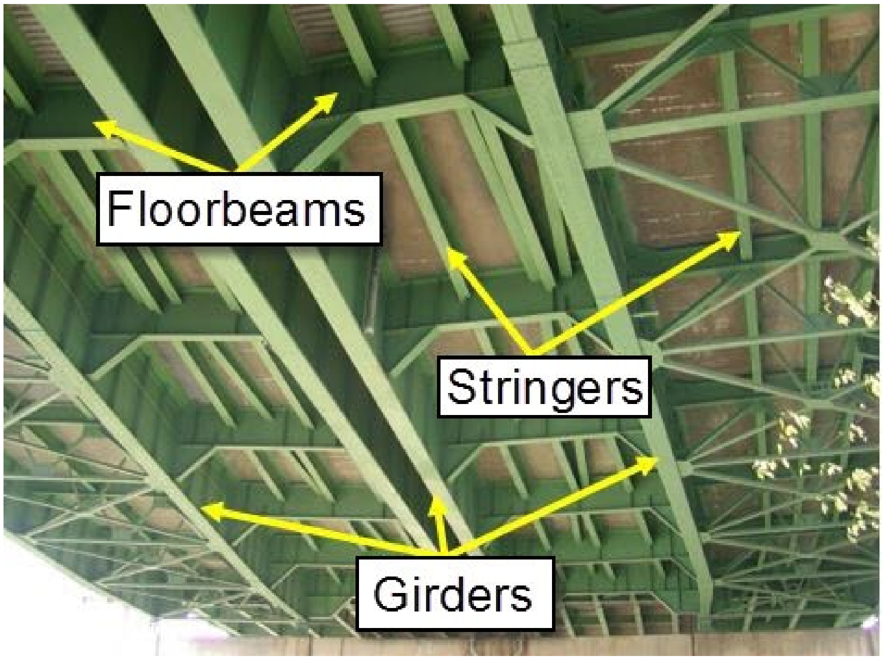


Figure 4.1.62 Floor System and Main Supporting Members

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 4.1.63 Deck Arch with Spandrel Columns

Secondary Members

Secondary members do not typically carry traffic loads directly. Typical secondary members can include:

- Diaphragms (see Figure 4.1.64).
- Cross or X-bracing (see Figure 4.1.65).
- Lateral bracing (see Figure 4.1.66).
- Sway-portal bracing (see Figure 4.1.66).
- Pin and hanger assembly parts - through bolts, pin caps, nuts, cotter pins on small assemblies, spacer washers, doubler plates.

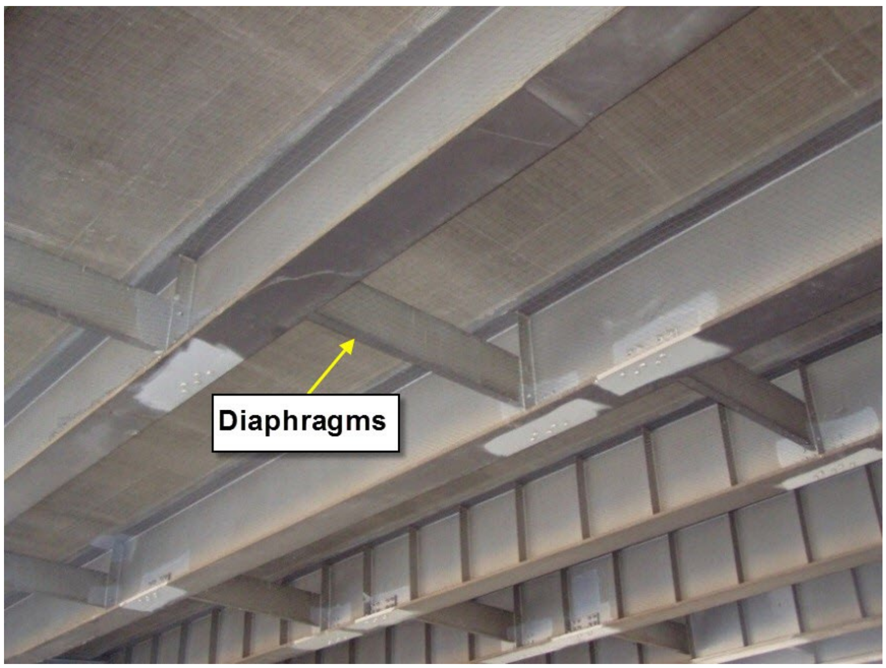


Figure 4.1.64 Diaphragms

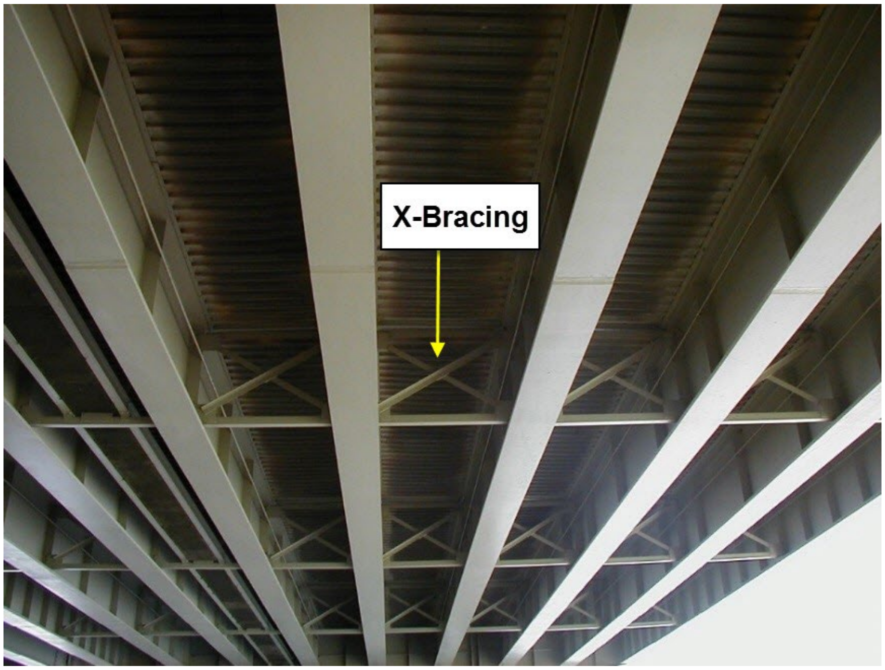


Figure 4.1.65 Cross or X-Bracing

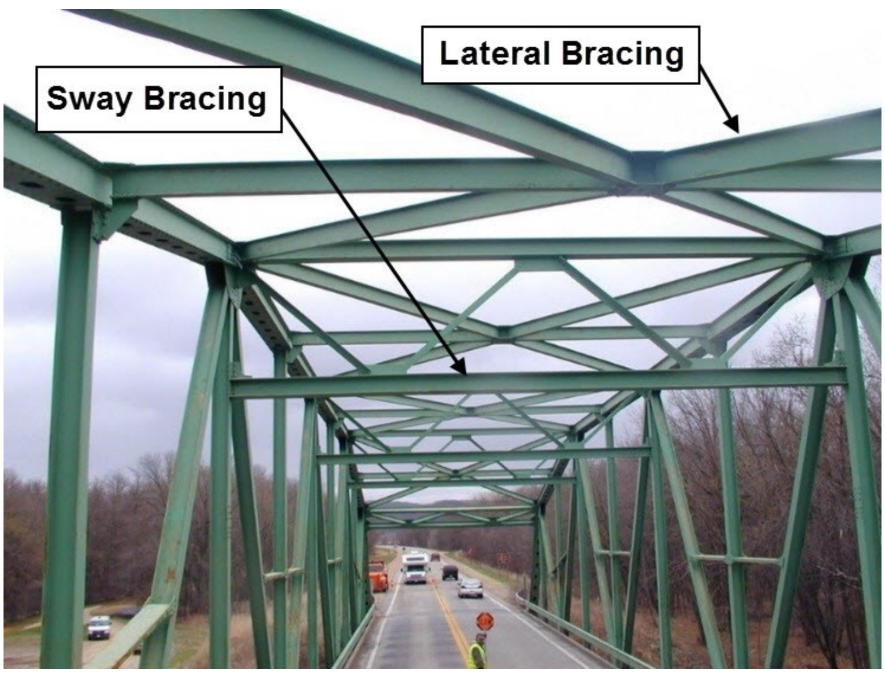


Figure 4.1.66 Top Lateral Bracing and Sway Bracing

4.1.10 Bearings

Purpose of Bearings

A bridge bearing is an element that 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.
- Enable rotation caused by dead and live load deflection.

Bearings that do not provide for horizontal movement of the superstructure are referred to as fixed bearings. Bearings that provide for horizontal movement of the superstructure are known as expansion bearings. Both fixed and expansion bearings should permit rotation. Refer to Chapter 13 for more detailed explanation on expansion/fixed bearings.

Bearing Types

Common bearing types that are utilized to accommodate superstructure support may include:

- Elastomeric.
- Movable (roller, sliding, etc.) (See Figure 4.1.67).
- Enclosed/concealed.
- Fixed.
- Pot.
- Disk.

Bearing Materials

Bridge bearings are usually constructed of steel or elastomeric (rubber type) materials. A bridge bearing can be typically categorized into four basic parts (see Figure 4.1.67):

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

Refer to Chapter 13 for detailed explanations of these four bearing parts.

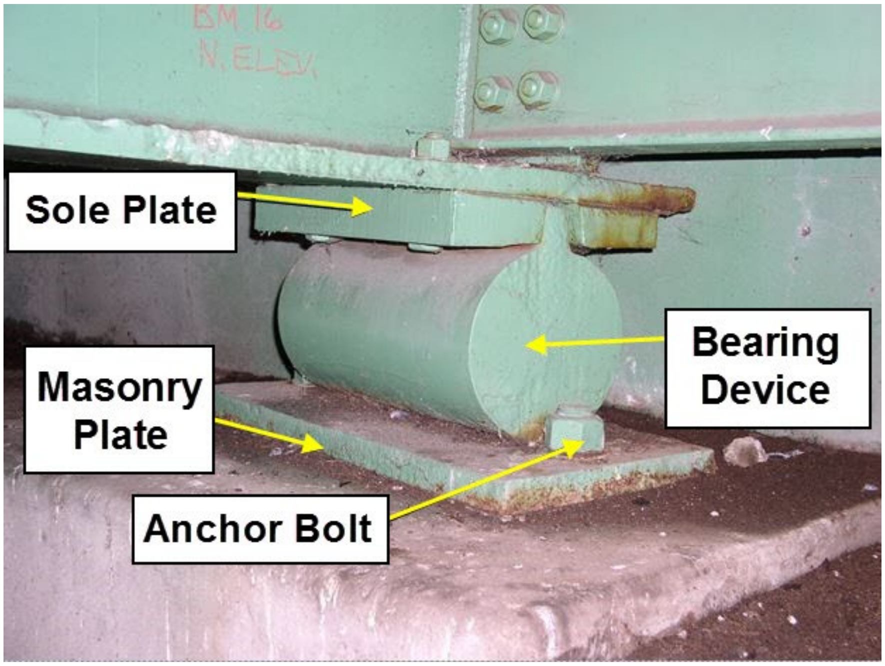


Figure 4.1.67 Steel Roller Bearing Showing Four Basic Parts

4.1.11 Substructures

The substructure is the member of the bridge that includes all the members that support the superstructure.

Substructure Purposes

The substructure is designed to transfer the loads from the superstructure to the foundation soil or rock. Typically, the substructure includes all members below the bearings.

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 4.1.68). Piers and bents provide support for the superstructure at intermediate points along the bridge spans (see Figure 4.1.69).

Figure 4.1.68 Abutment

Figure 4.1.69 Pier

Substructure Types

Abutments

Basic types of abutments can 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 4.1.70 and Figure 4.1.75).
- Stub, semi-stub, or shelf abutment - found 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 4.1.71 and Figure 4.1.76).
- 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 4.1.72 and Figure 4.1.77).
- 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 enables 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 4.1.73, Figure 4.1.74, and Figure 4.1.78).

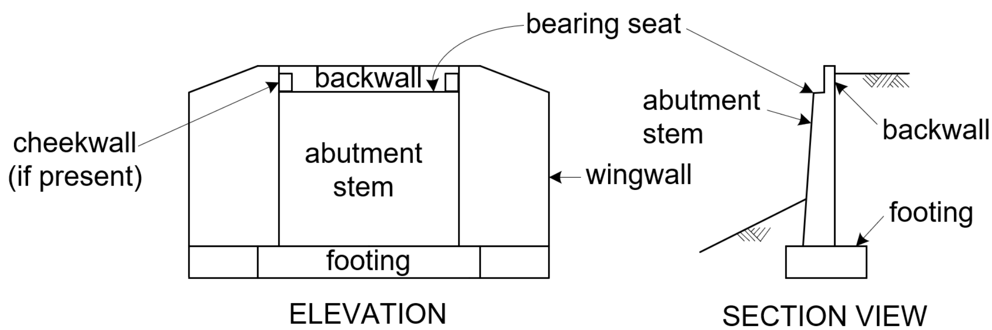


Figure 4.1.70 Sketch of Full Height Abutment

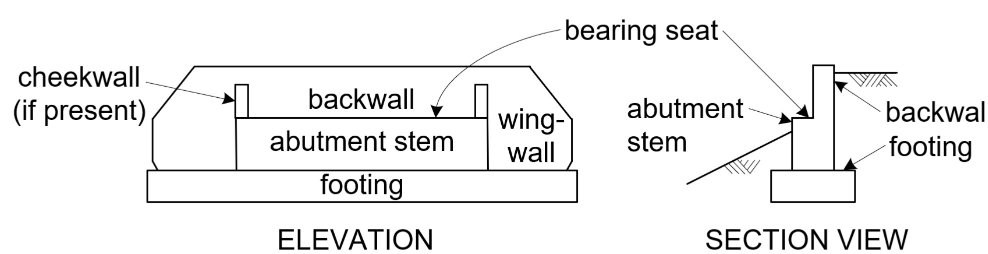


Figure 4.1.71 Sketch of Stub Abutment

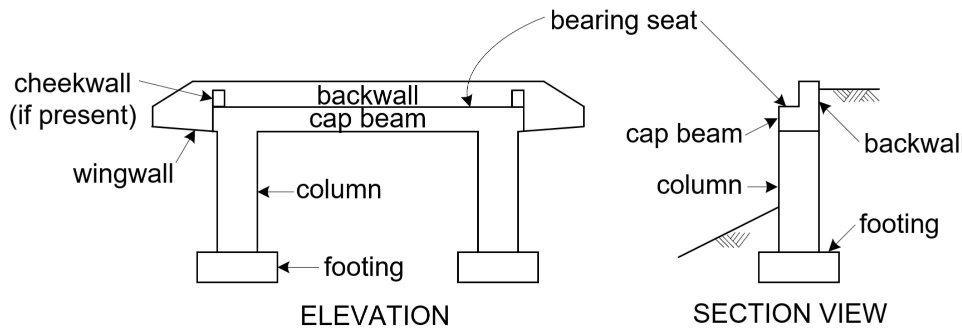


Figure 4.1.72 Sketch of Spill Through/Open Abutment

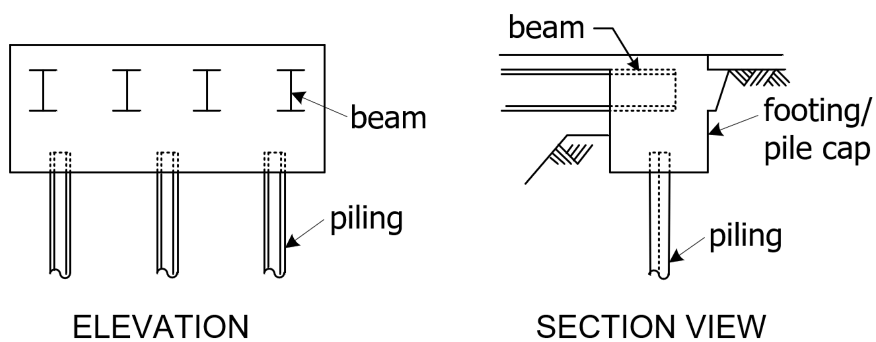


Figure 4.1.73 Sketch of Integral Abutment

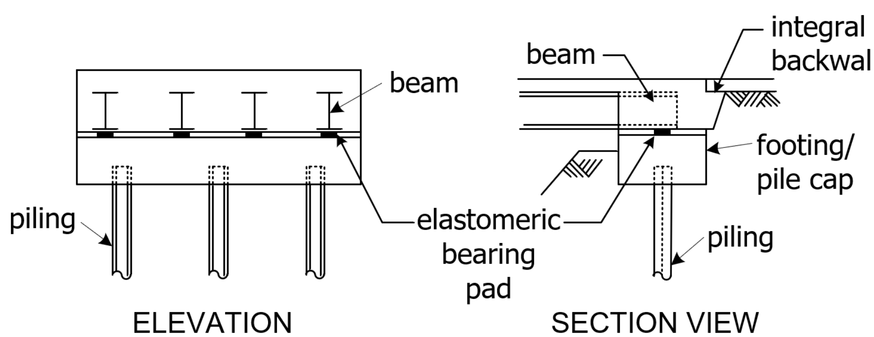


Figure 4.1.74 Sketch of Semi-Integral Abutment

Figure 4.1.75 Cantilever Abutment (or Full Height Abutment)

Figure 4.1.76 Stub Abutment

Figure 4.1.77 Spill Through or Open Abutment

Figure 4.1.78 Integral Abutment

Other less common abutment types include:

- Gravity.
- Counterfort.
- Pile bent with lagging.
- Crib.
- Cellular or vaulted.
- Reinforced soil.

The type of abutment is recorded for bridges using *SNBI* Item B.SB.04, Substructure Type. Refer to Chapter 14 for a more detailed explanation on bridge abutments.

Piers and Bents

A pier or bent is an intermediate substructure unit situated between the abutments. Their function is to support the superstructure at intermediate intervals with minimal obstruction to the flow of traffic or water below the superstructure. A pier has a footing at each column or wall in the substructure unit (the footing may serve as a pile cap). A bent can have several footings or no footing, as is the case with a pile bent. Refer to Chapter 14 for a more detailed explanation on bridge piers and bents.

There are three basic types of piers:

- Column (see Figure 4.1.79 and Figure 4.1.80).
- Tower Pier (Trestle) (see Figure 4.1.81).
- Pier Wall (see Figure 4.1.80 and Figure 4.1.83).

Column piers may be a singular column or multiple columns. These piers may have a web wall connecting columns as well. Some other, less common pier types include straddle or c-shaped piers and movable bridge piers.

Figure 4.1.79 Column Pier

Figure 4.1.80 Column Pier

Figure 4.1.81 Tower (Trestle) Pier

Figure 4.1.82 Column Pier with Web Wall and Cantilevered Pier Caps

Figure 4.1.83 Pier Walls with Cantilever or Hammerhead Caps

There are two basic types of bents supported by individual footings and supported by piles:

- Column bent - separate footing under supporting column, open or with a web wall (see Figure 4.1.84).
- Pile bent-pile - extended in the ground (see Figure 4.1.85).

Straddle or c-shaped are also bent types, but are less widely used.

Figure 4.1.84 Column Bent

Figure 4.1.85 Steel and Concrete Pile Bents

The type of pier or bent is recorded for bridges using *SNBI* Item B.SB.04, Substructure Type. Refer to Chapter 14 for a more detailed explanation on bridge piers and bents.

Substructure Materials

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

- Concrete.
- Steel.
- Masonry.
- Timber.

Other materials that are less commonly used include:

- Aluminum.
- Earth
- FRP.
- Iron.
- Plastic.

SNBI Item B.SB.03 reports Substructure Material for bridge abutments, piers, and bents.

Substructure Members

A bridge substructure can consist of several different members (see Figure 4.1.70 through Figure 4.1.74).

Typical members associated with abutments may include but are not limited to:

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

Typical members associated with piers and bents may include but are not limited to:

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

Refer to Chapter 14 for a detailed explanation of abutment, pier, and bent members.

Section 4.2 Basic Culvert Characteristics

4.2.1 Introduction

A culvert is a structure comprised of one or more barrels, beneath an embankment and designed structurally to account for soil-structure interaction. These structures are hydraulically and structurally designed to convey water, sediment, debris, and, in many cases, aquatic and terrestrial organisms through roadway embankments. Culvert barrels have many sizes and shapes and have inverts that are either integral or open, i.e., supported by spread or pile-supported footings. Many culverts take advantage of headwater submergence of the inlet to increase hydraulic efficiency and economy (*SNBI* Definitions).

If a structure resembling a culvert does not convey water, it should not be classified as a culvert. Similarly, if the structure's primary purpose is to separate traffic, it should be classified as a bridge or tunnel.

Culverts may qualify to be "bridge" length, and therefore be subject to the NBIS. Over the years, culverts have traditionally received less observation than bridges. Since culverts are typically 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.

In many cases, single or multiple barrel culverts, or long span culverts were built instead of bridges (see Figure 4.2.1). There have also been recent advances in culvert design and analysis techniques. Long span corrugated metal culverts with spans in excess of 40 ft were introduced in the late 1960s.

Figure 4.2.1 Culvert Structure

The failure of a culvert may be more than a mere driving inconvenience. Failure of a large culvert may be both costly and hazardous. Many of the metal culverts constructed years ago are needing replacement due to the corrosive environments in which they were placed.

Bridge-size culverts are inspected regularly, like all NBIS bridges, to identify potential safety problems and maintenance needs. Culverts not meeting NBIS bridge criteria may or may not be inspected, depending on the owner. 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.

4.2.2 Differentiation Between Culverts and Bridges

Culverts meeting the NBIS criteria for a bridge below are subject to NBIS requirements (CFR 650.313):

> "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 ft between under copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it includes multiple pipes, where the clear distance between openings is less than half of the smaller contiguous opening." (CFR 650.305)

The structural and hydraulic design of culverts is substantially different from bridges, as are construction methods, maintenance needs, and inspection procedures. A few of the more significant differences between bridges and culverts are hydraulic or structural.

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 generally 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. 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 transient 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.

Figure 4.2.2 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.

4.2.3 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. Similar to bridges, there are two general types of loads that are carried by culverts: permanent loads and transient loads.

Permanent

Permanent loads on the culvert 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

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 effect of live loads decreases as the height of cover over the culvert increases. Transient loads on culverts may be ignored if the depth of the cover is greater than 8 ft.

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 necessary stiffness to resist loads. In doing this, zones of bending and shear are created. The culvert material is designed to resist the corresponding stresses.

Rigid Culverts are presented in detail in Chapter 15.

Flexible Culverts

Flexible culverts are commonly made from steel or aluminum. In some states, composite materials or plastics may be 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 likely deflect if the surrounding fill material is loose. The vertical diameter decreases and the horizontal diameter increases. Soil pressures resist the increase in horizontal diameter.

For flexible culverts with large openings, sometimes longitudinal and/or circumferential stiffeners may be 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 Chapter 15.

4.2.4 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 necessary. Other factors such as the potential for clogging by debris, the need for a natural stream bottom, or structural and hydraulic demands may influence the selection of culvert shape. Each of the common culvert shapes are discussed in the following paragraphs.

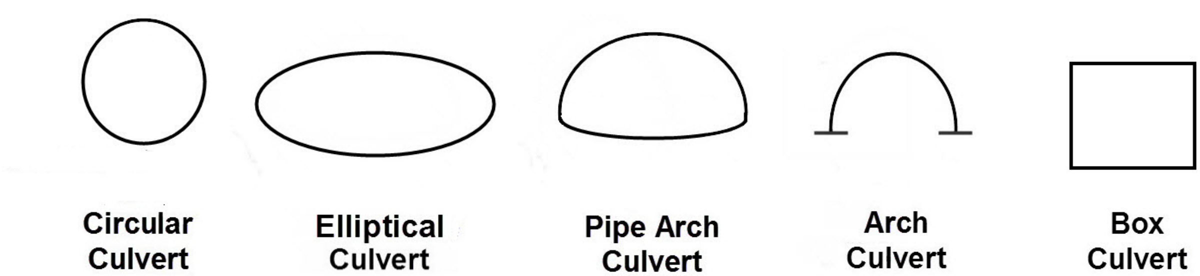


Figure 4.2.3 Typical Culvert Shapes

Circular

The circular shape is the most common shape manufactured for pipe culverts (see Figure 4.2.4). 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 4.2.4 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 4.2.5). 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 4.2.5 Pipe Arch Culvert

SNBI Item B.SP.06 for Span Type differentiates between rigid and flexible pipes. The pipe should be coded as flexible for pipes that rely on stability of the surrounding soil to maintain their shape (*SNBI* Subsection 2.1, Item B.SP.06 Commentary). Any steel or aluminum culvert would therefore be coded as flexible, regardless of shape.

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 4.2.6). Foundation conditions should be adequate to support the footings. Riprap is frequently used for scour protection.

Figure 4.2.6 Precast Frame Culvert

Box Sections

Rectangular cross-section culverts are easily adaptable to a wide range of site conditions including sites that necessitate low profile structures. Due to the flat sides and top, rectangular shapes are not as structurally efficient as other culvert shapes. Box sections have an integral floor. Box culverts are coded as four-sided frames in the *SNBI* Span Type item.

Multiple Barrels

Multiple barrels may be used to obtain adequate hydraulic capacity under low embankments or for wide waterways (see Figure 4.2.7). The span or opening length of multiple barrel culverts includes the clear distance between barrels as long as that distance is less than half the opening length of the adjacent barrels. 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 likely to occur in one or several of the barrels based upon the low flow conditions.

Figure 4.2.7 Multiple Cell Concrete Culvert

Frame Culverts

Frame culverts are constructed of cast-in-place concrete (see Figure 4.2.8) or precast reinforced concrete. This type of culvert has no floor (concrete bottom) and fill material is placed over the structure. The *SNBI* generally refers to this type of structure as a three-sided frame (*SNBI* Subsection 2.1, Span Type).

Figure 4.2.8 Frame Culvert

4.2.5 Culvert Materials

Cast-in-Place Concrete

Culverts that are reinforced cast-in-place concrete are typically 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 demands of a site. Due to the long construction time of cast-in-place culverts, precast concrete or corrugated metal culverts are sometimes selected. However, occasionally due to site conditions, cast-in-place culverts are more practical. Refer to Chapter 15 for more detailed information on cast-in-place concrete culverts.

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, with custom sizes being larger. Precast concrete arches on cast-in-place footings are available with spans up to 41 ft. Refer to Chapter 15 for more detailed information on precast culverts.

Metal

Flexible culverts are typically steel or aluminum and are constructed from factory-made corrugated metal pipe or field assembled from structural plates. Structural plate products are available as plate pipes, box culverts, or long span structures (see Figure 4.2.9 and Figure 4.2.10). Several factors such as span length, vertical and horizontal clearance, peak stream flow and terrain determine which flexible culvert shape is used. Refer to Chapter 15 for more detailed information of metal culverts.

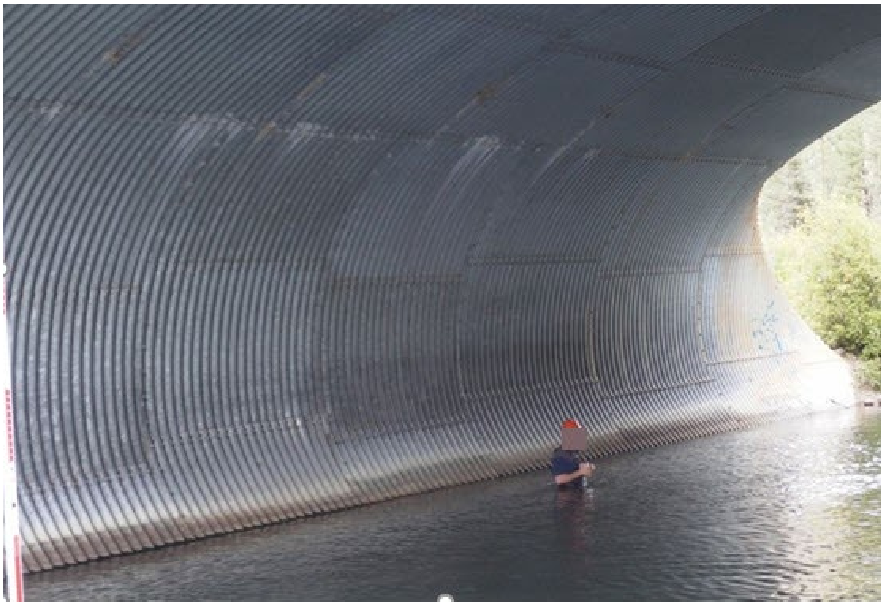


Figure 4.2.9 Large Structural Plate Pipe Arch Culvert

Figure 4.2.10 Large Structural Plate Box Frame Culvert

Masonry

Stone and brick are durable, low maintenance materials. Before the 1920s, 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 that 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 can be a single or multiple barrel configuration as shown in

Figure 4.2.11. Many times, the secondary barrel may be filled when there is a flood event. Refer to Chapter 15 for more detailed information on stone masonry culverts.

Figure 4.2.11 Stone Masonry Arch Culvert

Timber

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

Timber culverts are generally box culverts and are constructed from individual timbers planks or beams. Timber culverts are also analogous to a short span timber bridge on timber abutments (see Figure 4.2.12). Refer to Chapter 15 for more information on timber culverts.

Figure 4.2.12 Timber Box Culvert

Plastic

Currently, plastic culverts may be installed where conditions warrant such as when acidic water is present. They are round in shape, similar to corrugated metal culverts (see Figure 4.2.13). Refer to Chapter 15 for more information on plastic culverts.

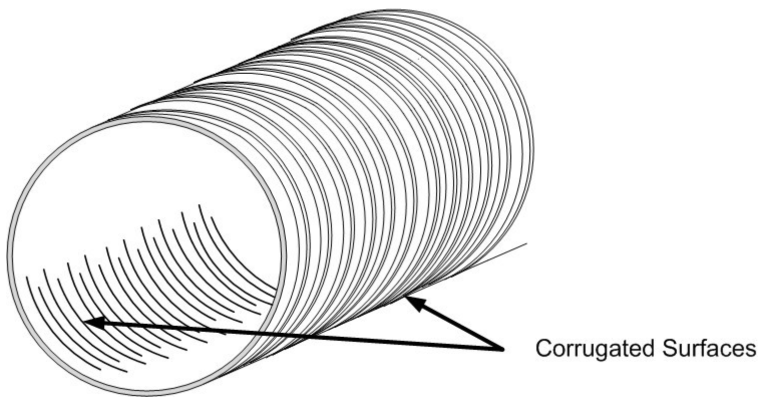


Figure 4.2.13 Schematic of a Single Walled Plastic Culvert

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 labor intensive (terra cotta) or used for specialized situations (stainless steel and cast iron) or have been replaced due to their age and condition.

4.2.6 Culvert End Treatments

Culverts may have end treatments or end structures. End structures are generally 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 include:

- Projecting - The barrel simply extends beyond the embankment. No additional support is used (see Figure 4.2.14).
- 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 4.2.15).
- Skewed - Culverts, that are not perpendicular to the roadway, may have their ends cut parallel to the roadway (see Figure 4.2.16).
- 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 4.2.17) but can be constructed of timber or masonry. Metal headwalls are usually found on metal box culverts.

Figure 4.2.14 Culvert End Projection

Figure 4.2.15 Culvert Mitered End

Figure 4.2.16 Culvert End

Figure 4.2.17 Culvert Head Wall 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 4.2.18). Most aprons include an upstream cutoff wall (also known as a toe wall) to protect against undermining.
- Energy Dissipaters - 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 dissipaters (see Figure 4.2.19).

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. Refer to Chapter 7 for anticipated modes of concrete deterioration and inspection procedures for concrete.

Figure 4.2.18 Apron

Figure 4.2.19 Riprap

4.2.7 Culvert Protective Systems

There are several protective measures that can be taken to increase the durability of culverts. Although protective systems help, each material type has a limited service life in severe corrosive environments. The more commonly used measures include the following options.

Extra Thickness

For some aggressive environments, it may be economical to provide extra material 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 necessary 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.

Section 4.3 Basic Waterway Characteristics

4.3.1 Introduction

Waterways are the most dynamic geomorphic system that engineers have to cope with in the design and maintenance of bridges and culverts. The geomorphic features of the river can change dramatically with time. During major floods, significant changes can occur in a short period of time. Rivers are dynamic and can meander. However, bridges are fixed or static.

There are several ways in which channels can change and thereby jeopardize the stability and safety of bridges and culverts. The channel bed can scour (degrade) so that bed elevations become lower, undermining the foundation of the piers, abutments, and culverts. Deposition of sediment on the channel bed (aggradation) can reduce conveyance capacity through the bridge or hydraulic opening. Flood waters are then forced around the bridge or culvert, 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 4.3.1 and 4.3.2). The other primary way in which bridges or culverts 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 4.3.3). Chapter 16 presents more detailed descriptions of waterway deficiencies.

Of all the bridges in the National Bridge Inventory (NBI), over 80 percent are built over waterways. Bridge inspectors should understand the relationship between the bridge members and waterways. 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.

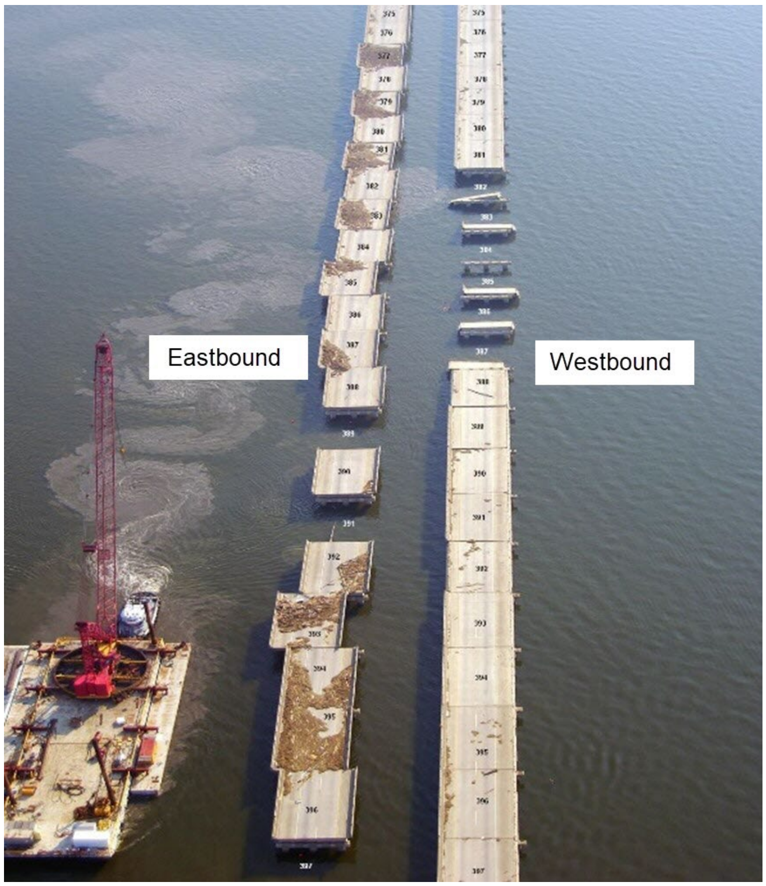


Figure 4.3.1 Failure Due to High Water Levels During Hurricane: Aerial View

Figure 4.3.2 Failure Due to High Water Levels During Hurricane: Close-Up View

Figure 4.3.3 Pier Foundation Failure

4.3.2 Properties Affecting Waterways

Safety to the traveling public 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.

4.3.3 Channel Characteristics

According to the *Hydraulic Design Series Number 6 (HDS-6) River Engineering for Highway Encroachments*, www.fhwa.dot.gov/engineering/hydraulics/pubs/nhi01004.pdf, channels are typically well-defined and confine the streamflow during normal flow conditions (see Figure 4.3.4).

Channel Terminology

- Streambed - the bottom or floor of the channel.
- Streambank - the sloped sides of the channel, that 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.

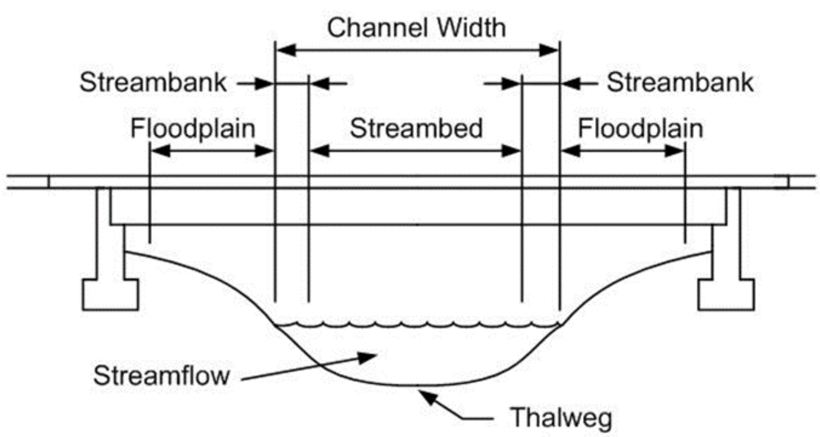


Figure 4.3.4 Typical Waterway Cross Section Showing Well Defined Channel Depression

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 which 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 4.3.5 shows some differences between the various river categories. Figure 4.3.6 illustrates the major characteristics of a meandering river.

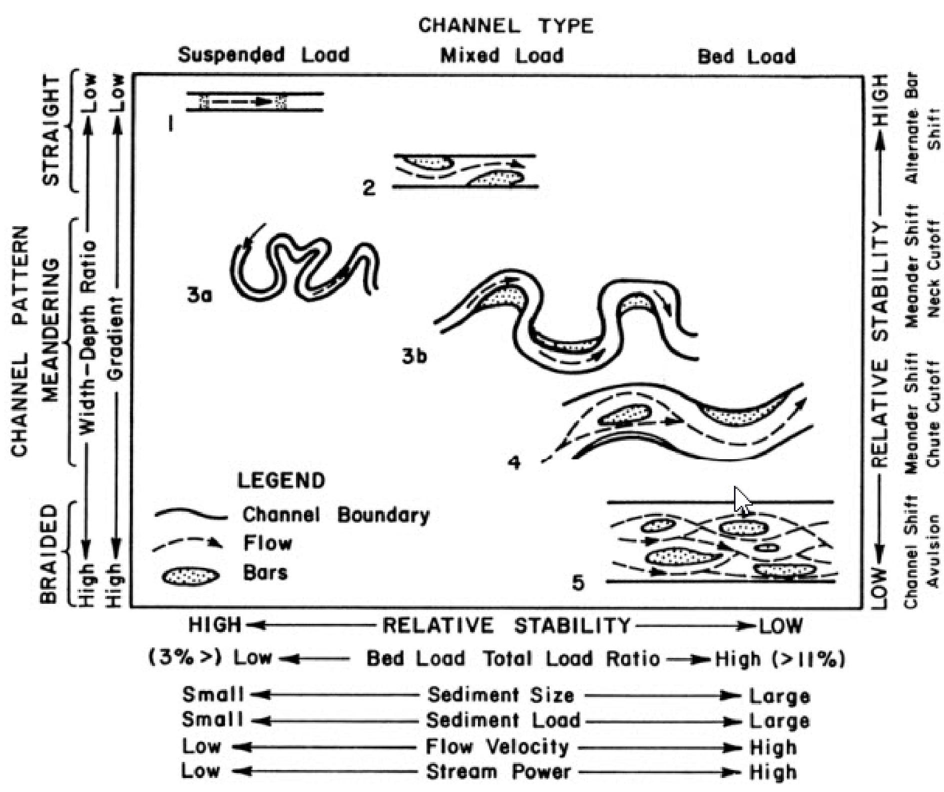


Figure 4.3.5 Plan View of River Categories

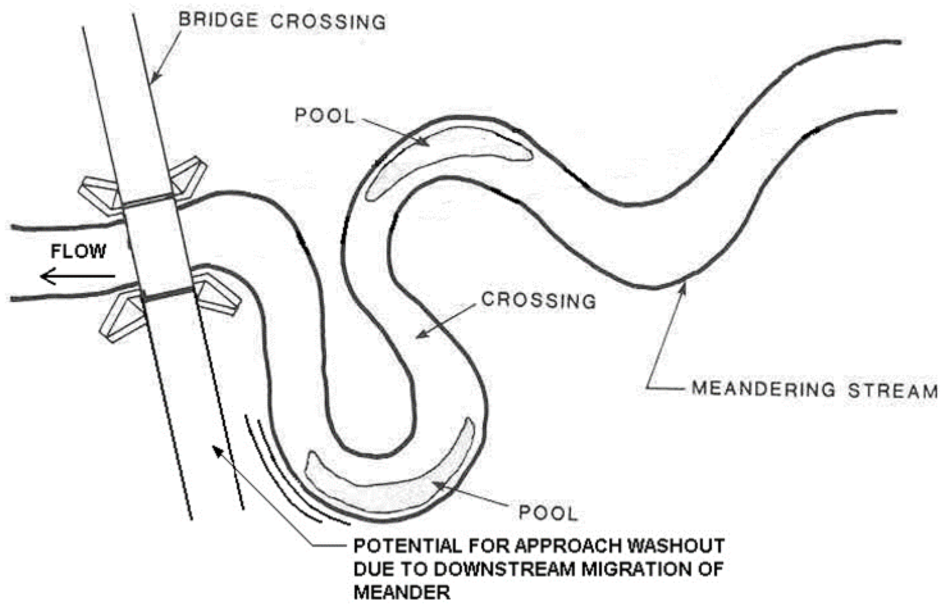


Figure 4.3.6 Meandering River

Braided Rivers

Braided rivers consist of multiple channels that are intertwined in braided form. At flood stages, the appearance of braiding is less noticeable. The bars dividing the multiple channels may become submerged, and the river will likely 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 which can be a hazard to bridges and cause accelerated scour.

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.

Steep Mountain Streams

Steep mountain streams are controlled by geologic formations, rock falls, and waterfalls. They experience very small changes in 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.

4.3.4 Floodplain Characteristics

The floodplain is the overbank area outside the channel that carries flood flows in excess of channel capacity (see Figure 4.3.7). It is common to find bridges with substructure units built within the floodplain. For many bridge sites, the floodplain is quite large, as compared to the channel. Observations made during periods of high water can help the inspector identify the floodplain.

Waterway Terminology

- 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 4.3.7).
- Normal stage – the streamflow stage prevailing during the greater part of the year (between low and high-water levels).
- Waterway area – the entire area beneath the bridge that is available to pass flood flows.

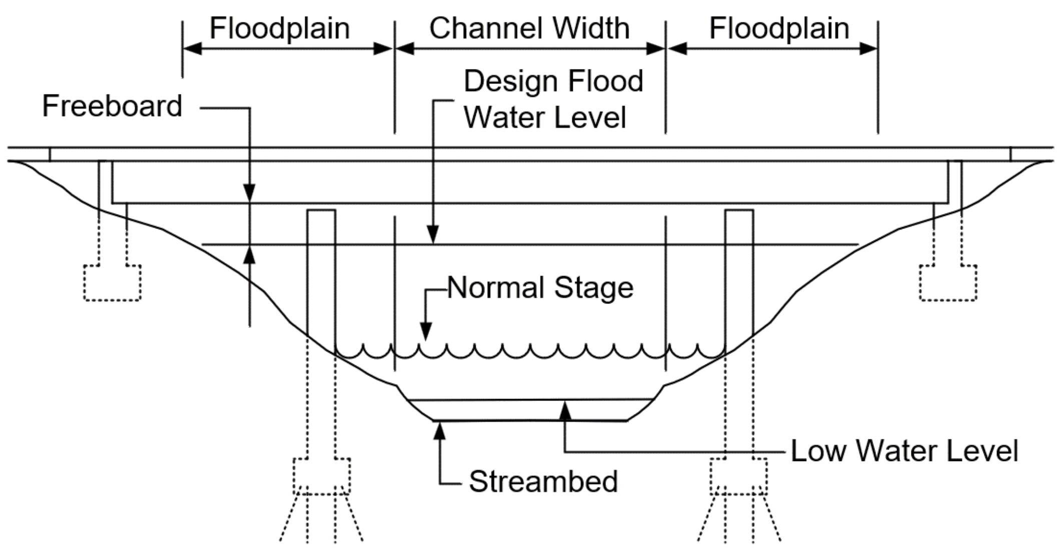


Figure 4.3.7 Typical Floodplain

4.3.5 Hydraulic Opening

The hydraulic opening is the entire area beneath the bridge that is available to pass flood flows (see Figure 4.3.8). The bottom of the superstructure, the two bridge abutments, and the streambed or ground elevation bounds the hydraulic, waterway, or opening. For multiple spans, intermediate supports such as piers or bents restrict the hydraulic or bridge waterway opening.

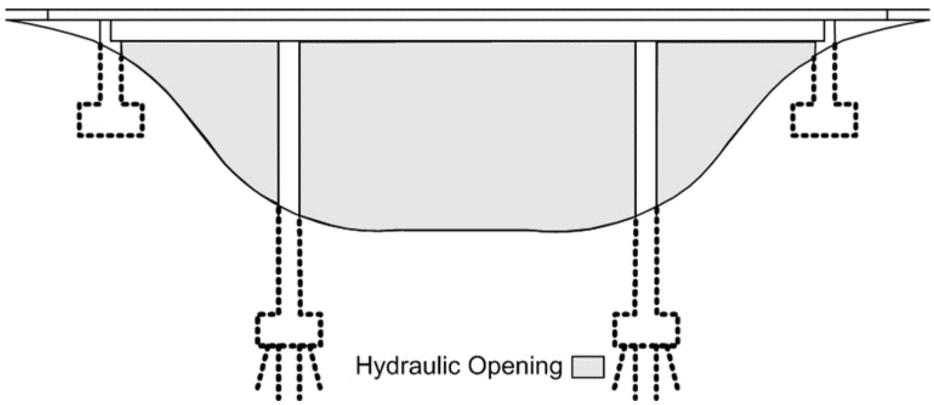


Figure 4.3.8 Hydraulic Waterway Opening

4.3.6 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 placed device designed to direct streamflow and protect against lateral migration or scour. These flow hydraulic control countermeasures may be utilized 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 Training Structures

River training structures are countermeasures designed to modify the flow to help prevent channel damage. A couple examples of river training structures are spurs and guide banks. A complete list of the various types of river training structures can be found in *HEC-23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, 3rd edition*, which can be found at www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09112.pdf.

Spurs

Spurs are linear structures, designed with properly sized and placed rocks, that project 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 4.3.9). 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.

Figure 4.3.9 Spurs

Guide Banks

Guide banks are dikes that extend upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening (see Figure 4.3.10). Scour hole formation occurs at the upstream ends of the guide banks if left unprotected. Riprap is a common scour prevention device for guide banks.

Figure 4.3.10 Guide Banks Constructed on Kickapoo Creek Near Peoria, Illinois

Armoring Countermeasures

Armoring countermeasures tend not to alter the flow significantly when designed properly but are designed 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 can be found in *HEC-23 Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, 3rd edition* (www.fhwa.dot.gov/engineering/hydraulics/pubs/09111/09112.pdf).

Riprap

Riprap consists of 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 4.3.11). Riprap has also been used for almost all kinds of armor which may include wire-enclosed riprap partially grouted riprap, sacked concrete, and concrete slabs. Riprap should be protected against subsurface erosion by filters formed of properly graded sand/gravel or of synthetic fabrics developed and utilized to replace the natural sand/gravel filter system. It should 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 necessitates placement of the riprap on side-slopes no steeper than 1.5 to 1 vertical (1.5H:1V). Flatter side-slopes, 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.

Figure 4.3.11 Stone Riprap

Gabions

Rectangular rock- or cobble- filled wire mesh baskets or compartmented rectangular containers, anchored in union and generally anchored to the surface they are protecting (see Figure 4.3.12). Gabions may be placed on steeper slopes than riprap or may even be stacked vertically, depending upon the design procedure and site conditions.

Figure 4.3.12 Gabion Basket Serving as Slope Protection

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. It is anticipated the various stabilization methods will likely fill-in with sediment and help sustain plant growth. The roots from the plants contribute to stabilize the embankment or flood plain.

Channel lining is a concrete pavement that extends across the streambed (see Figure 4.3.13).

Figure 4.3.13 Formed Concrete Channel Lining

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 enables use over varying land contours and grades (see Figure 4.3.14). Channel linings may also consist of formed concrete. This type is less flexible and versatile than revetment mats and other bed armoring.

Figure 4.3.14 Concrete Revetment Mat

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 4.3.15 and Figure 4.3.16). 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 4.3.15 Concrete Footing Apron on a Masonry Abutment

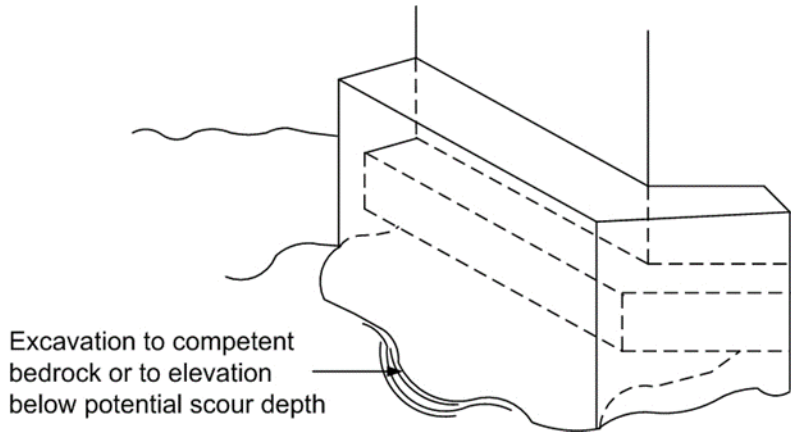


Figure 4.3.16 Concrete Footing Apron to Protect a Spread Footing from Undermining

PART II – BRIDGE, CULVERT, AND WATERWAY CHARACTERISTICS

CHAPTER 5 TABLE OF CONTENTS

CHAPTER 5 LIST OF FIGURES

Chapter 5 Bridges with Complex Features

Section 5.1 Introduction

Characteristics for basic bridge types are presented in Chapter 4. This chapter covers the characteristics of bridges with complex features that were not presented in the previous chapter. Complex features are bridge components or members with advanced or unique structural members or operational characteristics, construction methods, and/or requiring specific inspection procedures. Complex features also include the mechanical and electrical elements of movable spans and cable-related members of suspension and cable-stayed superstructures (23 CFR 650.305). Cable-supported bridges may be necessary when the span length needs are extremely long. Movable bridges are utilized to provide necessary vertical navigable clearances for bridges with low approach spans. Floating bridges may be appropriate for deep water locations, where the foundations would be very expensive. Ferry transfer bridges allow vehicles access to ferries. Ferries can be considered extensions of the road from one side of the waterway to the other.

Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

Section 5.2 Cable-Supported Bridges

5.2.1 Design Characteristics

A cable-supported bridge is a bridge that has the roadway supported by or "suspended from" cables. This section is limited to describing the cables and their elements. All other members of a cable-supported bridge are described in Chapters 9 and 10.

The most notable cable-supported bridge types are suspension bridges (see Figure 5.2.1) and cable-stayed bridges (see Figure 5.2.2). The *Specifications for the National Bridge Inventory (SNBI)* also includes extradosed and other as cable-supported bridge types. Extradosed bridges are a combination of a cable-stayed bridge and a prestressed box girder bridge, in which the cables are essentially external prestressing tendons. Other cable-supported bridges exist and may include less common bridges, such as stress-ribbon bridges, which includes suspension cables embedded in the deck.

Figure 5.2.1 Suspension Bridge

Figure 5.2.2 Cable-Stayed Bridge

Types of Cables

A cable may be composed of one or more parallel wire cables, structural wire strands, structural wire ropes, parallel strand cables, or locked coil strands. Generally, a main suspension cable will likely be many strands or wires. However, a suspender cable may only have one or a few strands. Parallel wire cable consists of several wires that run parallel with each other (see Figure 5.2.3 and Figure 5.2.4). The diameter varies depending on the span length and design loads. Parallel wire cables in older suspension bridges are typically Improved Plow Steel or Double-Improved Plow Steel, which are hard, high strength steels. Parallel wire cables in more modern suspension cables typically conform to ASTM A416 or ASTM A421, which are high strength steel wires typically used in the prestressing industry.

Figure 5.2.3 Parallel Wire Cross Section

Figure 5.2.4 Parallel Wire Cables

A structural wire strand is an assembly of wires formed helically around a center wire in one or more symmetrical layers (see Figure 5.2.5). Sizes typically range from 2 to 4 inches in total diameter. Structural wire rope is an assembly of strands formed helically around a center strand (see Figure 5.2.6). The center strand is typically a steel strand but is sometimes formed from manila (natural) rope or a composite core.

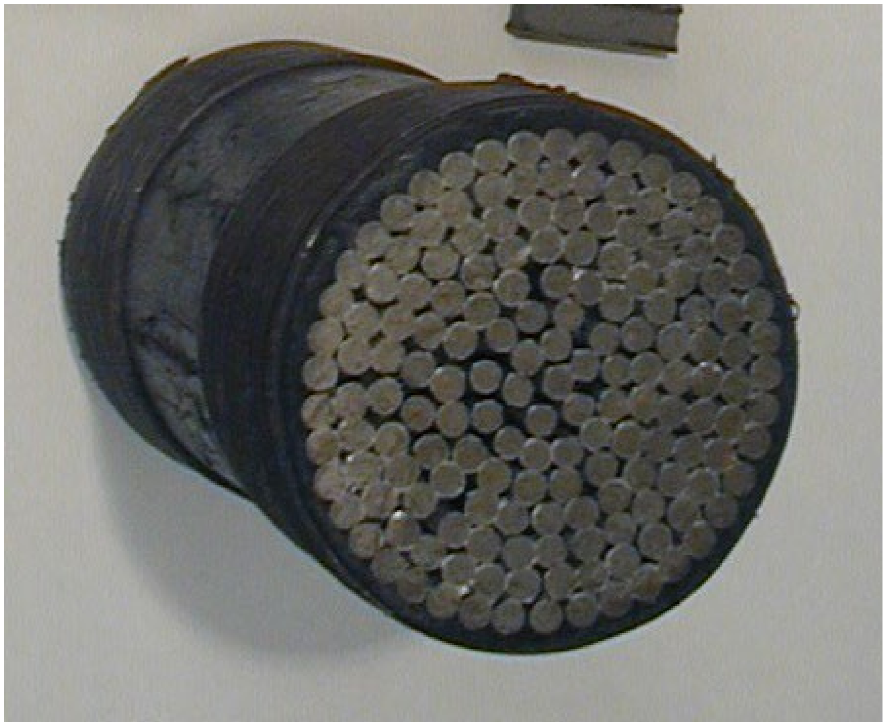


Figure 5.2.5 Structural Wire Strand

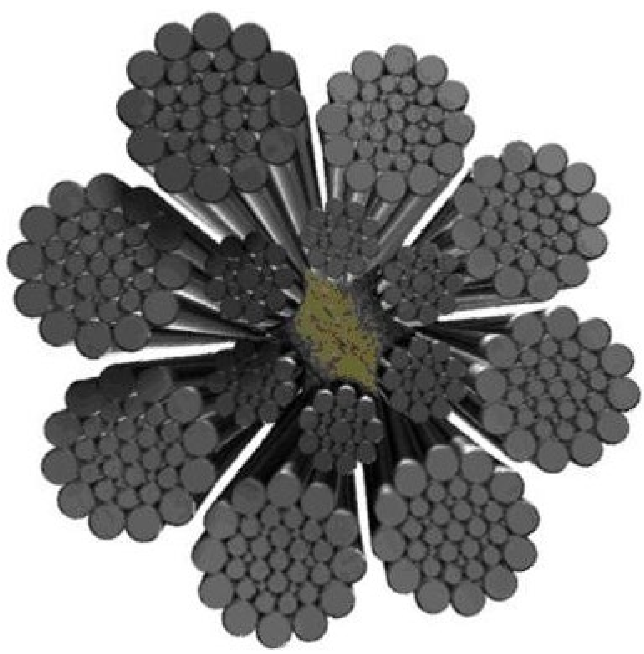


Figure 5.2.6 Structural Wire Rope

Parallel strand cable is a parallel group of strands (see Figure 5.2.7). Seven-wire strands, commonly used for modern cable-supported bridges, typically conforms to ASTM A416. ASTM 416 is high strength, low-relaxation steel typically used in the prestressing industry.

Figure 5.2.7 Parallel Strand Cable

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 5.2.8).

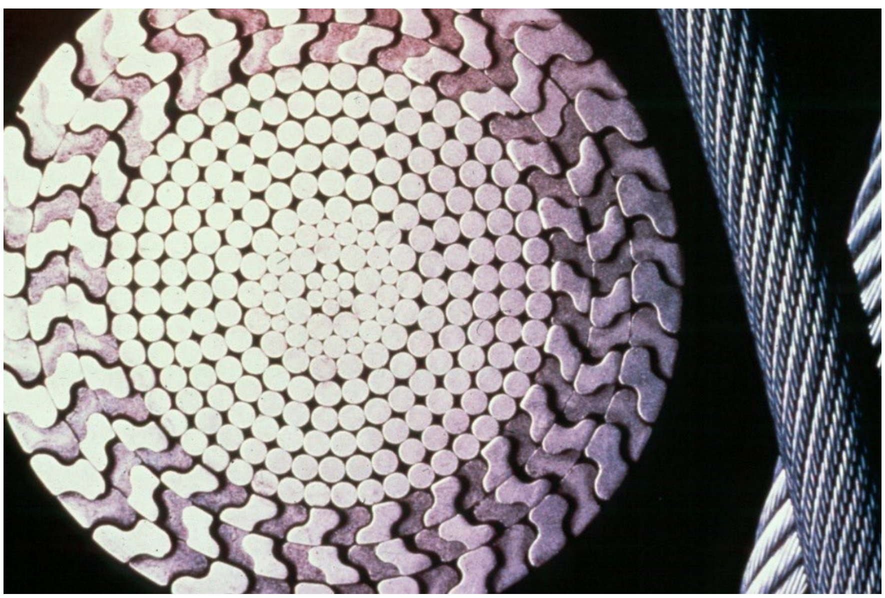


Figure 5.2.8 Locked Coil Strand Cross Section

Cable Corrosion Protection

As the cables are made of steel, it is important to provide protection against deterioration. Methods that have been used for corrosion protection include:

- Galvanizing the individual wires (typically, the outer wires receive a thicker coating of zinc than the interior wires).
- Painting the finished cable.
- Wrapping the finished cable with spirally wound soft galvanized wire, neoprene, or plastic wrap tape (see Figure 5.2.9).
- High Density Polyethylene (HDPE) sheathing filled with cement grout or grease.
- HDPE sheathing with no grouting (common in newer cables).
- Coating the interstices between the wires/strands with a water-repelling paste, previously red lead paste, but zinc-based in current practice.
- Dehumidification systems.
- Any combination of the above systems.

Figure 5.2.9 Cable Wrapping on Older Suspension Bridge

5.2.2 Suspension Bridges

A suspension bridge has a roadway and floor system that 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 or substructure. Suspension bridges are typically constructed when intermediate piers are not feasible because of long span demands due to navigational clearances (see Figure 5.2.10). Modern suspension bridges generally have main spans longer than 1400 ft and can currently reach span lengths up to 6100 ft.

Figure 5.2.10 Suspension Bridge

Main Suspension Cables and Suspender Cables

Main suspension cables are generally supported 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 5.2.11).

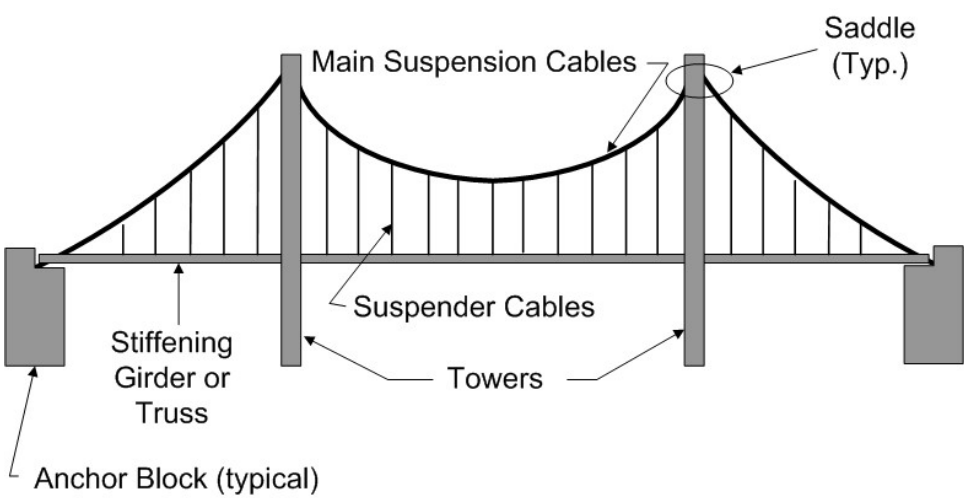


Figure 5.2.11 Three-Span Suspension Bridge Schematic

If a suspension bridge has only two main suspension cables, the cables are considered nonredundant steel tension members (NSTM) since there is no load path redundancy. Refer to Chapter 7 for a detailed description of NSTMs and redundancy. Both parallel wire and parallel strand cable exhibit excellent internal redundancy, as many individual wires have to break in order to appreciably affect their load-carrying capacity. Also, although individual wires can break, the large amount of friction between a broken wire and the adjacent, unbroken wires can re-develop the broken wire's tension, akin to the development length of rebar embedded in concrete.

Another type of suspension bridge is the self-anchored suspension bridge, where the main suspension cables are anchored into the edge stiffening girders or trusses that span continuously from end to end in the suspension spans. The tensile force in the cables is equilibrated by compression in the stiffening girders and trusses. The edge girders support the floor system and the suspender cables support the edge girders/trusses in this arrangement.

This type of 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 and the suspension span creates the wide navigation channel. The anchoring is self-contained within the superstructure of the suspension spans (see Figure 5.2.12).

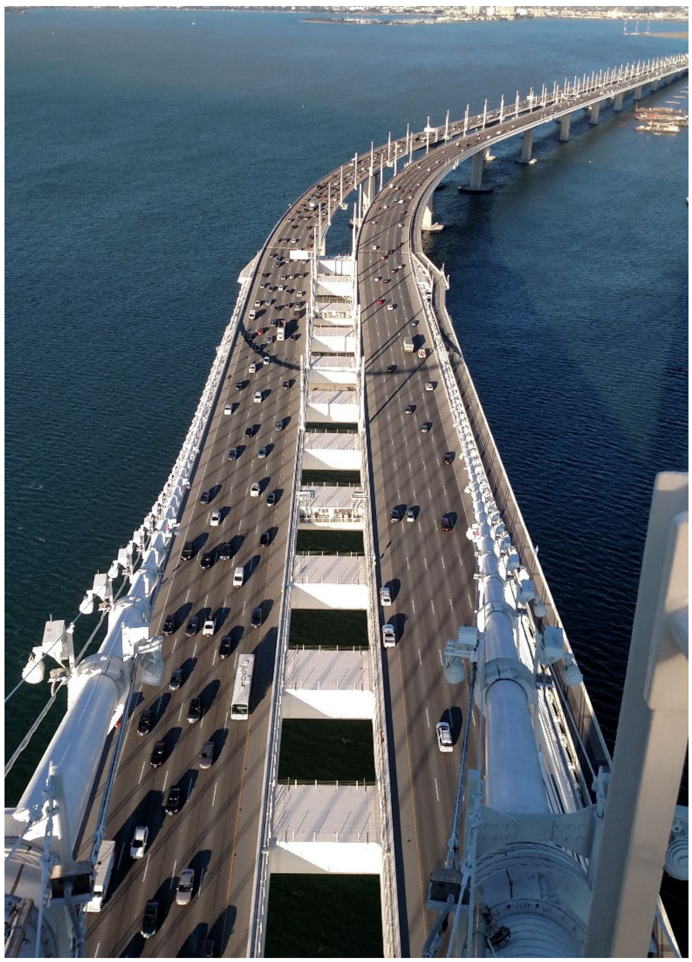


Figure 5.2.12 Oakland Bay View Bridge

Anchorage and Connections

In bridges with common earth-anchored cable systems, above or below ground, the total force of the main suspension cable is transferred into the anchor block (see Figure 5.2.13). The void area inside the anchor block is referred to as the chain gallery or anchor vault. The force from the main cable is distributed through the splay saddle, bridge wires, strand shoes, and the anchor bars. The cable wrapping is terminated at the splay saddle, and the individual cable strands are separated and anchored to the strand shoes. 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.

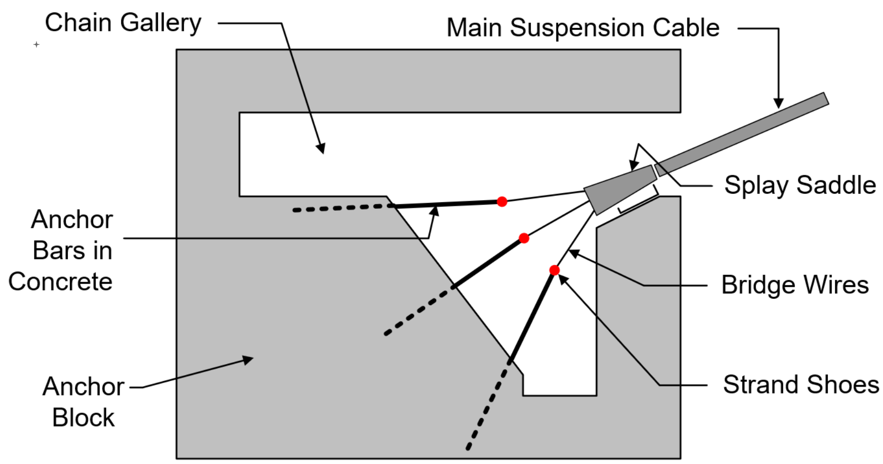


Figure 5.2.13 Anchor Block Schematic

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 5.2.14). Saddles are commonly made from fabricated steel or castings.

Figure 5.2.14 Cable Saddle

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 strength steel bolts to develop the necessary friction necessary to maintain the saddle's position on the cable.

Grooved cable bands have been used in the majority of suspension bridges. The top surfaces of the bands are grooved to receive the suspender cables, which are looped over the cable band (see Figure 5.2.15).

Figure 5.2.15 Grooved Cable Bands

Instead of looping the hanger cables around the main suspension cable, the hanger cable may terminate in cast sockets, which can connect to the cable bands and floor system using pins, stiffeners, bearing shoes, or a combination of these. The connection is called an open socket suspender cable connection (see Figure 5.2.16). Connection to the deck and floor system can also be a similar open socket arrangement or it can be connected directly to a floorbeam.

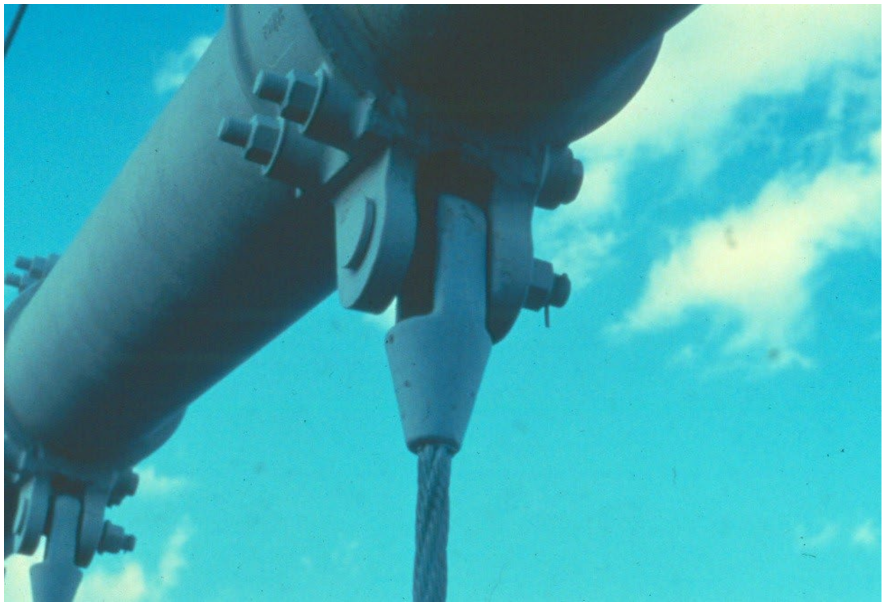


Figure 5.2.16 Open Socket Suspender Cable Connection

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. Wind-induced vibrations, particularly the aerodynamic instability known as flutter, can produce forces that create an oscillating effect on the structure. The term local vibration is used when dealing with the vibration in an individual member (see Figure 5.2.17). When the vibration of the entire structure is analyzed, it is known as global vibration (see Figure 5.2.18).

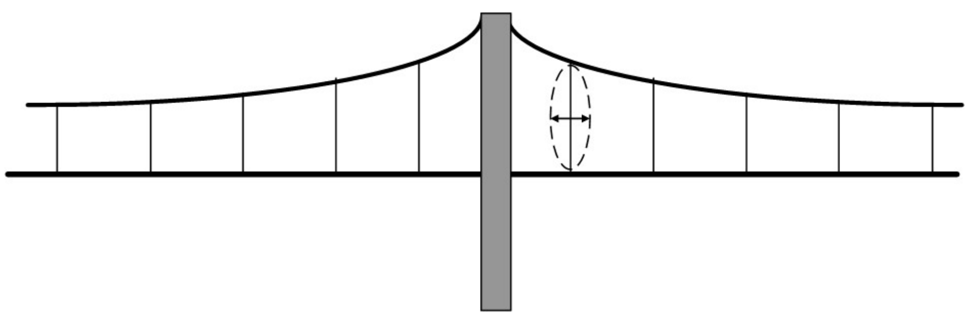


Figure 5.2.17 Cable Vibrations Local System Schematic

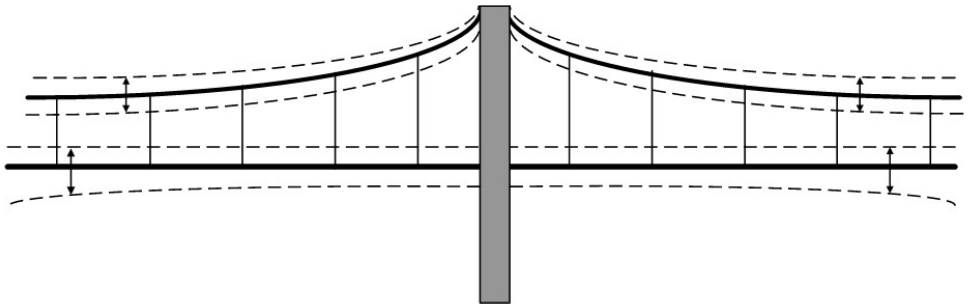


Figure 5.2.18 Cable Vibrations Global System Schematic

Due to the amount of vibration in cable-supported structures, it may be common to observe 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 5.2.19 and Figure 5.2.20).

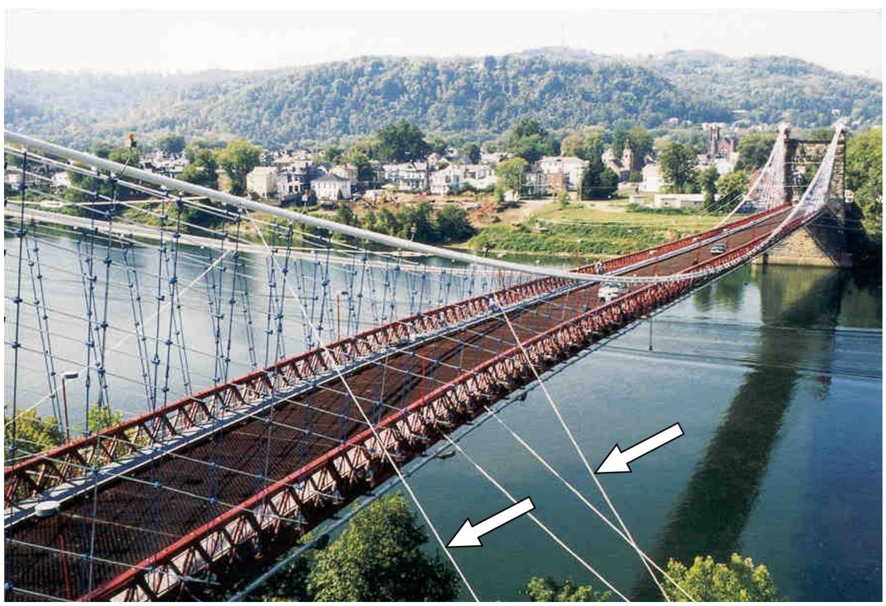


Figure 5.2.19 Cable Damping System for Suspension Bridge

Figure 5.2.20 Cable Damper System

Vibrations can affect suspension cables in several ways. Vibration can open cable wires allowing entry of water and atmospheric contaminants, including salts and potentially corrosive chemicals. The clamps on neoprene boots near the transition from HDPE to steel anchorage pipes can be loosened by vibrations in the cables. Vibrations can also create fretting wear, cracks in the protective coating and cement grout, and accelerate corrosion and possibly fatigue.

Towers

For a suspension bridge, the portal tower is the typical tower type. Towers are constructed of reinforced concrete, steel, masonry, or a combination of these materials. (see Figure 5.2.21 and Figure 5.2.22).

Figure 5.2.21 Concrete Portal Tower

Figure 5.2.22 Steel Portal Tower

5.2.3 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 situated at the main piers. Cable-stayed bridges are a relatively modern development within the area of cable-supported bridges. There was a need for a type of structure that could span a distance greater than a truss or arch, but short enough that a suspension bridge was not economical. Spans generally range from 700 to 1400 ft (see Figure 5.2.23).

Figure 5.2.23 Cable-Stayed Bridge

In cable-stayed bridges, the deck and floor system are supported directly from the tower with taut stay cables, as opposed to suspension bridges requiring additional vertical suspender cables. During construction, the forces in the stays may change dramatically, and the amount of force in each stay may be adjusted independently to produce a particular stress state in the deck and floor system. Due to the complexity of the various cable arrangements and systems, system redundancy for individual cable-stayed structures can only be determined through a detailed structural analysis.

Cable Arrangements and Systems

Several types of cables have been used for cable-stayed bridges. The three most common main tension-resisting elements (MTE) are locked-coil strand, parallel wire, and parallel seven-wire strand described previously in this section. MTEs could also be single bar or multiple parallel bars. Most of the cable-stayed bridge inventory outside of the United States utilizes galvanized, lock-coil strand. Stay cables in the United States are typically parallel-wire or seven-strand cables.

Cable-stayed bridges may be categorized according to the various stay cable arrangements. These cable arrangements are categorized into the following four basic systems:

- Radial or Converging.
- Harp.
- Fan.
- Star.

In a radial or converging cable system, all stay cables anchor at the top of the tower at a common point. Structurally, this arrangement is typically the most effective and therefore generally preferred. By anchoring all the cables to the tower top, the maximum inclination to the horizontal is achieved (see Figure 5.2.24). If a cable is more inclined, the stresses in the cable decrease, which can result is a smaller necessary cross section of the tower. As the incline decreases, the length of the cables increases, resulting in generally higher stresses and larger cables.

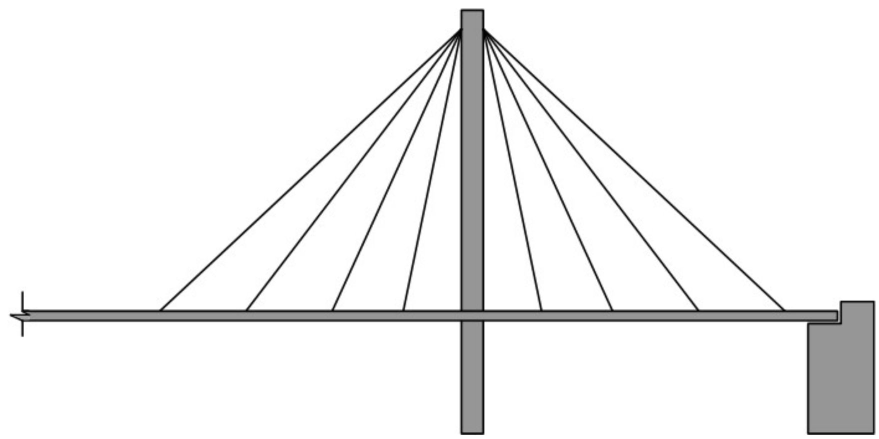


Figure 5.2.24 Radial or Converging Cable System Schematic

The harp system, as the name implies, resembles harp strings. In this system, the cables are typically 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 5.2.25).

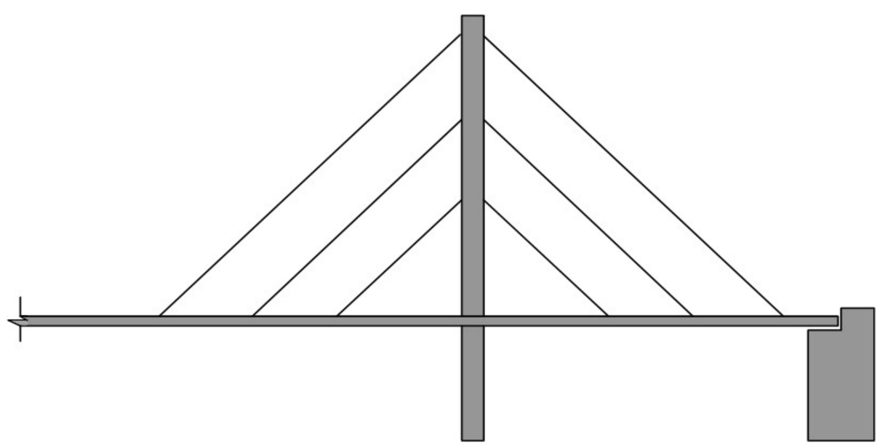


Figure 5.2.25 Harp or Parallel Cable System Schematic

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 5.2.26).

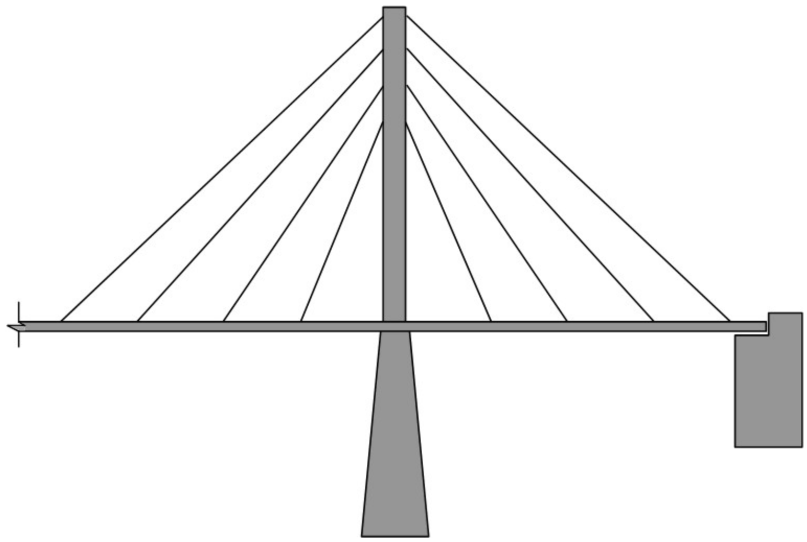


Figure 5.2.26 Fan or Intermediate Cable System Schematic

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 situated over the abutment or end pier. The star system is uncommon compared to the three systems previously presented. The star system demands a much stiffer superstructure since the cables are not distributed longitudinally along the span (see Figure 5.2.27).

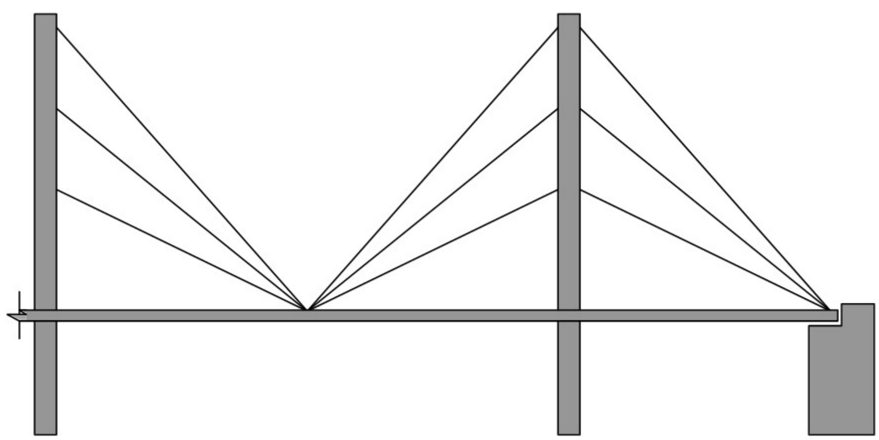


Figure 5.2.27 Star Cable System Schematic

Cable Planes

The cables may lie in a single or a double plane, may be symmetrical or asymmetrical, and may lie in oblique or vertical planes.

The single-plane cable arrangement can be 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 5.2.28).

The double vertical plane system incorporates two vertical cable planes connecting the tower to the edge girders along the deck structure (see Figure 5.2.28). The structure may utilize twin towers or a portal frame tower. The portal frame tower is a twin tower with a connecting strut at the top. Wider bridges may utilize a triple plane system which is basically a combination of the single and double plane systems.

Figure 5.2.28 Single and Double Vertical Plane Cable System

In a double-plane oblique system, the cable planes slope toward each other from the edges of the deck and intersect at the tower along the longitudinal centerline of the deck (see Figure 5.2.29). Generally, the tower is an A-frame type, receiving the sloping cables that intersect close to the centerline on the tower.

Figure 5.2.29 Oblique Double-Plane Cable System

Anchorages and Connections

The cables may be continuous and pass through or over the tower or may be terminated at the tower.

If the cables are continuous across the tower, a saddle is incorporated. The cable saddles are similar to those for suspension bridges (see Figure 5.2.14). Between the end and center spans differential forces will most likely 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.

If the cables are terminated at the tower, an end fitting or anchorage is incorporated. A similar anchorage is utilized at the edge girder (see Figure 5.2.30). In the cable free length, or the portions of the cable that are not within the anchorage zone, the wires or strands are bound. To attain proper anchorage, the wires or strands should be separated. A basic type of anchorage is the point anchorage concept (see Figure 5.2.31). It includes an anchor plate with gripping wedges to hold the wire strands in place. The end cap, or socket, and anchorage pipe houses the length of the anchorage and may or may not be filled with a protective substance.

Figure 5.2.30 Cable Deck Anchorage

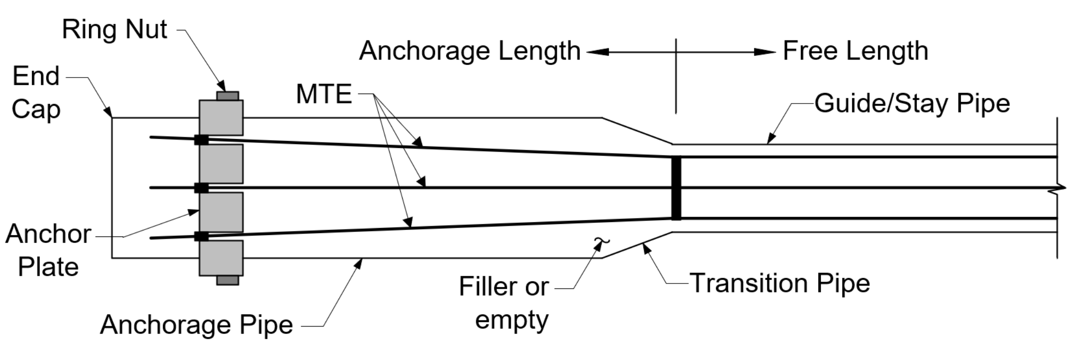


Figure 5.2.31 Point Anchorage Schematic

A poured zinc socket is seldom used for the anchoring of parallel-wire strands in the United States. In a poured-zinc socket system, the wires are led through holes in a locking plate at the end of the socket (see Figure 5.2.32). The individual wires or strand wires are separated within the wedge-shaped socket reservoir and the void of the reservoir is filled with molten zinc or a molten zinc alloy. Additionally, there are epoxy- or composite-based casting materials available that do not necessitate heat. Both methods achieve the same goal of locking the cable wires into the socket.

Figure 5.2.32 Anchor Inspection

Problems encountered with low fatigue strength of zinc-poured sockets led to the development of High Amplitude (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 a compound composed of zinc dust, steel ball bearings, and an epoxy bonding compound. This method of anchoring the stays increases the magnitude of fatigue resistance to almost twice that of zinc-poured sockets.

Another common anchorage type for strands is the Freyssinet type anchor. In the Freyssinet anchor 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 likely be transferred 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, vibration of cables with other bridge elements excited by wind, inclined cable galloping, and vortex excitation of single cables or groups of cables. Due to the amount of vibration in cable-supported structures, it may be common to observe various types of damping systems attached to cables. Damping systems may be a tie between two cables (see Figure 5.2.33), neoprene cushions, shock absorbers mounted to the cables (see Figure 5.2.34), or other systems that act to dampen the cable vibrations (see Figure 5.2.35).

Figure 5.2.33 Crossties between Cables

Figure 5.2.34 Anti-Rattling Device Installed on Cable

Figure 5.2.35 Damper for Cable-Stayed Bridge

Vibrations can affect stay cables in several ways. Vibration opens cable wires allowing entry of corrosive chemicals. The clamps on neoprene boots near the transition from HDPE to steel anchorage pipes can be loosened by vibrations in the cables, allowing water to infiltrate the system. Vibrations create fretting wear, cracks in the protective coating, and cement grout cracking (if present). All of these issues can ultimately lead to accelerated corrosion and possibly fatigue in the cables.

Towers

Cable-stayed bridges can utilize different tower shapes, also called pylons, depending on the design of the structure. Some of the shapes constructed to date are:

a. Portal tower (see Figure 5.2.36 (a)).
b. Towers fixed to pier (see Figure 5.2.36 (b)).
c. Towers fixed to superstructure (see Figure 5.2.36 (c)).
d. Single column tower (see Figure 5.2.36 (d) and Figure 5.2.37).
e. A-frame tower (see Figure 5.2.36 (e) and Figure 5.2.37).
f. Laterally offset tower fixed to pier (see Figure 5.2.36 (f)).
g. Diamond shaped tower (see Figure 5.2.36 (g)).

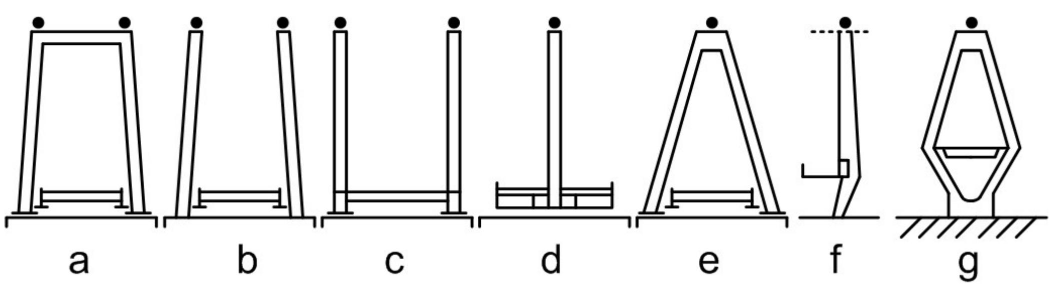


Figure 5.2.36 Shapes of Towers Used for Cable-Stayed Bridges

Figure 5.2.37 Single Column Tower and A-Frame Tower

Section 5.3 Movable Bridges

5.3.1 Design Characteristics

Movable bridges (see Figure 5.3.1) contain many unique members in addition to the members that are common in other bridge types. Each movable bridge type has unique complex features and unique mechanisms for movement.

Figure 5.3.1 Movable Bridge

Movable bridges are typically constructed only when it is too expensive or impractical for a fixed bridge to accommodate channel traffic. Movable bridges are constructed across designated "Navigable Waters of the United States", in accordance with "Permit Drawings" approved by the U.S. Coast Guard.

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 the vast majority of the total number of movable bridges within the United States. These categories include:

- Swing bridges.
- Bascule bridges.
- Vertical lift bridges.

5.3.2 Swing Bridges

Swing bridges consist of two-span continuous trusses or girders that rotate horizontally about the center (pivot) pier (see Figure 5.3.2). The spans are usually, but not necessarily, equal. When open, the swing spans are cantilevered from the pivot (center) pier and should 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 5.3.2 Center-Bearing Swing Bridge

Swing spans can be subdivided into two types: center-bearing and rim-bearing. 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 have also been used. The bridge is rotated horizontally by a circular rack and pinion arrangement, or by hydraulic cylinders.

Center-Bearing

Center-bearing swing spans carry the entire load of the bridge on a central pivot, usually metal discs (see Figure 5.3.3). Balance wheels are placed on a circular track around the outer edges of the pivot pier to prevent tipping. 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. These wedges generally do not raise the span at the pivot pier but are merely driven tight.

Recently constructed or re-constructed swing bridges are nearly all designed as center-bearing. Center-bearing swing spans are typically less complex and less expensive to build than rim-bearing swing spans.

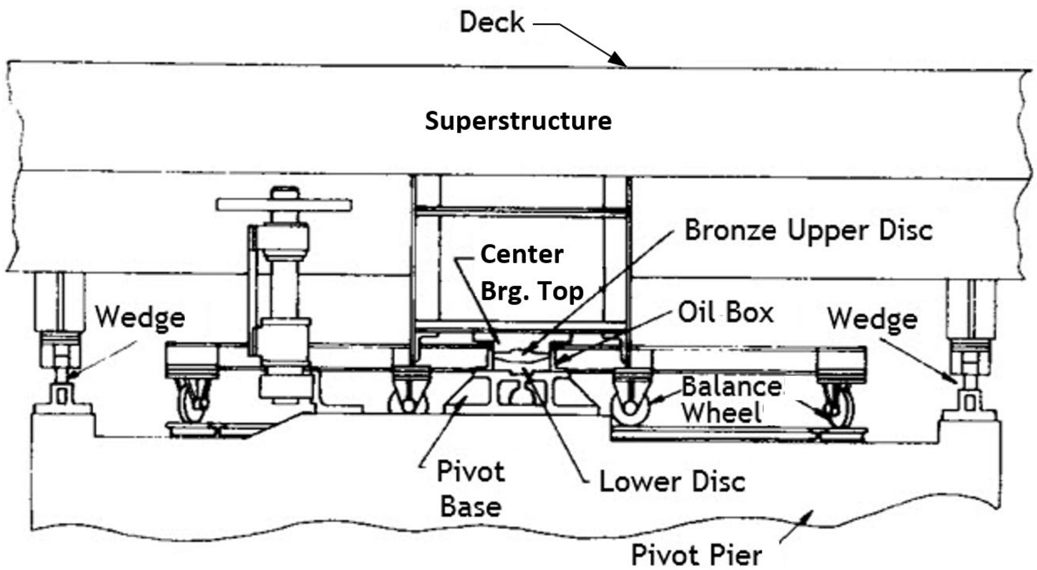


Figure 5.3.3 Center-Bearing Schematic

Rim-Bearing

Rim-bearing swing spans (see Figure 5.3.4) 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 (see Figure 5.3.5). 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.

Figure 5.3.4 Rim-Bearing Swing Span in Open Position

Figure 5.3.5 Rim-Bearing Swing Span Rollers

5.3.3 Bascule Bridges

Bascule bridges open by rotating a single "leaf" or two "leaves" (movable portion(s) 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 5.3.6). If the channel is narrow, a single-leaf bascule bridge may be sufficient (see Figure 5.3.7). For wider channels, two leaves are typically used, one on each side of the channel. This is known as a double-leaf bascule bridge. When the leaves are in the lowered position, they meet at the center of the channel.

Figure 5.3.6 Double-Leaf Bascule Bridge in the Open Position

Figure 5.3.7 Single-Leaf Bascule Bridge in the Open Position

A counterweight is necessary to hold the raised leaf in position. In older bridges, the counterweight is usually overhead. However, in more modern bascule bridges, the counterweight is placed below the deck and lowers into a pit as the span is opened.

The leaf is lifted by rotating vertically about a horizontal axis. The weight of the counterweight is adjusted by removing or adding balance blocks in counterweight 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 situated 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 connect the superstructure members.

There are many types of bascule bridges, but the most common are:

- 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 (McBriarty). 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 when the moving leaf rotates open (see Figure 5.3.8 and Figure 5.3.9).

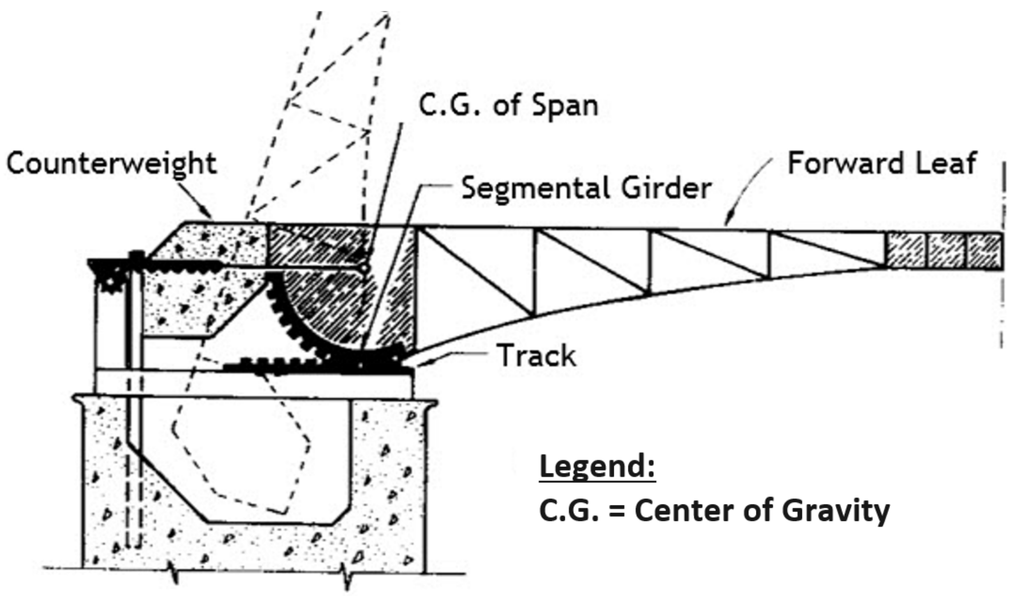


Figure 5.3.8 Rolling Lift Bascule Bridge Schematic

Figure 5.3.9 Double-Leaf Rolling Lift Bascule

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.

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.

The two leaves act as a three-hinged arch when the span is 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.

Simple Trunnion (Chicago) Bridge

The Chicago Bridge Department designed and built the first Chicago-type simple trunnion bascule bridge in 1902 (McBriarty). This type of bascule bridge consists of a forward cantilever arm over the channel and a rear counterweight arm. The leaf rotates about the trunnions. Each trunnion is typically 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 Figure 5.3.10 and Figure 5.3.11).

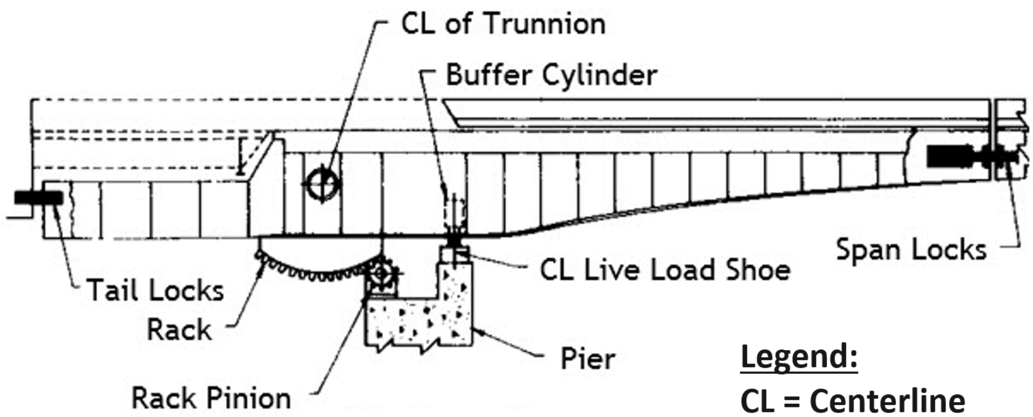


Figure 5.3.10 Trunnion Bascule Bridge Schematic

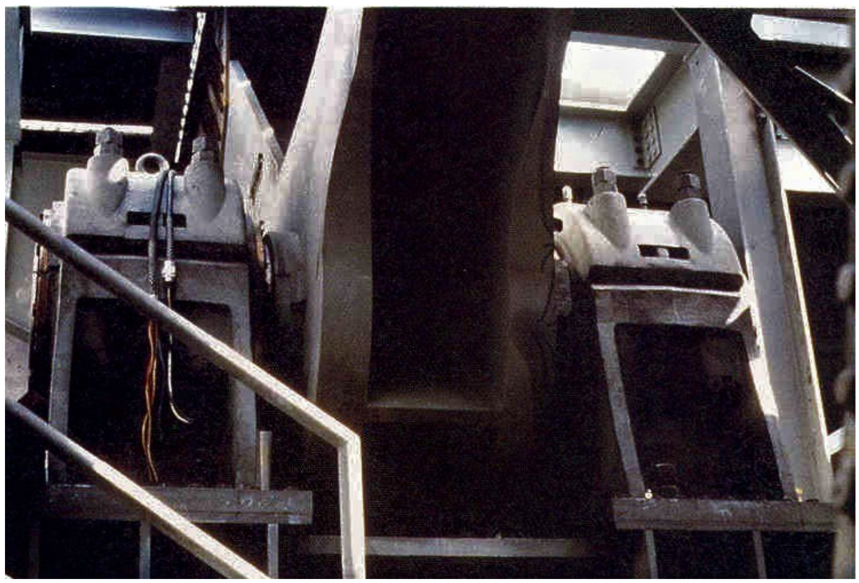


Figure 5.3.11 Trunnion Supported by Two Bearings

The machinery is situated outside of the superstructure, on the pier. Forward bearing supports (live load shoes) in front of the trunnions are engaged when the leaf reaches the fully closed position. They are intended to support only live load reactions. Uplift supports are 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. Of the three types of bascule bridges, the simple trunnion is by far the most common.

Multi-Trunnion (Strauss) Bridge

The first multi-trunnion (Strauss) bascule bridge was designed by J.B. Strauss and completed in 1905 in Cleveland, Ohio (Griggs). 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 typically 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 when the moving leaf rotates about the main trunnions (see Figure 5.3.12). In the figure, the solid line in the trunnion indicates the open position and the dashed line for the trunnion indicates the closed position.

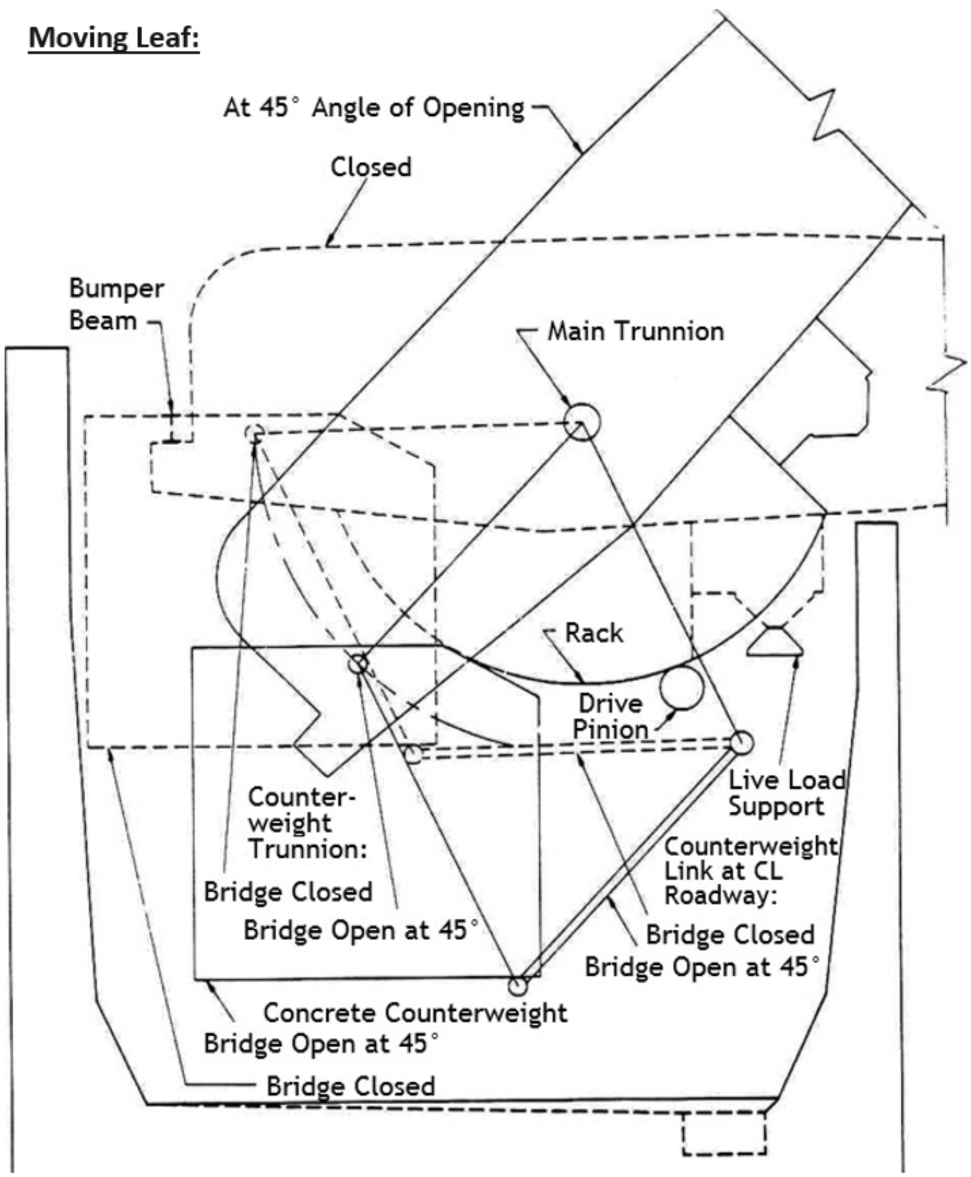


Figure 5.3.12 Multi-Trunnion, Strauss Type Bascule Bridge

5.3.4 Vertical Lift Bridges

Vertical lift bridges have a movable span with a fixed tower at each end. The span is supported by steel wire ropes at its four corners when opened. The ropes pass over sheaves (pulleys) atop the towers and connect to counterweights. The counterweights descend as the span ascends. 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 in the closed position and carrying traffic (see Figure 5.3.13).

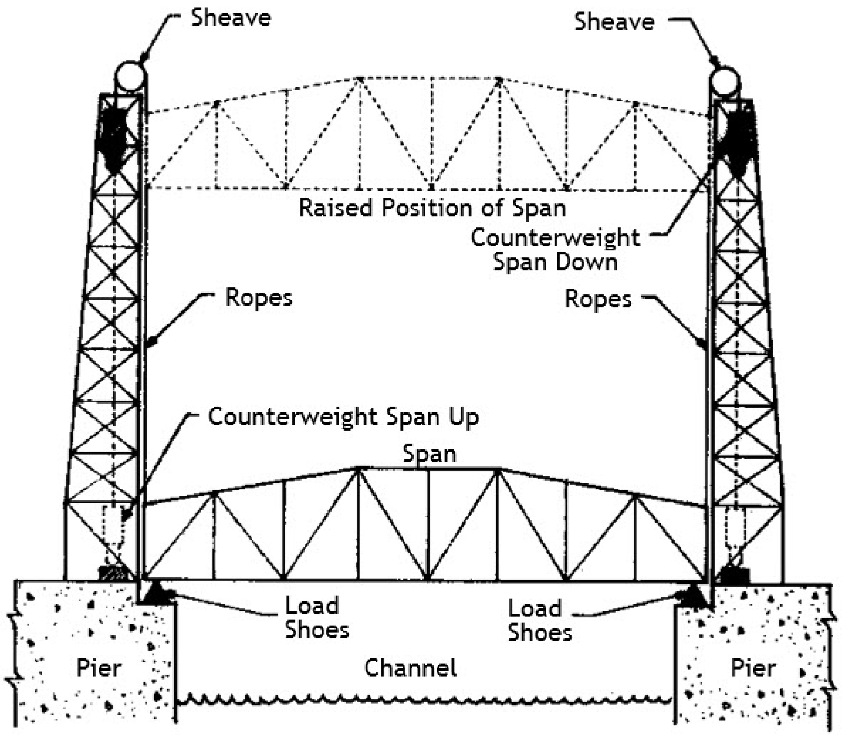


Figure 5.3.13 Vertical Lift Bridge Schematic

There are three basic types of vertical lift bridges:

- Connected-tower.
- Span-drive.
- Tower-drive.

Connected-Tower Systems

This type of vertical lift bridge has a rigid span between the towers (see Figure 5.3.14). The operating machinery is typically at the center of the rigid span and drives all four sheaves to raise and lower the movable span. It is generally used in applications where a shorter span length is adequate for the lift span.

Figure 5.3.14 Connected-Tower Vertical Lift Bridge

Span-Drive Systems

The span-drive vertical lift bridge was the first vertical lift bridge of an appreciable size built in the United States. The South Halsted Street Bridge crossing the Chicago River was a 130-foot-long span-drive lift bridge completed in 1894 (Spivey). It was capable of providing a vertical clearance of 155 ft. The bridge was designed by J.A.L. Waddell, who is considered the father of movable bridge design.

The drive machinery house is typically on top of the lift truss span for this bridge type (see Figure 5.3.15). The span is lifted using "up-haul and down-haul ropes" where turning drums wind the up-haul (lifting) ropes as they simultaneously unwind the down-haul ropes. The operating drums rotate to wind the up-haul (lifting) ropes as they simultaneously unwind the down-haul ropes. The operating ropes do not support the weight of the span as it moves up and down, instead the weight is carried entirely by the counterweight ropes. A variation of this type provides drive pinions at both ends of the lift span which engage racks on the towers.

Figure 5.3.15 Span-Drive Vertical Lift Bridge

Tower-Drive Systems

Two sets of drive machinery can be found on top of both towers for this basic type of vertical lift bridge, where drive pinions operate against circular racks on the sheaves. A large sheave is mounted on each side of the tower, with counterweight ropes wrapping 180 degrees around the sheaves. One end of the rope is attached to the span and the other to the counterweight. When the machinery is found on the towers, the rim gears and operating sheaves are rotated to raise and lower the bridge (see Figure 5.3.16). The lifting speed at both towers should be synchronized to keep the span horizontal as it is lifted.

Figure 5.3.16 Tower-Drive Vertical Lift Bridge

5.3.5 Unique Features of Movable Bridges

Movable bridges (swing, bascule, and vertical lift) need numerous unique devices for operation that are not found in typical bridges. Features commonly found on movable bridges may include:

- 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 safety devices, including signs and signals.
- Navigable waterway signals and signs, including lighting.

Open gearing is used to transmit power from one shaft to another and to alter the speed and torque output of the machinery (see Figure 5.3.17). Beveled gears are also used to change direction of motion.

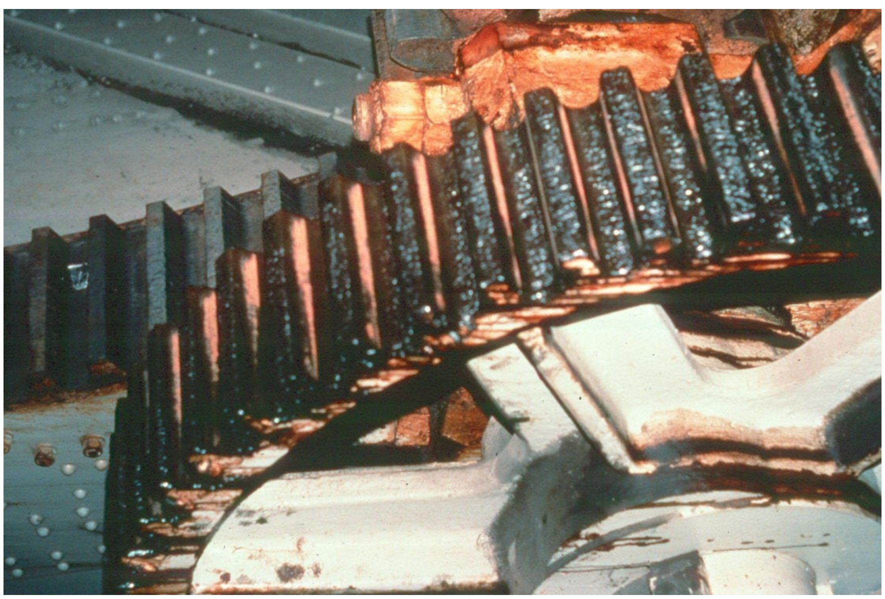


Figure 5.3.17 Open Gearing

Speed reducers (see Figure 5.3.18), including differentials, serve the same function as open gearing. However, they may contain several gear sets, bearings, and shafts to provide a compact, packaged unit, which protects its mechanical parts and lubrication system within an enclosed housing. Differential speed reducers also function to equalize torque and speed from one side of the mechanical operating system to the other.

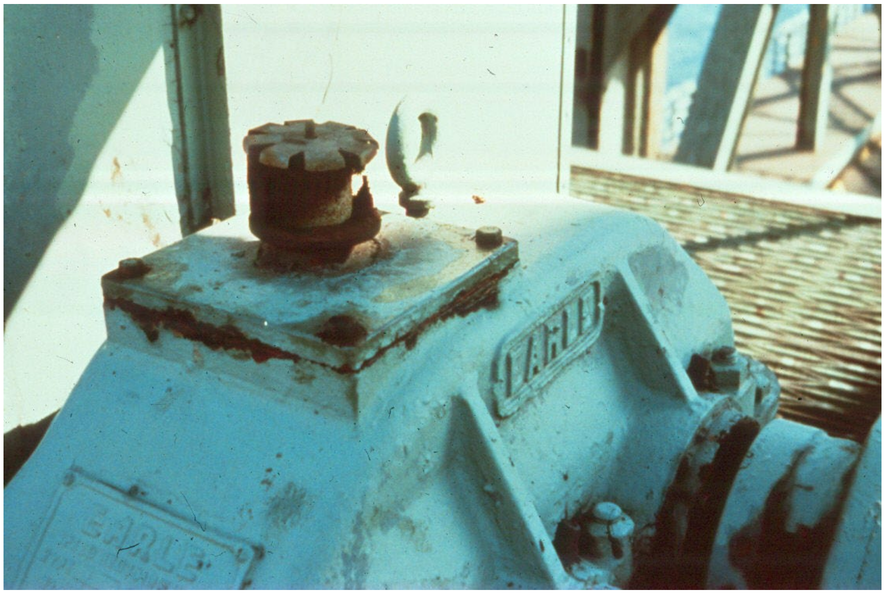


Figure 5.3.18 Speed Reducer

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 5.3.19).

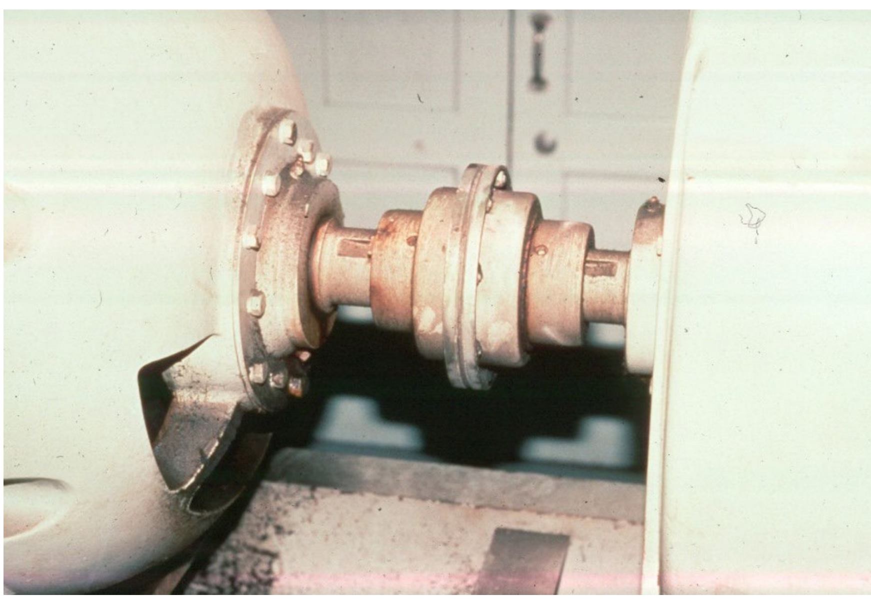


Figure 5.3.19 Coupling Connecting Two Shafts

Bearings provide support and prevent misalignment of rotating shafts, trunnions, and pins (see Figure 5.3.20).

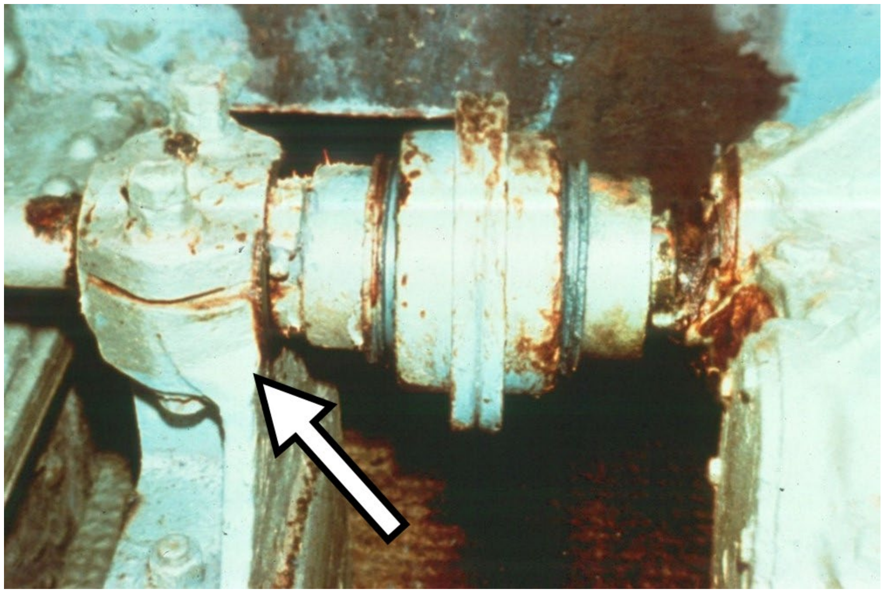


Figure 5.3.20 Bearing

Brakes can be shoe- or disc-type, and can be released manually, electrically, or hydraulically (see Figure 5.3.21 and Figure 5.3.22). They are generally spring applied for fail-safe operation. Motor brakes are 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 typically found closer to the operating interface between movable and fixed parts of the bridge and are typically used to hold the span statically, in addition to serving as emergency brakes in many cases. Supplemental emergency brakes are sometimes also provided.

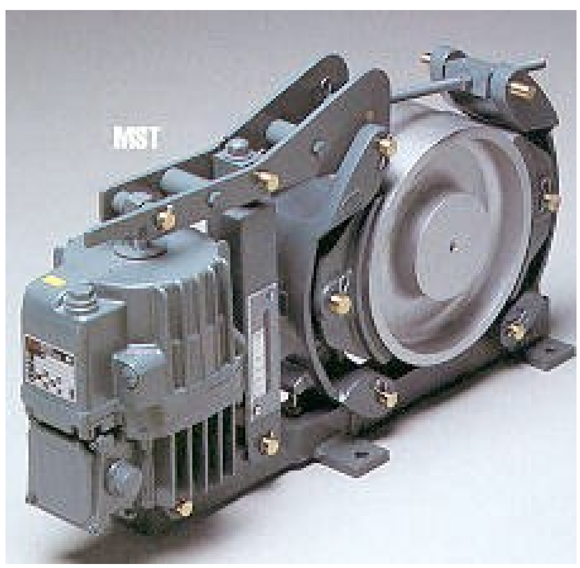


Figure 5.3.21 Shoe Type Brake

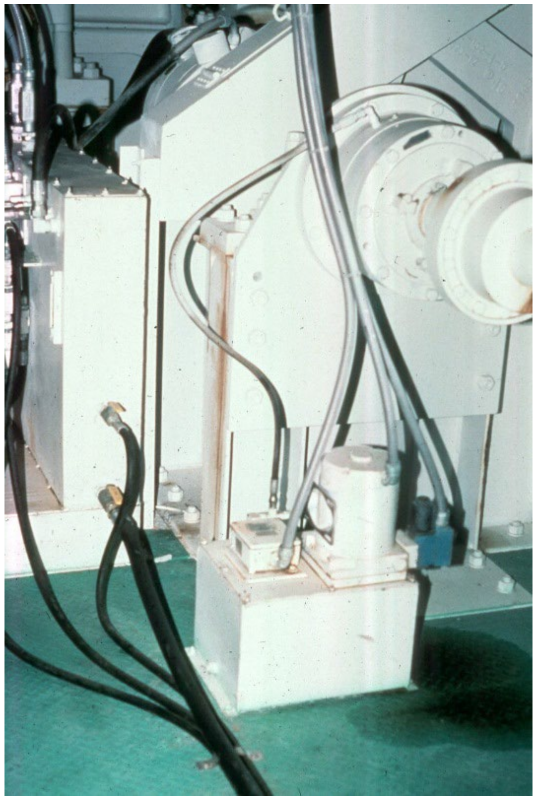


Figure 5.3.22 Spring Set Hydraulically Released Disc Brake

Drives can consist of electric motors, hydraulic equipment, or auxiliary drives.

For electric motors, AC or DC power may be used. AC power is often used to power any wound-rotor motors with torque controllers on older bridges. However, 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 large actuating cylinders or hydraulic motors (see Figure 5.3.23). The prime mover should 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.

Figure 5.3.23 Low Speed High Torque Hydraulic Motor

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 5.3.24). In some cases, to prevent the need for larger auxiliary generators, the auxiliary motors are necessary for use any time the auxiliary generators are used, which results in increased time of operation.

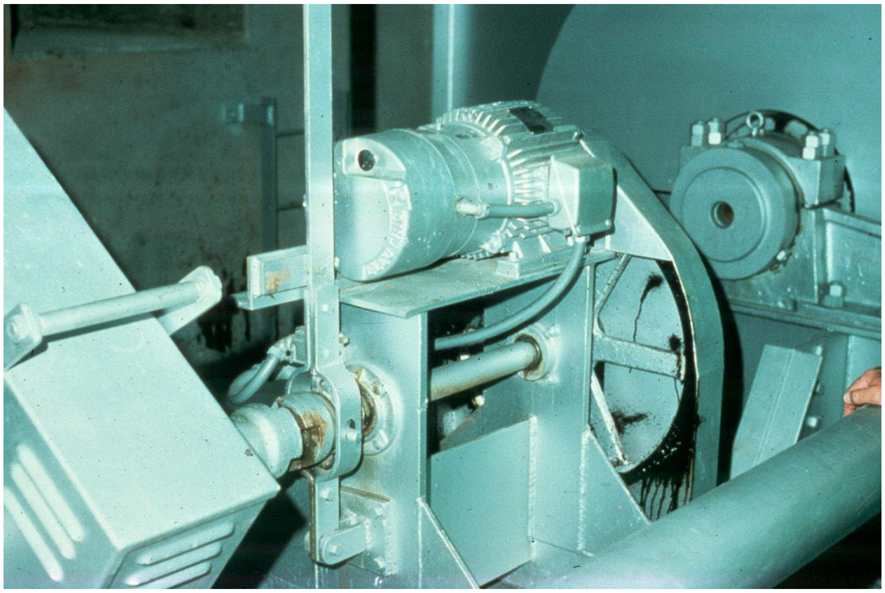


Figure 5.3.24 AC Emergency Motor

Air buffers and shock absorbers may be found between the span and the pier at points where impact may occur between the two (see Figure 5.3.25). 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 5.3.26). A pressure relief valve allows the air to escape beyond the pressure setting. Forces are necessary 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.

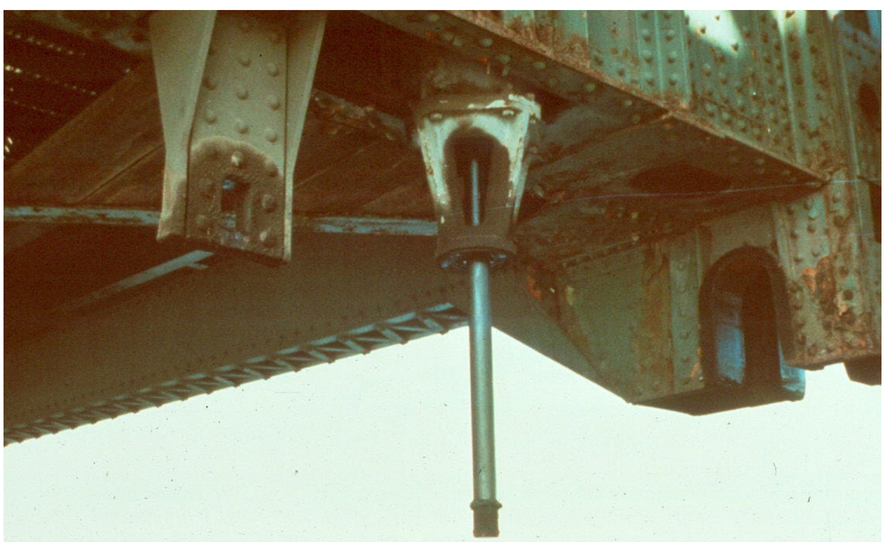


Figure 5.3.25 Air Buffer

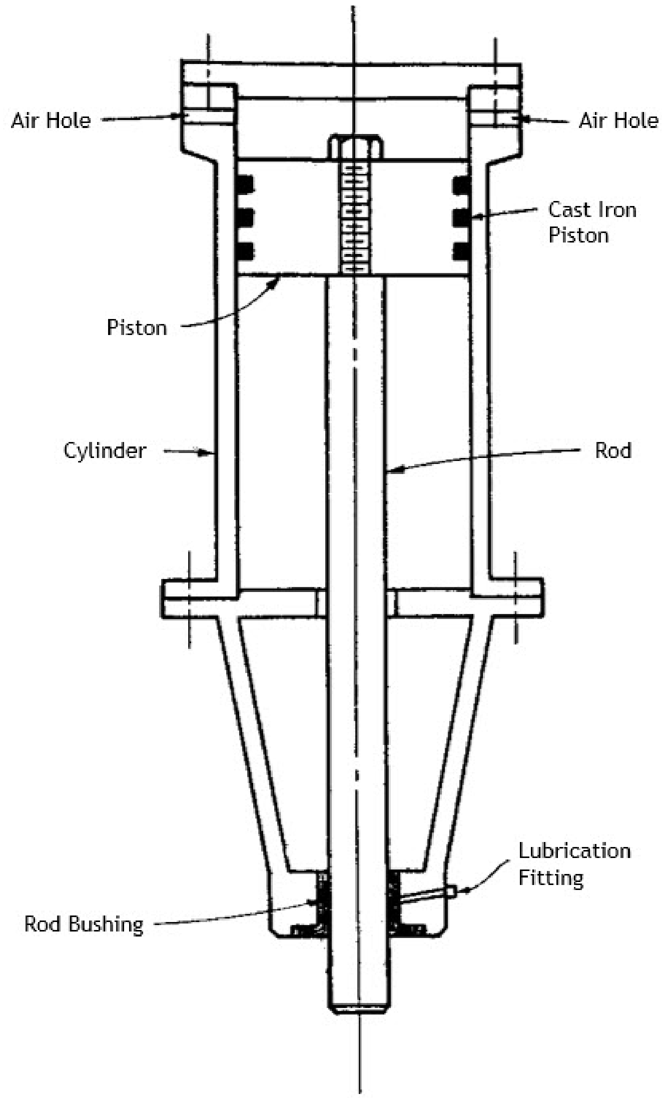


Figure 5.3.26 Typical Air Buffer Schematic

Span lock bars at the end of the span are set when the span is fully closed to prevent differential 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 mechanically or hydraulically (see Figure 5.3.27 and Figure 5.3.28).

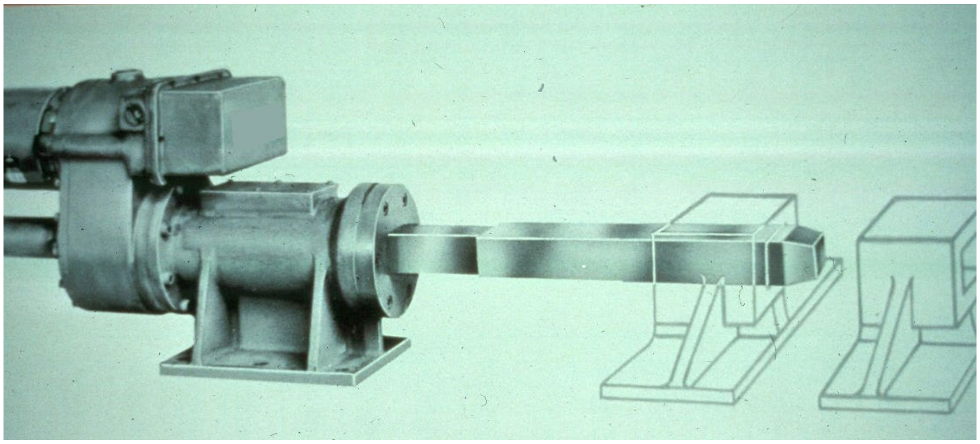


Figure 5.3.27 Mechanically Operated Span Lock

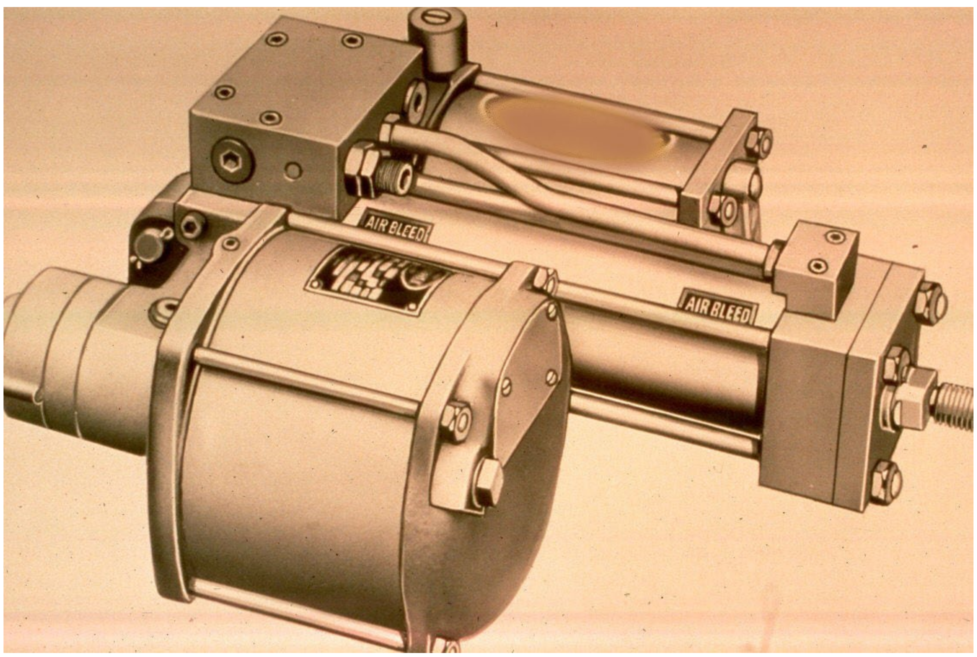


Figure 5.3.28 Hydraulically Operated Span Lock

Adjustment of the counterweight may be provided using modular counterweight blocks 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. A movable span is designed to function in a balanced condition, and seriously unbalanced conditions will most likely cause overstress or even failure of the mechanical or structural members.

Live load shoes and strike plates serve to transfer the live load from the movable leaf to the stationary part of the bridge. The live load shoe is mounted to the movable leaf or span and the strike plate is mounted to the fixed structure. The bottom of the shoe is curved to allow for some slight misalignment or deflection.

Because movable bridges require a stoppage of vehicle traffic to allow for waterway traffic, traffic safety devices, including signs and signals are often present on or adjacent to the bridge. The functionality of these are key to safe use of the bridge. Section 5.3.6 discusses traffic barriers and signals in further detail. In addition to signage and lighting for vehicular traffic, movable bridges often include signals and lighting for waterway navigation.

Swing Bridge Unique Features

Swing bridges may utilize the following features specific to their design:

- Pivot bearings.
- Balance wheels.
- Rim-bearing rollers.
- Wedges.
- End latches.

Pivot bearings are common in center-bearing type structures 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 5.3.29). In rim-bearing types, the pivot bearing is also enclosed but will most likely be radially loaded, maintaining the position of the pivot shaft or king pin.

Figure 5.3.29 Center Pivot Bearing

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 5.3.30). At other times, when the span is not subjected to unbalanced loads, a gap should be present between each wheel and the rail.

Figure 5.3.30 Balance Wheel in Place Over Circular Rack

Rim-bearing rollers are usually tapered to allow for the differential rolling distance between the inside and outside circumferences of the rail circle. Rim-bearing rollers are subject to full bearing at all times.

End wedges are typically used to raise the ends of the span and support traffic live loading (see Figure 5.3.31). The end wedge bearings are under all four corners of the span. Center wedges are typically 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 linkages forcing the wedges to actuate in union or each wedge may have its own actuator (see Figure 5.3.32).

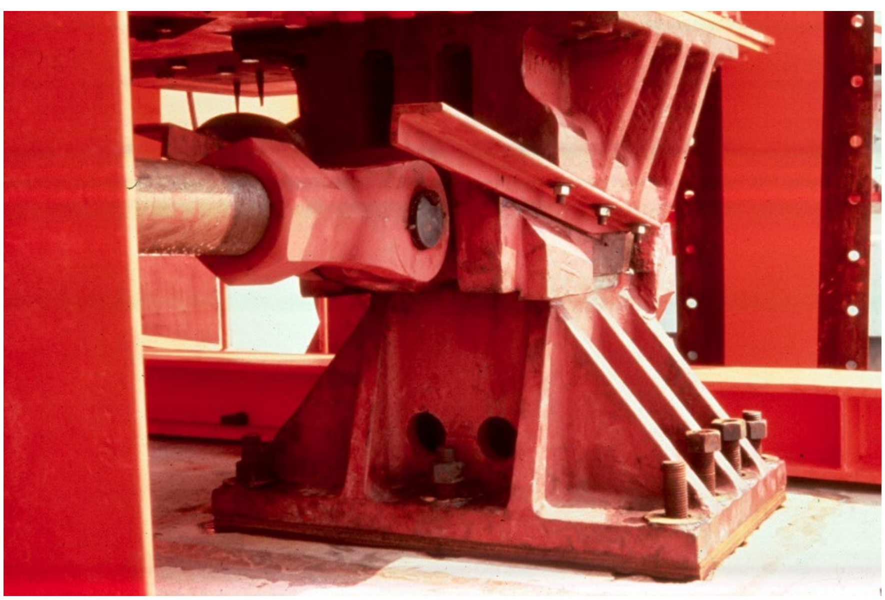


Figure 5.3.31 End Wedge

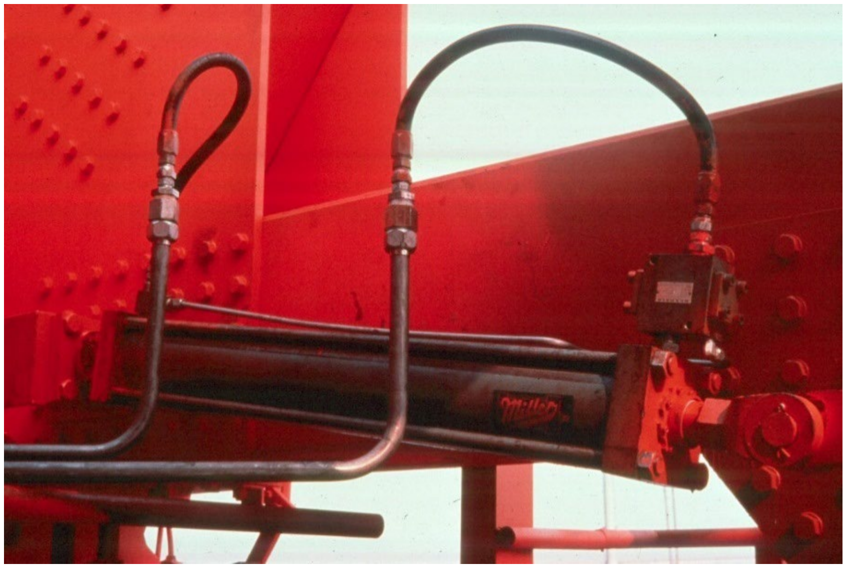


Figure 5.3.32 Hydraulic Cylinder Actuator

At the center of one or both rest piers, end latches generally consist of a guided tongue with roller mounted on the movable span which 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 5.3.33). As the span is swung open, the latch tongue can 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 can drop to center the span.

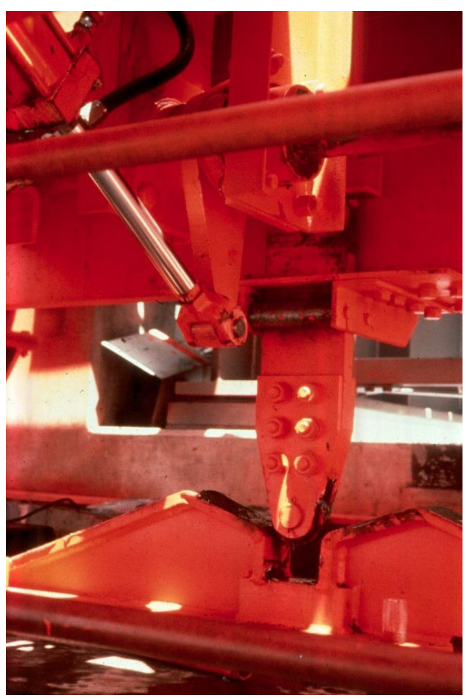


Figure 5.3.33 End Wedges Withdrawn and End Latch Lifted

Bascule Bridge Unique Features

Bascule bridges utilize the following features specific to their design:

- Rolling lift tread and track castings.
- Racks and pinions.
- Trunnions and trunnion bearings.
- Hopkins frames.
- Tail (rear) locks.
- Center locks.
- Transverse locks.

Rolling lift tread and track castings are rolling surfaces that support the bascule leaves as they roll open or closed (see Figure 5.3.34). 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.

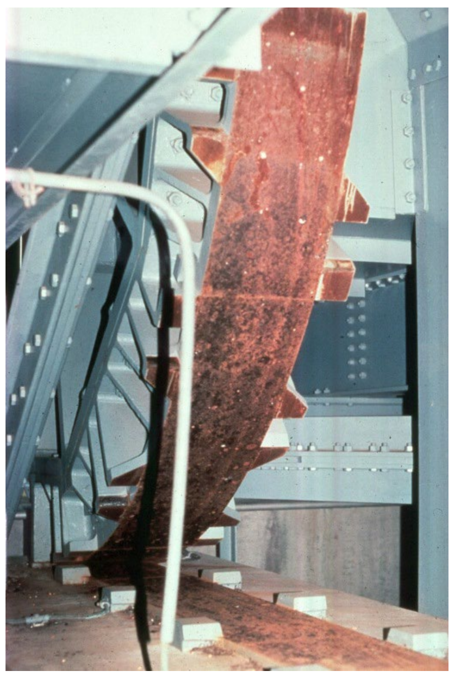


Figure 5.3.34 Circular Lift Tread and Track Castings

In the rolling lift rack and pinion, the driving pinion engages the rack teeth at the centerline of the roll (see Figure 5.3.35), whereas 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 Figure 5.3.36).

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.

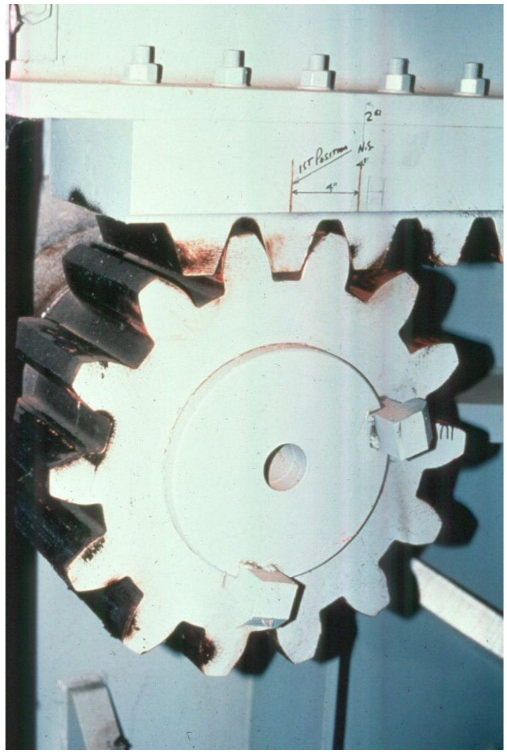


Figure 5.3.35 Rack Casting and Pinion

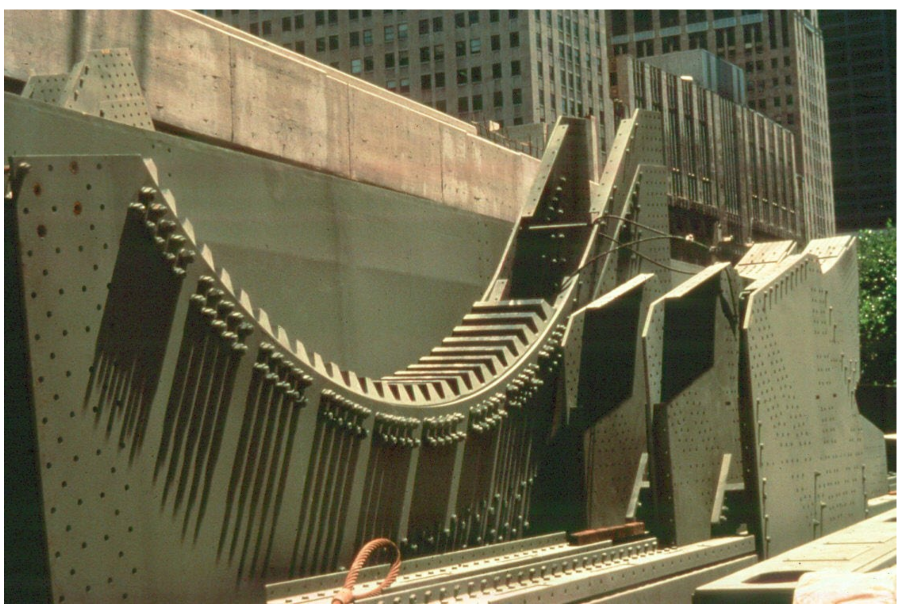


Figure 5.3.36 Rack Casting Ready for Installation

Trunnions and trunnion bearings (see Figure 5.3.37) 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 have the trunnions carrying live load in addition to dead load.

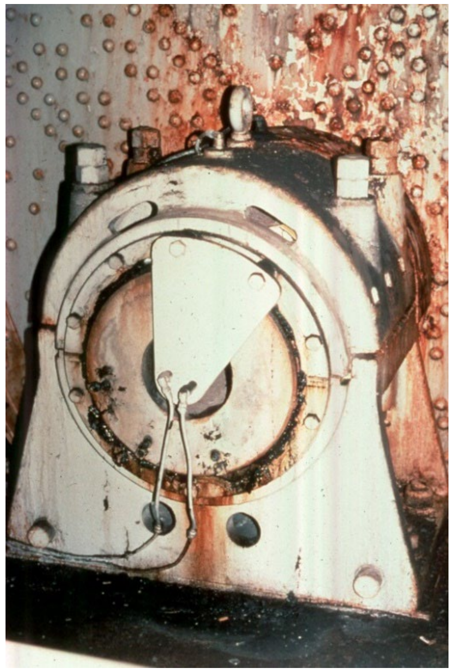


Figure 5.3.37 Trunnion Bearing

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.

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 5.3.38).

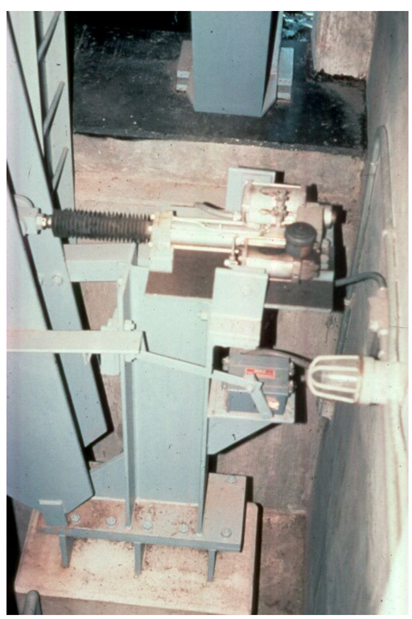


Figure 5.3.38 Rear Tail Lock Assembly

Center locks are provided to transfer shear load from one leaf to the other when the bridge is open to 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 5.3.39).

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, the center locks are engaged. These locking mechanisms are designed to transmit shear necessary to produce equal deflections at the mid-point under unbalanced transient loads. These center locks are not typically designed to carry superstructure moment.

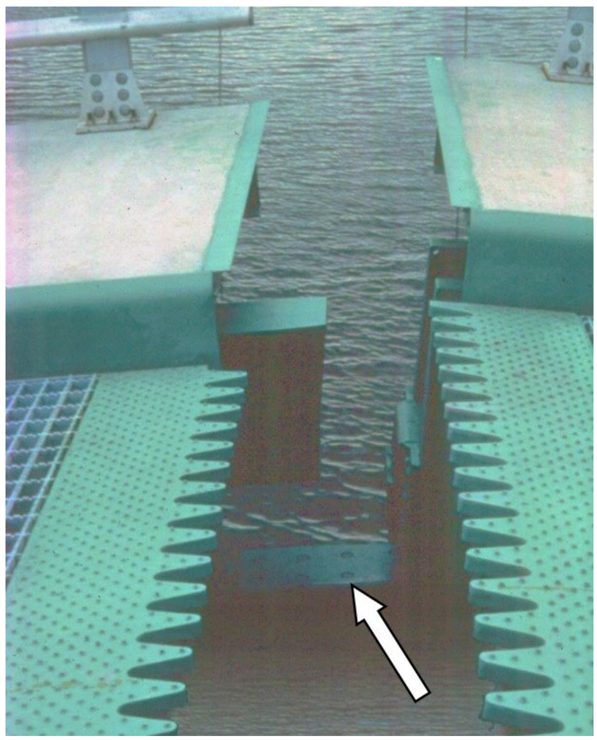


Figure 5.3.39 Center Lock

In twin bascule bridges that are split longitudinally to allow flexibility during construction, repair, or rehabilitation, transverse locks between the inside girders are typically used to keep the pairs connected during operation (see Figure 5.3.40). These are usually operated manually, as they are not typically released unless necessary for construction phasing or repair.

Figure 5.3.40 Engaged Transverse Locks

Vertical Lift Bridge Unique Features

Vertical lift bridges may utilize the following features specific to their design:

- Wire ropes and sockets.
- Drums, pulleys, and sheaves.
- Span and counterweight guides.
- Balance chains.
- Span leveling devices.

Wire ropes and sockets include up-haul and down-haul operating ropes and counterweight ropes (see Figure 5.3.41). 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.

Drums are generally used to wind a rope to extend or retract portions of the bridge (see Figure 5.3.42). Drums wind up the up-haul (lifting) ropes as they simultaneously unwind the down-haul ropes. Pulleys and sheaves change the direction of the rope or guide it at intermediate points between ends of the rope.

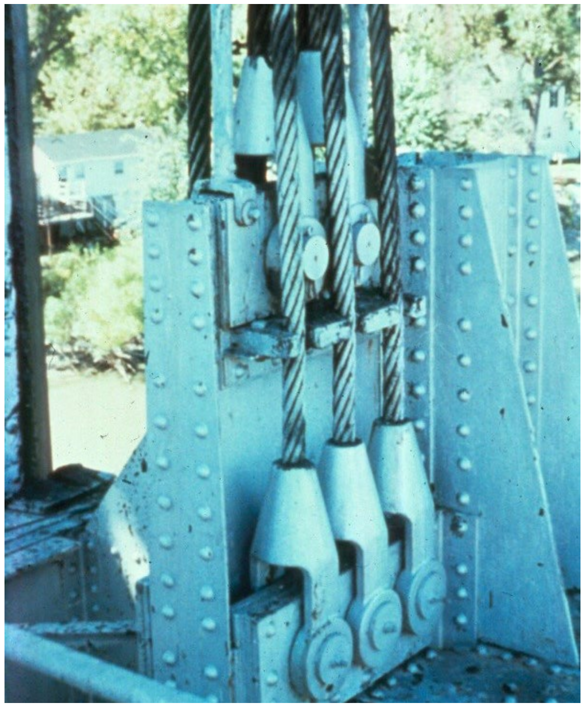


Figure 5.3.41 Wire Rope, Sockets, and Fittings

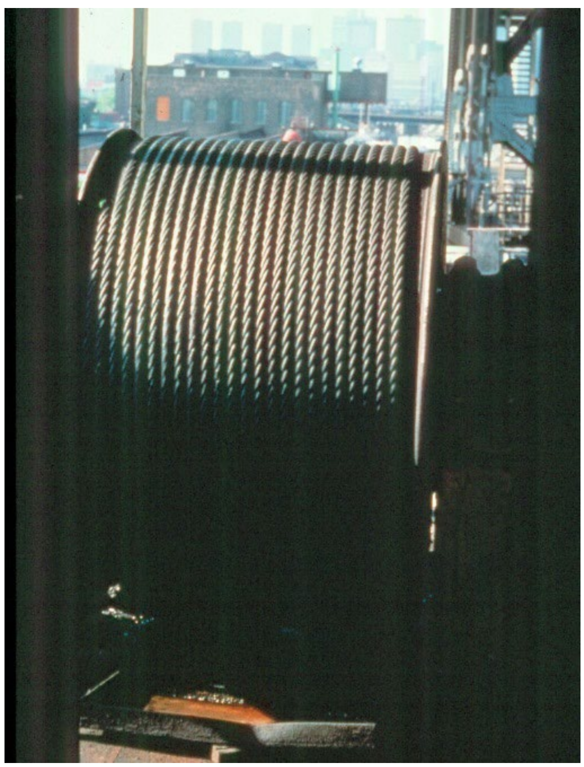


Figure 5.3.42 Drum with Wound Rope

Span and counterweight guides may be found between the tower and span or counterweight to prevent misalignment.

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.

Mechanical or electrical span leveling devices compensate and adjust the movement of the two ends of the span during operation to prevent unsynchronized movement.

5.3.6 Traffic Barriers and Signals

Traffic barriers are heavy-duty movable gates or posts that are designed to prevent a vehicle from plunging from the roadway into the channel or into the pit below the bridge (see Figure 5.3.43). These are redundant systems that might include gates, signals, and restraining cable barriers. The functionality of the traffic barriers, as well as traffic signals, warning lights or flashing beacons and auditory warnings, are important for public safety. They are typically used mainly in situations where a large opening exists between the approach span and the movable span when it is open. Additional barriers may also be present for pedestrian or bicycle traffic (see Figure 5.3.43).

Figure 5.3.43 Traffic Barrier and Signal

Section 5.4 Floating Bridges

5.4.1 Design Characteristics

Although uncommon, some states have floating bridges that are not supported by a substructure. They are supported by, or float on, the water and the bridge elevation will most likely change as the water level fluctuates (see Figure 5.4.1).

Floating bridges may be 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 100- to 200-ft deep water and a very soft bottom extending another 100 to 200 ft, a floating bridge is estimated to cost three to five time less than a conventional multi-span fixed bridge or 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.

Figure 5.4.1 Floating Bridge During Stormy Weather

Floating bridges take advantage of the natural law of buoyancy of water to support the loads. This is achieved using pontoons secured into place by an anchoring system to support the superstructure. 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 on traditional foundations for vessels to pass through, or an elevated span for vessels to pass under (see Figure 5.4.2 and Figure 5.4.3).

Figure 5.4.2 Movable Bridge Section

Figure 5.4.3 Floating Bridge and Movable Bridge Elevation

5.4.2 Pontoons

Floating bridges may be constructed of wood (see Figure 5.4.4), concrete, steel, or a combination of materials depending on the design demands. Concrete pontoons are generally used in newer bridges.

The pontoons are large, water-tight chambers constructed off site and floated into place (see Figure 5.4.5 and Figure 5.4.6). 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. Prestressed concrete or reinforced concrete may be used. The pontoons are held into place by steel cables anchored deep below the mud line. Bridge pontoons are designed to safely withstand wind and wave forces, major storms, and vessel collisions.

Figure 5.4.4 Floating Timber Bridge

Figure 5.4.5 Concrete Pontoons Under Construction

Figure 5.4.6 Concrete Pontoons being Transported to Site

To control water leaking into the interior of the pontoons and ultimately sinking the bridges, each pontoon contains several watertight cells. This confines any flooding to a small area of the pontoon. 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.

Pontoons are generally classified as continuous pontoon type or separate pontoon type. Continuous pontoon bridges are made of individual pontoons longitudinally connected to each other. The roadway may be placed directly on top of the pontoons or a superstructure may be built on top of the pontoons. The size of each pontoon is determined by design demands as well as constraints imposed by the construction facilities and the transportation route to the bridge site.

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. The superstructure should be 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 the mud line.

5.4.3 Anchorage Systems

Floating bridges can be held in place in a variety of ways, but frequently include a combination of piles, caissons, cables, anchors, and fixed guide structures. The most common type of system consists of cables and anchors. Anchor cables can be several inches in diameter and consist of many individual steel strands (see Figure 5.4.7).

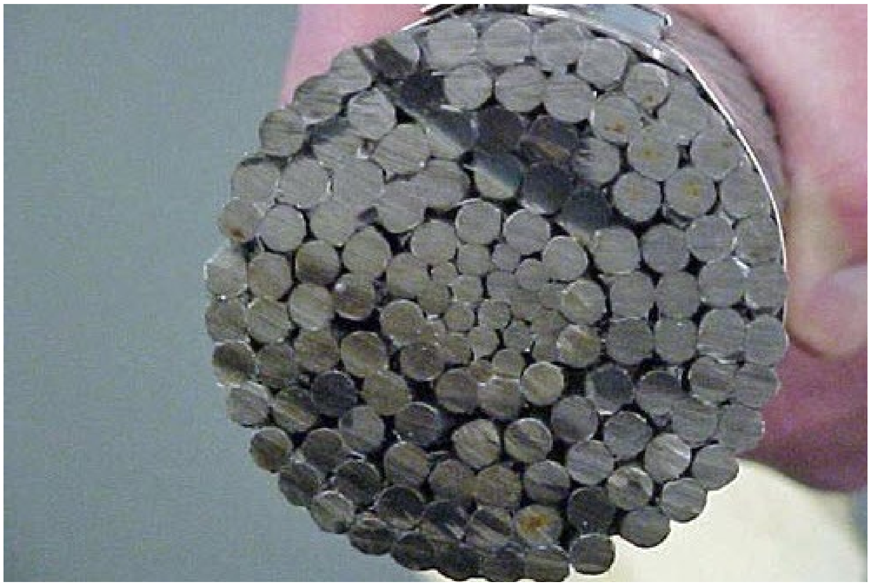


Figure 5.4.7 Cross-Section of Anchor Cable

Anchor cable saddles are typically used within the pontoon to guide and hold the cable in place (see Figure 5.4.8). Hydraulic jacks are placed inside the pontoon to tighten or release the pressure on the cables as the water level fluctuates under the bridge.

Figure 5.4.8 Anchor Cable Saddle

Depending on the depth of the water and the soil conditions, there are four primary types of anchoring systems used in floating bridges:

- Precast concrete fluke-style anchor.
- Pile anchor.
- Open-cell gravity block anchor.
- Solid gravity stackable slab anchor.

Precast concrete fluke-style anchors are typically 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.

Pile anchors are designed for use in water depths less than 88 ft and with hard soil. Piles are driven into the surface to a specified depth and tied in union to increase capacity.

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 of anchor is used in deep water where the soil is hard.

Solid gravity slab anchors are a gravity type of anchor. They can be used in 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 necessary 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.

Section 5.5 Ferry Transfer Bridges

5.5.1 Design Characteristics

Ferry transfer bridges allow vehicles to board a ferry from the roadway (see Figure 5.5.1). Although rare, this type of bridge is inventoried because it may carry a public highway from land onto a ferry. Because the elevation of the ferry deck varies depending on the water level, ferry ramps have movable components to raise or lower the bridge deck. To enable mobility and control the traffic entering the ferry, there may be mechanical, hydraulic, and/or electrical equipment associated with the bridge. The equipment may be similar to that presented in Section 5.3.

Figure 5.5.1 Elevation View of Ferry Transfer Bridge

It is common for these structures to be made of steel, as it is easier to lift or lower the bridge. Since a ferry can only accommodate a limited number of cars, it is not necessary for ferry ramps to be very long or wide (see Figure 5.5.2). The ferry boats generally have a weight limit as well, so it is not necessary for the ramps to have a heavy load capacity.

Figure 5.5.2 Ferry Transfer Bridge Steel Grid Deck

Although the structure is basically a multi-beam bridge, the bearings on ferry transfer bridges differ from those of a standard bridge. Ferry ramps have fixed pin bearings on the shore end of the bridge, which the deck rotates about when lifted (see Figure 5.5.3). The other end of the bridge bears on the ferry boat when docked; therefore, the bearings should be attached to the beams. This can be achieved with a fixed shoe (see Figure 5.5.4) or something similar on the vessel/ferry side.

Figure 5.5.3 Fixed Pin Bearing

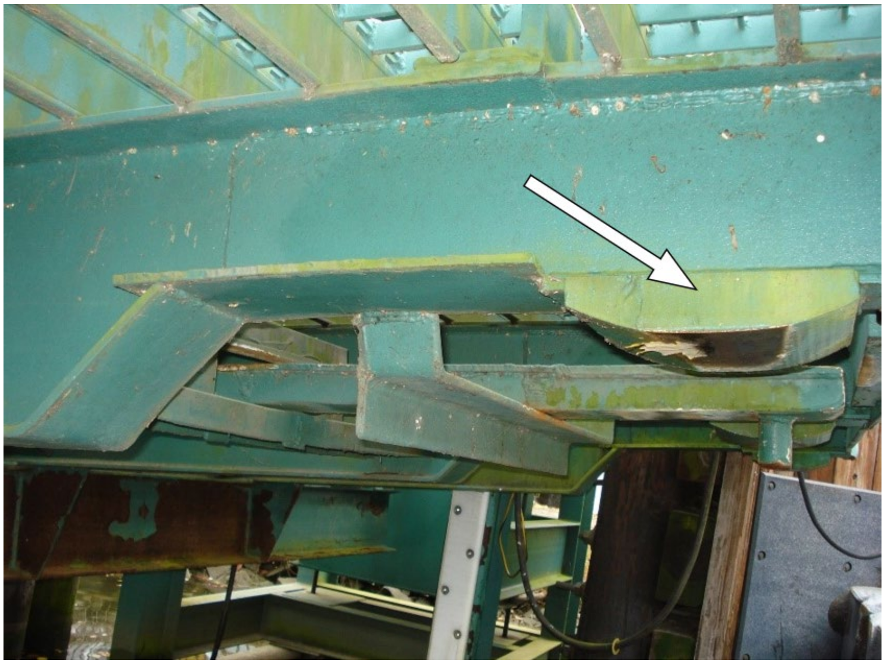


Figure 5.5.4 Fixed Shoe Bearing

It is typical for the superstructure to include a lifting girder, which is a transverse member that is used to raise and lower the span (see Figure 5.5.5). The lifting girder is connected to the lifting mechanism, which can be a cable and counterweight system with hoist towers, a hydraulic system, or another type.

Figure 5.5.5 Lifting Girder

PART II – BRIDGE, CULVERT, AND WATERWAY CHARACTERISTICS

CHAPTER 6 TABLE OF CONTENTS

CHAPTER 6 LIST OF FIGURES

CHAPTER 6 LIST OF TABLES

Basic Equations of Bridge Mechanics

$$f_a = \frac{P}{A} \qquad \sigma = \frac{F}{A}$$

$$f_b = \frac{Mc}{I} \qquad \varepsilon = \frac{\Delta L}{L}$$

$$f_v = \frac{V}{A_w} \qquad E = \frac{\sigma}{\varepsilon}$$

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
P	=	force
σ	=	stress
V	=	vertical shear force due to external loads
ΔL	=	change in length
ε	=	strain

Common units:

lbs.	=	pounds
in	=	inches
ft	=	feet
k	=	kip
psi	=	pounds per square inch
ksi	=	kips per square inch

Chapter 6 Mechanics

Section 6.1 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. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

The two most important reasons for a bridge inspector to understand 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.

A culvert is a type of bridge for purposes of coding and bridge inventory in many cases, but the structure behaves differently with regards to mechanics. Culvert mechanics utilize the same basic principles as bridge mechanics, but the way the load is applied differs in some ways. Refer to Section 6.3 for further information on culvert mechanics.

Section 6.2 Bridge Mechanics

6.2.1 Bridge 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 typically considered concentrated loads.

A distributed load can be applied to a section or the entire member, and the amount of load per unit of length is generally constant. The weight from superstructure self-weights, bridge decks, wearing surfaces, and bridge parapets produce distributed loads. Secondary loads, such as wind, stream flow, earth cover and ice, are also usually considered 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*, as supplemented by agency criteria as applicable. During the 1990s 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 Section 6.2.6.

Bridge design loadings can be divided into two principal categories: permanent loads and 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 6.2.1). 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.

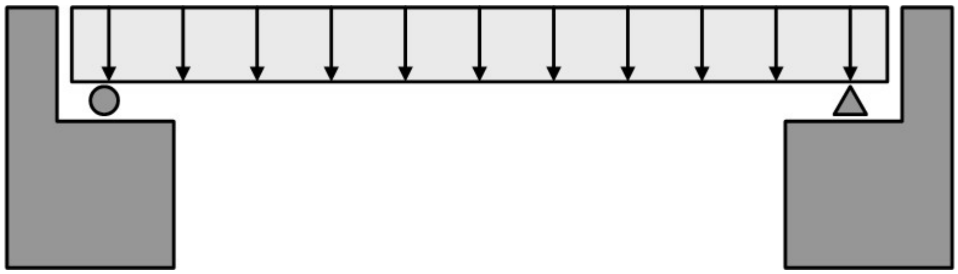


Figure 6.2.1 Permanent Load on a Bridge

Dead loads are a static load due to the weight of the structure itself. Any bridge feature may or may not contribute to the strength of the structure. The 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 can be a combination of horizontal (EH) or vertical (EV) loads, which can be very large and 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 ft long beam weighs 50 lbs./ft. The total weight of the beam is 1000 lbs. This load 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.

Primary Transient Loads

A transient load is a temporary load and force that is applied to a structure that changes over time. In bridge applications, transient loads are commonly referred to as live loads and are a result of moving vehicular or pedestrian loads (see Figure 6.2.2). Standard AASHTO vehicle live loads do not represent actual vehicles, but they are a good approximation for bridge design and rating. AASHTO has designated standard pedestrian loads for design of sidewalks and other pedestrian structures.

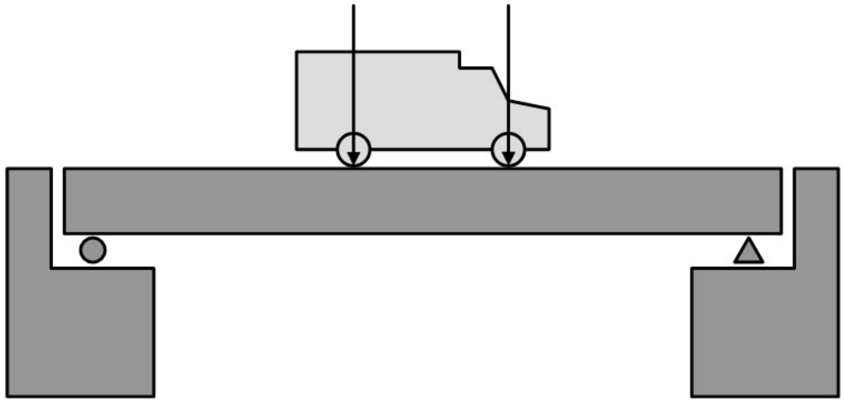


Figure 6.2.2 Vehicle Transient Load on a Bridge

To account for the effects 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.

The *Specifications for the National Bridge Inventory (SNBI)* uses the Design Load, Item B.LR.01 to record the most restrictive design load for a bridge. The coding options include several AASHTO live load vehicles (*SNBI* Subsection 5.1).

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 *AASHTO Standard Specifications for Highway Bridges*. A third type of loading is used for AASHTO Load and Resistance Factor Design (LRFD) and Rating (LRFR).

The first type is a single unit vehicle with two axles spaced at 14 ft and designated as a highway truck or "H" truck (see Figure 6.2.3). The weight of the front axle is 20 percent of the gross vehicle weight. The weight of the rear axle is 80 percent of the gross vehicle weight. The "H" designation is followed by the gross tonnage of that 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 for Highway Bridges*. Since 2007, highway bridges have been primarily designed in accordance with AASHTO *LRFD Bridge Design Specifications* using the HL-93 loading (FHWA Memorandum *Bridge Load Ratings for the National Bridge Inventory*).

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 ft 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 generally 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.

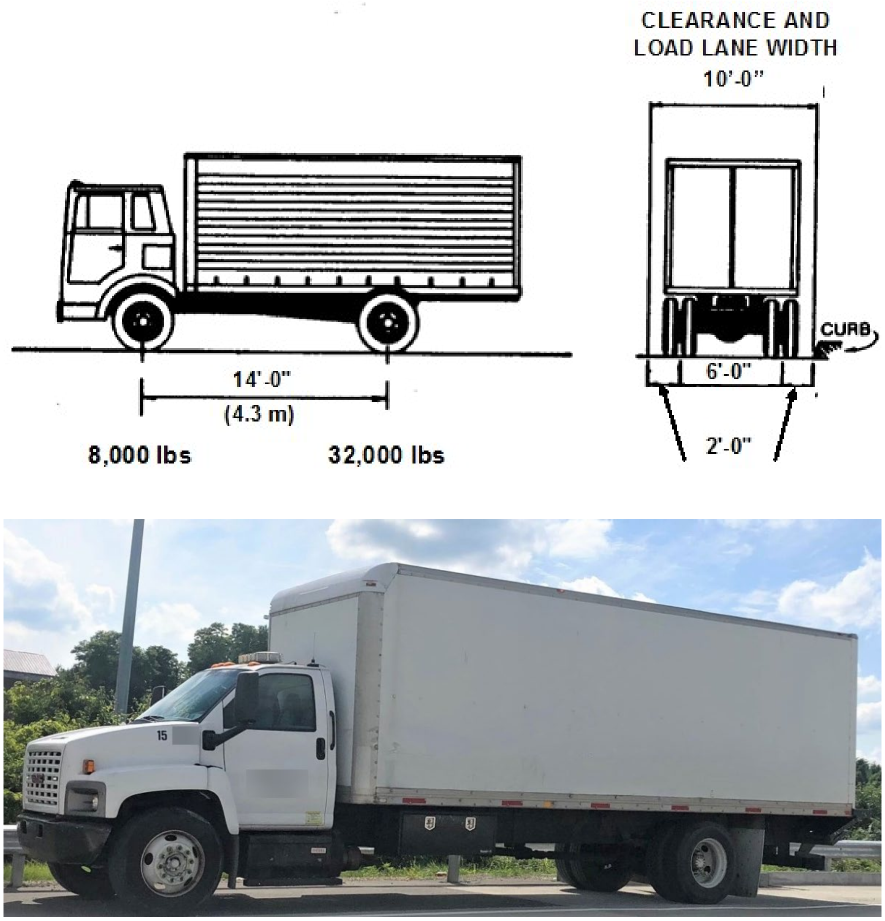


Figure 6.2.3 AASHTO H20 Design Truck and Similar Actual Truck

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 6.2.4).

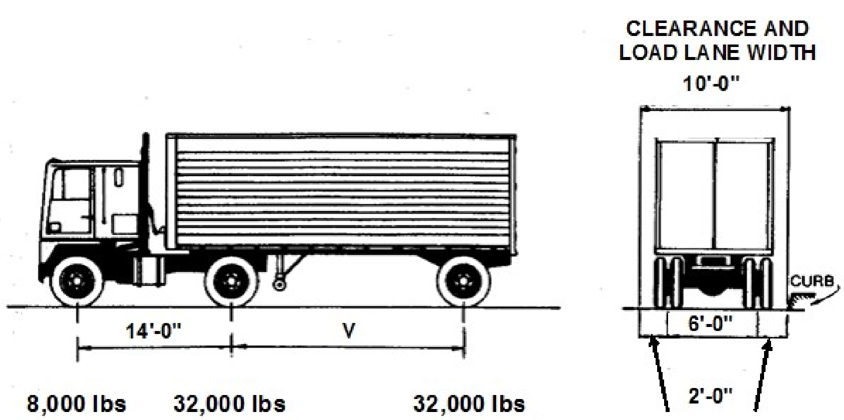


Figure 6.2.4 AASHTO HS20 Design Truck

The tractor weight and axle 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 ft. The “HS” designation is followed by a number indicating the gross weight in tons of the tractor only.

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 before 1944, a standard loading of H15 was used. In 1944, the policy of affixing the publication year of design loadings was adopted. In specifications before 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 “umbrella” loads. Like the notional load, these loads were developed based on research to envelope the actual vehicles that typically travel across structures. 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 were used for the design of bridge members. Depending on items such as highway classification, truck usage, and span classification, 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. Load rating methods will be discussed further in Section 6.2.7.

AASHTO Lane Loadings

In addition to the standard truck loadings, a system of equivalent lane loadings was developed to provide a simple method of calculating bridge response to a series, or “train” of trucks. Lane loading consists of a uniform load per linear foot of traffic lane combined with a concentrated load on the span to produce the most critical situation in the structure (see Figure 6.2.5). Two concentrated loads are used for negative moment in continuous spans. (Reference *AASHTO Standard Specifications for Highway Bridges 17th Edition*; *Article 3.7*).

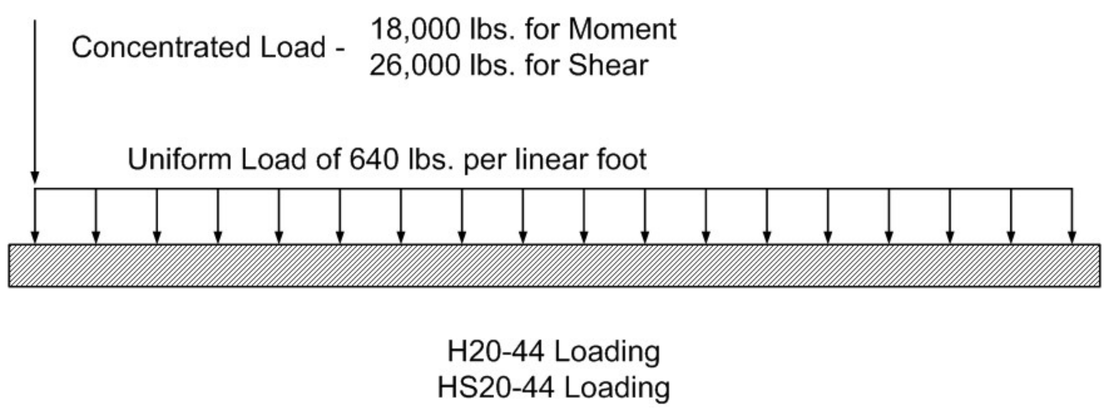


Figure 6.2.5 AASHTO Lane Loadings

For design and load capacity rating analysis, the engineer investigates both a truck loading and a lane loading, including the concentrated load, 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.

Alternate Military Loading

The Alternate Military Loading is a single unit vehicle with two axles spaced at 4 ft and weighing 12 tons (or 24 kips) each (see Figure 6.2.6). It has been part of the AASHTO *Standard Specifications for Highway Bridges* since 1977. Bridges on interstate highways or other highways which are potential defense routes are designed for whichever loading produces the greatest stress.

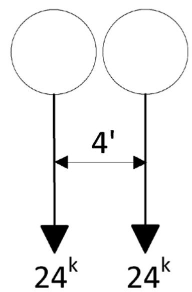


Figure 6.2.6 Alternate Military Loading

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 a lane load and applied to each loaded lane. Reference AASHTO *LRFD Bridge Design Specifications 9th Edition*; Article 3.6.1.2.

The LRFD design truck is 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 ft apart. The transverse wheel spacing of the truck and tandem is 6 ft.

The magnitude of the HL-93 lane load is equal to that of the HS-20 lane load. The lane load is 640 lbs./ft longitudinally and it is distributed uniformly over a 10 ft 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 6.2.7).

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.

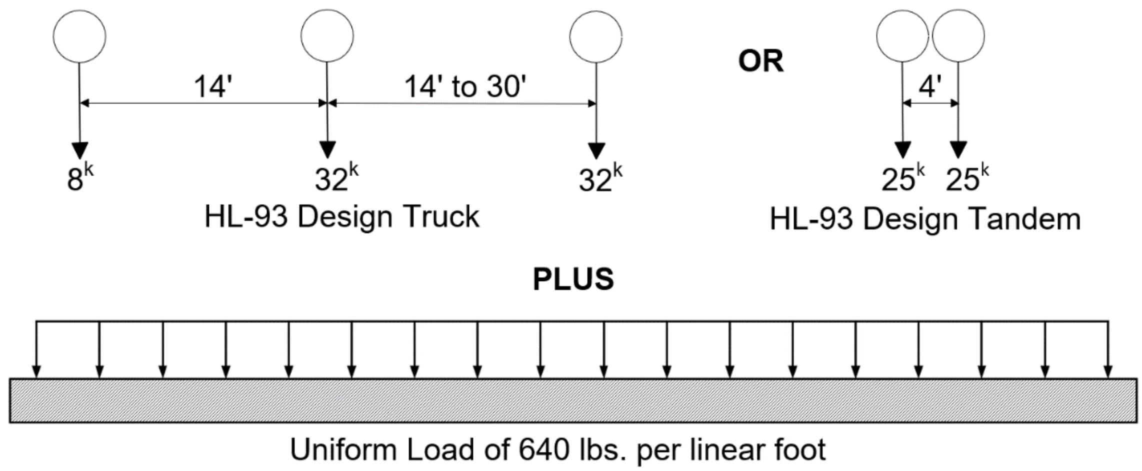


Figure 6.2.7 AASHTO LRFD Design Loading

Permit Vehicles

Permit vehicles are oversize/overweight vehicles that, in order to travel a state's highways, apply for a permit from that state agency. They are usually heavy trucks (e.g., combination trucks, construction vehicles, electrical generators, or cranes) that have varying axle weights and spacings depending upon the design of the individual truck (see Figure 6.2.8). 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 gross weight, number of axles, axle spacing, and maximum axle weights for a designated route.

Figure 6.2.8 Permit Vehicle

Legal Vehicles/Loads

Legal load is the maximum load permitted by law for each vehicle configuration in the state where the bridge is situated. Title 23 of the United States Code (U.S.C.), Section 127 stipulates the vehicle size and weight limits of trucks legally operating on interstate highways, and state statute sets the truck size and weight limits for legal vehicles on other highways in the state. Legal load ratings help establish a need for posting or bridge strengthening.

There are also specialized hauling vehicles, emergency vehicles, and farm equipment that can legally travel and are exempt from attaining a permit, due to the nature of the vehicle use. These types of vehicles should be considered as well.

Other Vehicles

Other vehicle types are applied to the bridge to establish the load ratings. AASHTO recommended load rating vehicles include:

- Type 3.
- Type 3-S2.
- Type 3-3.
- Notional Rating Load (NRL).
- Single Unit 4 Truck (SU4).
- Single Unit 5 Truck (SU5).
- Single Unit 6 Truck (SU6).
- Single Unit 7 Truck (SU7).
- The maximum legal load vehicles of the state.
- State routine permit loads.
- FAST Act vehicles (EV2/EV3).

The axle spacing and weights of the routine AASHTO legal vehicles - Type 3, Type 3-S2, and Type 3-3 units are based on actual vehicles (see Figure 6.2.9). However, as described previously, the H, HS, and HL-93 loadings do not represent actual vehicles. The HS and HL loadings are still used today in reporting load ratings to the National Bridge Inventory (NBI).

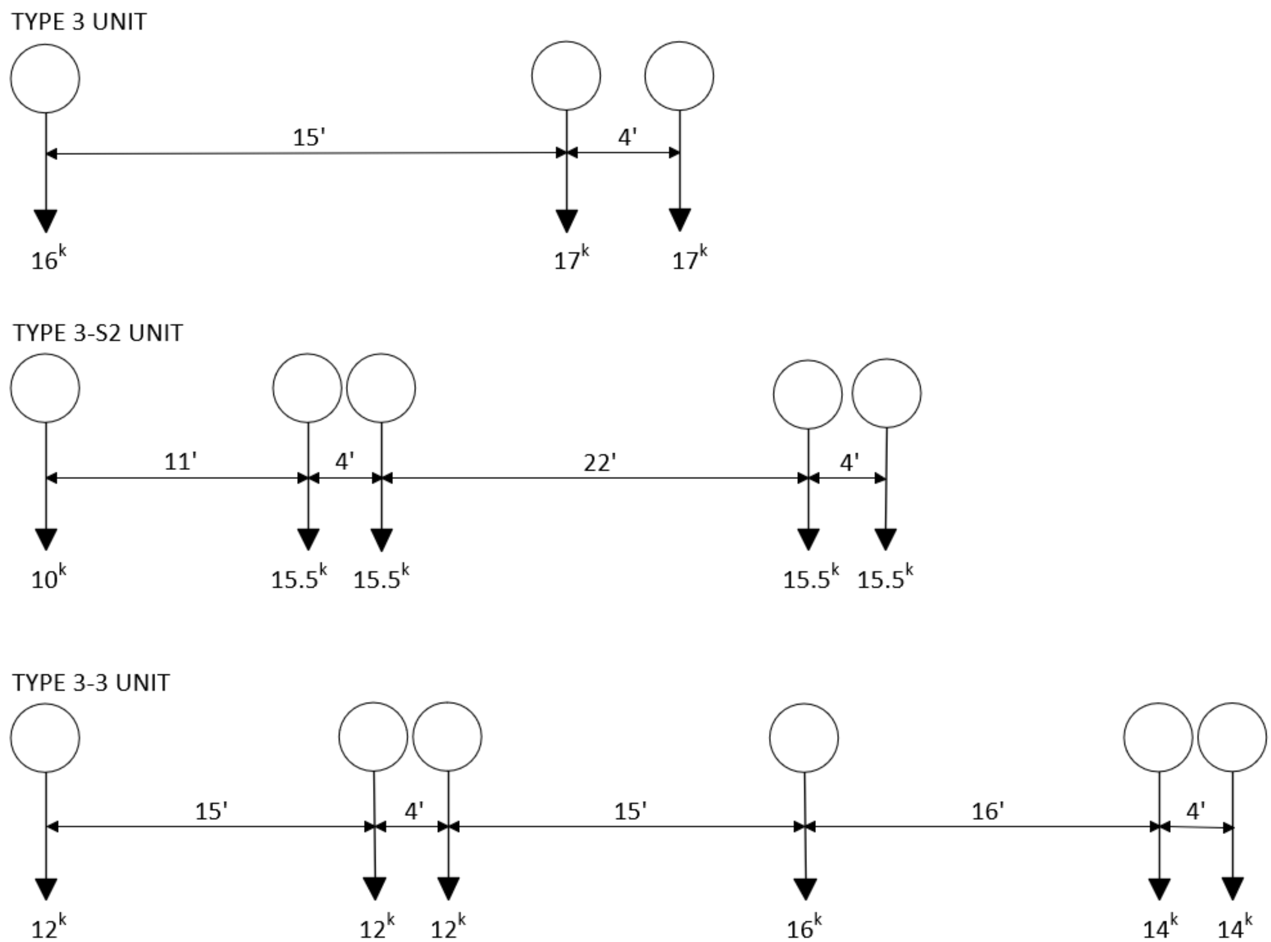


Figure 6.2.9 AASHTO Legal Vehicles

Like H and HS vehicles, a Notional Rating Load (see Figure 6.2.10) is an envelope loading that was developed to simulate the load effects of a Special Hauling Vehicle (SHV). The Single-Unit Bridge Posting Loads (see Figure 6.2.11) were also developed to imitate the variety of special hauling loads which are now being hauled on a regular basis. The four SHVs are also considered AASHTO legal loads.

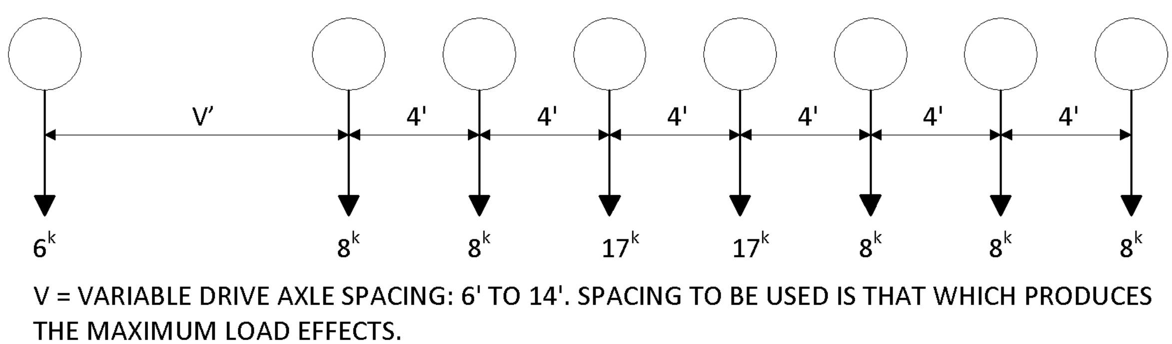


Figure 6.2.10 Notional Rating Load

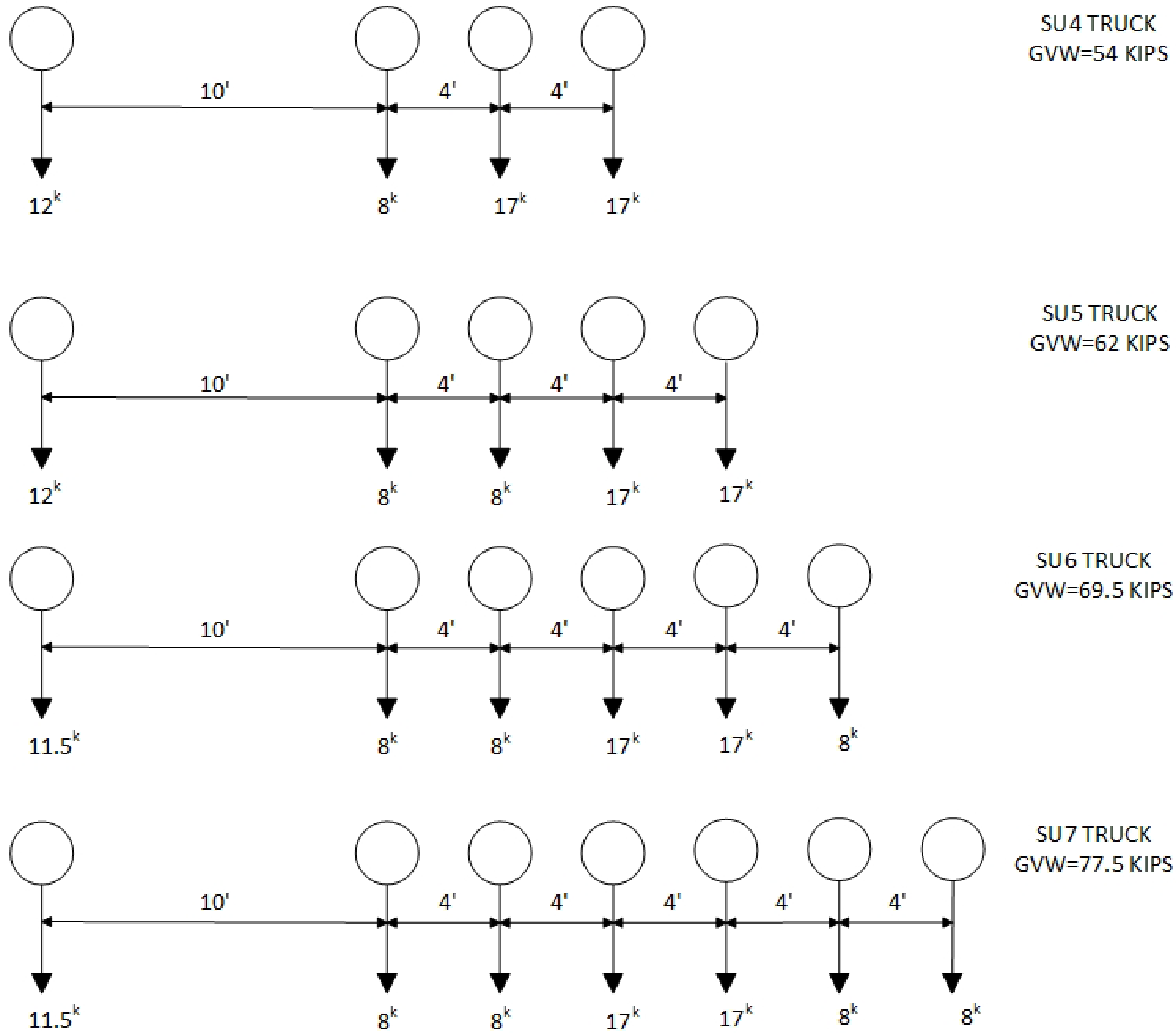


Figure 6.2.11 Single-Unit Bridge Posting Loads

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.

The Fixing America's Surface Transportation Act (FAST Act) was signed into law in 2015 and includes new truck size and weight provisions that affect bridge load rating and posting. The act provides an exemption for emergency vehicles (EV), allowing them to surpass the nationwide interstate weight limits. The Federal Highway Administration (FHWA) has determined that, for load rating, two emergency vehicle configurations produce load effects in typical bridges that envelope the effects resulting from the typical emergency vehicles covered by the FAST Act: EV2 and EV3 (see Figure 6.2.12). Bridges should be posted according to the emergency vehicles if they govern the load ratings.

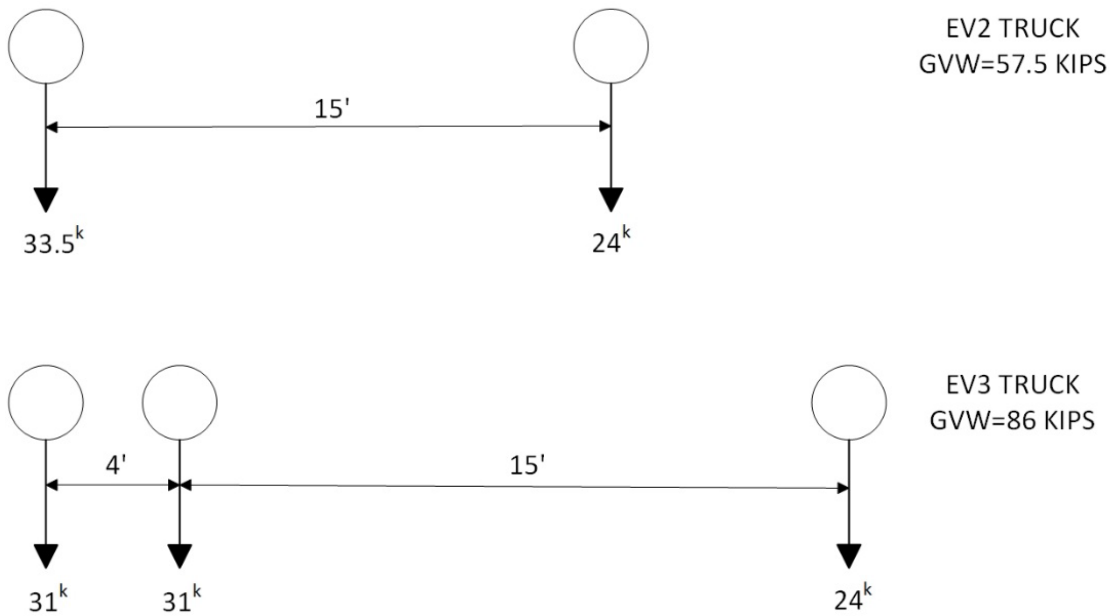


Figure 6.2.12 FAST Act Emergency Vehicles

Secondary Transient Loads

In bridge applications, the secondary transient loads are temporary loads and can consist of the following:

- Vehicular braking force - a force parallel to traffic 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 the superstructure or substructure of a bridge.
- Vessel collision (allision) force - the force caused by the collision of a water vessel into the superstructure or substructure of a bridge.
- Earthquake load - seismic loading in all directions, against bridge members.
- Friction load - the force 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 members.
- Vehicular dynamic load allowance - loads that account for vibrations and resonance of the bridge due to live load, and vibrations due to surface discontinuities (i.e., deck joints, potholes, cracks).
- Live load surcharge - a load where vehicular live load is expected on the surface of backfill within a distance of 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, including sidewalks and other structures.
- Force effects due to settlement - a force acting on earth-retaining substructure units, such as abutments and retaining walls.

- Temperature – since common bridge materials expand as temperature increases and contract as temperature decreases, the forces caused on bridge members by these dimensional changes.
- Water loads - forces acting on bridge members within flowing water, including horizontal forces from static or stream pressure and uplift forces due to buoyancy.
- Wind load on structure - wind pressure on the exposed area of a bridge.
- Wind load on live load - wind effects transferred through the live load vehicles on a bridge.

A bridge may be subjected to several of these loads simultaneously. AASHTO *LRFD Bridge Design Specifications* have established a table of load combinations and load factors. For each limit state, a load combination is defined and considered with individual load factors to be applied to each load. (Reference *AASHTO LRFD Bridge Design Specifications 9th Edition, Table 3.4.1-1 – Load Combinations and Load Factors*.)

6.2.2 Basic Terminology

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 6.2.13). Every force can be divided into three 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.

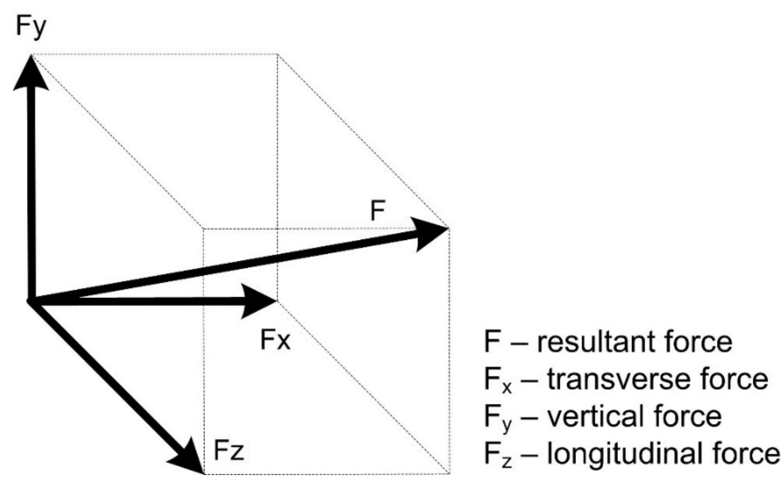


Figure 6.2.13 Basic Force Components

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 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 (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 in^2, then the stress caused by this force is 3000 psi (or 3 ksi).

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-ft-long column causes an axial deformation of 0.002 ft, then the resulting axial strain is 0.002 ft divided by 20 ft, or 0.0001 ft/ft. This strain can also be expressed simply as 0.0001 (with no units) or as 0.01 percent.

Stress-Strain Relationship

For most structural materials, values of stress and strain are directly proportional (see Figure 6.2.14). However, this proportionality exists only up to a specific 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, plastic deformation occurs, and strain is not directly proportional to a given applied stress. The material property that defines a material's stress-strain relationship, is called the modulus of elasticity, or Young's modulus.

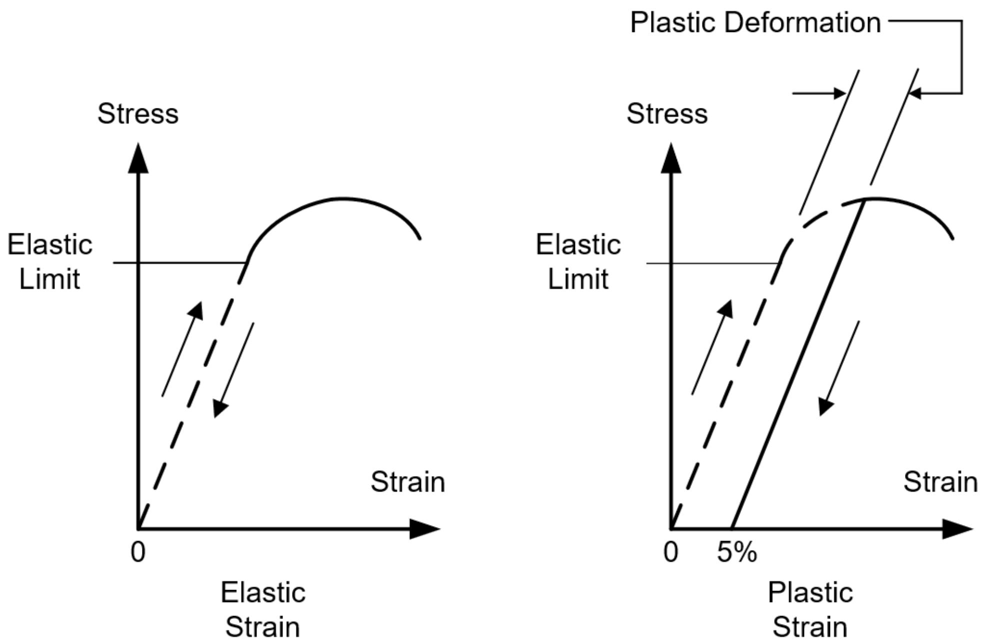


Figure 6.2.14 Stress-Strain Diagram

Modulus of Elasticity

Each material has a unique modulus of elasticity that 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 if 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).

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.

$$E = \frac{2{,}900\ psi}{0.0001\ inch/inch} = 29{,}000{,}000\ psi = 29{,}000\ ksi$$

This is approximately equal to the modulus of elasticity for steel. The modulus of elasticity for concrete is approximately 3000 to 4500 ksi, whereas for commonly used grades of timber it is approximately 1600 ksi.

Deformation

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

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 transient loads are removed.

Example of elastic deformation: A stretched rubber band should 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 due to strain. 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 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 typically associated with elastic deformation. Creep is 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 untouched on a thin wooden shelf for several months, the shelf will likely gradually deflect and change in shape. This deformation is due to the sustained application of a constant 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. Any thermal changes in these members should be considered since they can cause significant stresses.

Bridge materials expand as temperature increases and contract as temperature decreases. The amount of thermal deformation in a member may depend on:

- The 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.

Overloads

Overload damage may occur when members are overstressed. Overload occurs when the stresses applied are greater than the elastic limit for the material, which may result in irreversible deformation. Buckling or crushing may be an indication of overloading in compression. Permanent member elongation or cracking may be an indication of overloading in tension.

Buckling

Buckling is the tendency of a member to crush or bend out-of-plane when subjected to a compressive force that may be well below the elastic limit. As the length and slenderness of a compression member increases, the likelihood of buckling also increases. Additional cross-sectional area or bracing to resist buckling are necessary for compression members.

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 elastic or plastic.

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

Ductility and Brittleness

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

Example of ductility: Pizza dough can be stretched a great deal before it will break into two sections. When the dough finally does break, it will have a greatly reduced cross-sectional area.

Structural materials for bridges that are generally considered ductile include:

- Steel.
- Aluminum.
- Wood.

Brittle, or non-ductile, materials will likely 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, as 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 considered 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. However, 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.

Refer to Chapter 7 for a description of fatigue categories for various steel details.

6.2.3 Bridge Response to Loadings

Each member of a bridge is designed to respond to loads in a particular way. It is important to understand how loads are applied to each member in order to evaluate if it is functioning as intended. An inspector should understand a bridge member's normal response to loadings, so they can 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

For an object such as a bridge to be stable, the forces must be equal and opposite; therefore, a force on a bridge must be met with an equal and opposite force to remain static. In calculating forces and reactions, the analysis is governed by equations of equilibrium. Equilibrium equations represent a balanced force system and may be expressed as:

$$\Sigma V = 0$$
$$\Sigma H = 0$$
$$\Sigma M = 0$$

Where: Σ = summation of
V = vertical forces
H = horizontal forces
M = moments (bending forces)

Axial Forces

An axial force is a push or pull type force that 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 6.2.15). Axial forces are generally expressed in English units of pounds or kips.

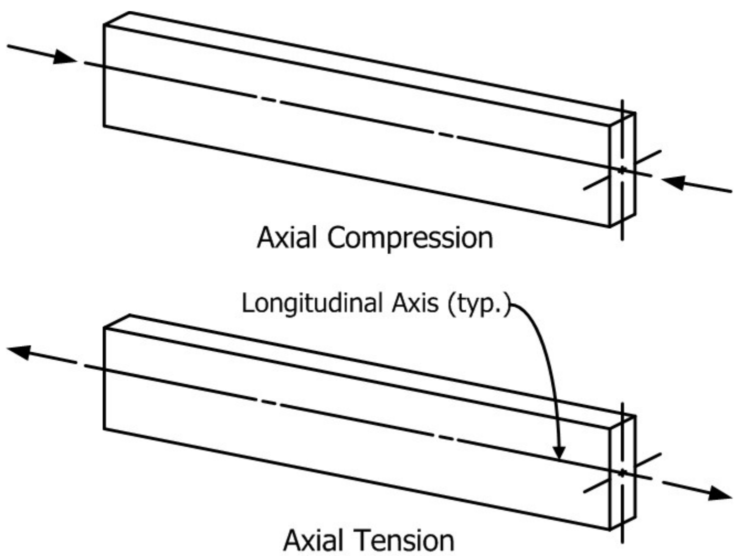


Figure 6.2.15 Axial Forces

Examples of axial forces: A person 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 members that carry axial loads. They are designed for compression or tension forces. Cables in a cable-supported bridge are designed for axial forces in tension. A vertical load on a pile is an axial force in compression.

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 likely vary depending on the magnitude of the force, whether the force is tensile or compressive, and the type of material used.

For tension and compression members, the cross-sectional area must satisfy the previous equation for a tolerable axial stress. However, the tolerable axial compressive stress is generally lower than the tolerable tensile stress because of a phenomenon called buckling, or a failure by an inelastic change in alignment.

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 that causes a member to bend. The greatest bending moment that a beam can resist is generally the governing factor that 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 6.2.16). Moments are generally expressed in English units of lb.-ft. or k-ft.

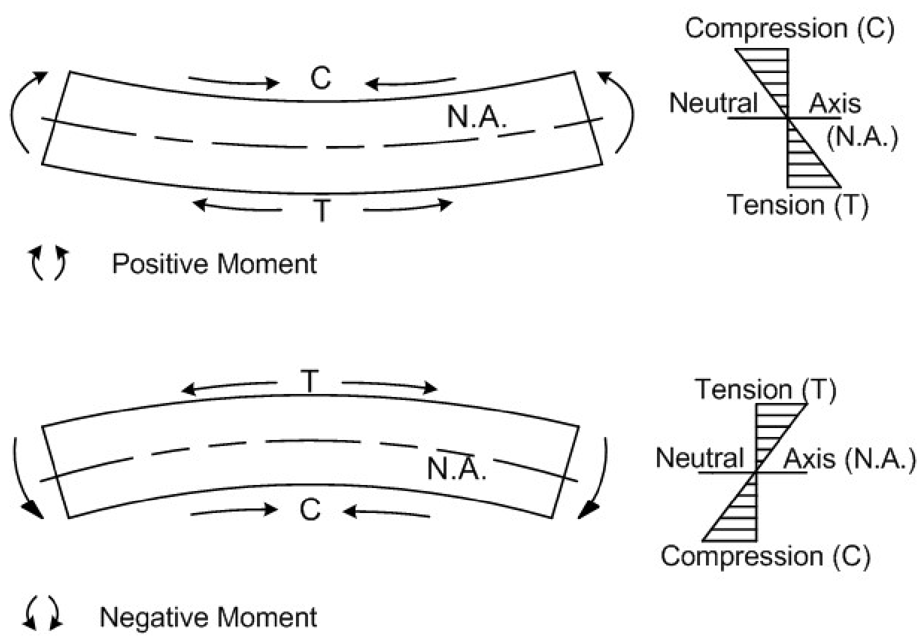


Figure 6.2.16 Positive and Negative Moment

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, and 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.

Beams and girders are the most common bridge members 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 6.2.17).

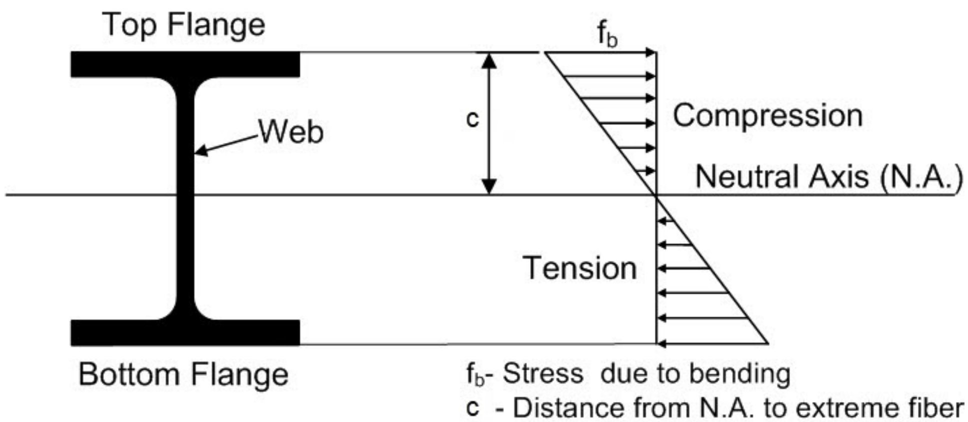


Figure 6.2.17 Girder Cross Section Resisting Positive Moment

Bending stress is typically 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 Figure 6.2.16 and Figure 6.2.17).

The formula for maximum bending stress is:

$$f_b = \frac{M\,c}{I}$$

Where: f_b = bending stress on extreme fiber (or surface) of beam (kips per square inch)
M = applied moment (kip-inches)
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) (in^4)

Shear Forces

Shear is a force resulting from equal but opposite transverse forces, that tend to slide one section of a member past an adjacent section (see Figure 6.2.18). Shear forces are generally expressed in English units of pounds or kips.

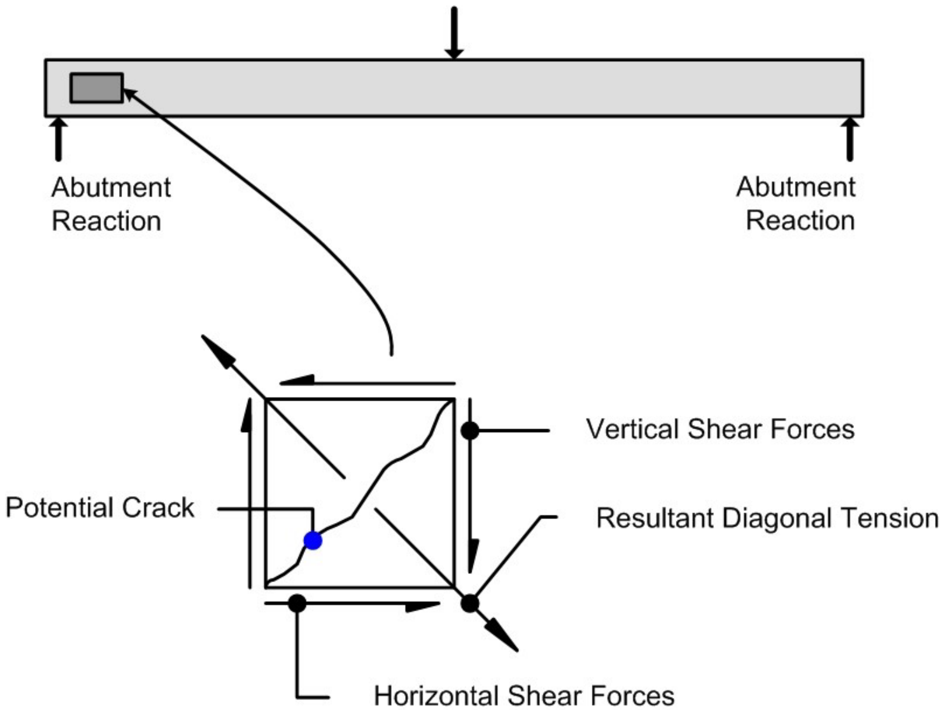


Figure 6.2.18 Shear Forces

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.

Beams and girders are common shear resisting members. In an I-beam or T-beam, most of the shear is resisted by the web (see Figure 6.2.18). 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 necessary to keep the member in equilibrium (see Figure 6.2.18). Vertical shear strength is generally considered in most design criteria. The formula for vertical shear stress in I-beam 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 resulted from externally applied moments that tend to rotate or twist a member about its longitudinal axis (see Figure 6.2.19). Torsional force is commonly referred to as torque and is generally expressed in English units of pound-ft or kip-ft.

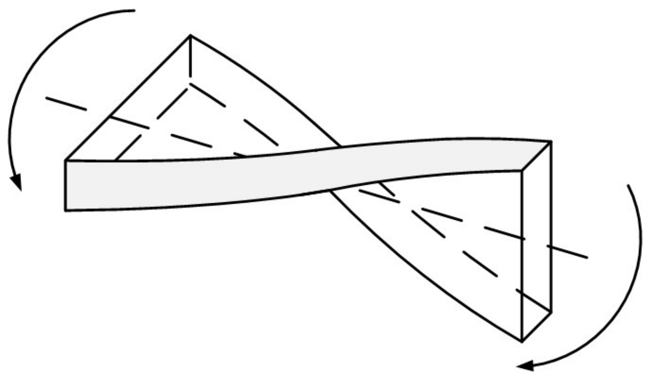


Figure 6.2.19 Torsion

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 (vertical). 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 (horizontal). The steel bar is resisting a torsional force or torque which has twisted it 90˚ with respect to its original orientation.

Torsional forces develop in bridge members, which are typically interconnected and may experience unbalanced loadings. Bridge members are generally not designed as torsional members. However, in some bridge superstructures where components are framed in union, torsional forces can occur in longitudinal members. When these members experience differential deflection, adjoining transverse members apply twisting moments resulting in torsion (see Figure 6.2.20). In addition, curved bridges are generally subject to torsion.

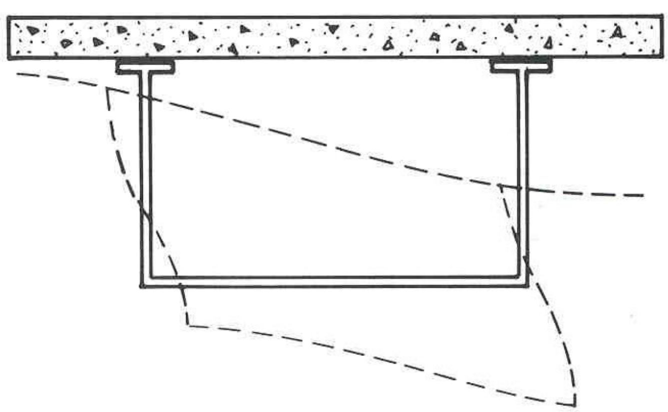


Figure 6.2.20 Torsional Distortion

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. Reactions are most commonly vertical forces (R_V), but a reaction can also be a horizontal force (R_H) or even a moment (M) (see Figure 6.2.21). 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, or kip-ft for moment reactions.

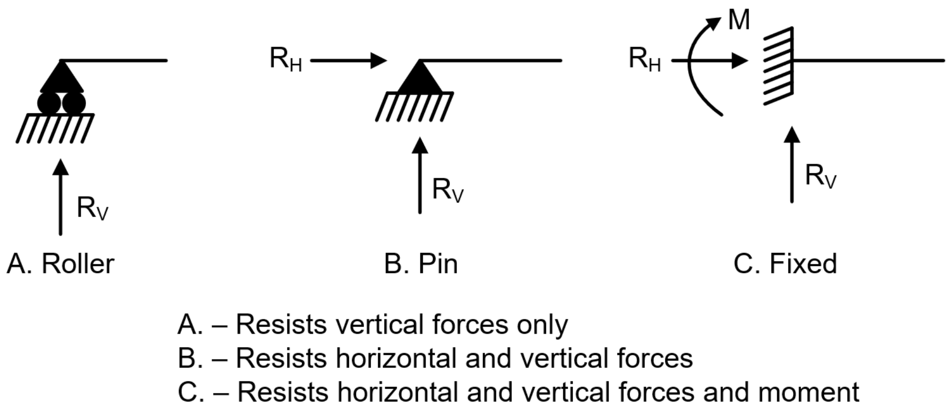


Figure 6.2.21 Types of Supports

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 should increase. As the books are shifted to one side, the reaction provided by the bricks at that side should increase, and the reaction at the other side should decrease.

The total of the reactions at each bridge support equals the applied permanent and transient loads. Equilibrium keeps the bridge in place.

6.2.4 Mechanics of Materials

Materials respond to loadings in a manner dependent on their mechanical properties. Various mechanical properties are below.

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 maximum amount of tensile stress that it can be subjected to before failure. It measures the force necessary to pull the material apart.

Compact Sections

For a cross section to be compact, the entire section must reach the yield point in compression without any local buckling failures. It is desirable for rolled steel beams to be compact sections, if possible, as localized buckling is not possible under transverse loading. For a section to qualify as compact, its flanges must be continuously connected to the web. It is also necessary to satisfy flange and web thickness ratios of the compression members. If the limiting width-thickness ratios are not met, the section may be considered non-compact or slender.

Toughness

Toughness is a measure of the energy necessary 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 likely demand more energy to break and thus exhibit more toughness. The unit for toughness is ft-lbs. @ degrees F. For steel highway bridges, the CVN (Charpy V-notch) toughness is the value typically used. It is an indicator of the ability of the steel to resist crack propagation in the presence of a notch or flaw. Wood is also considered a tough material and is particularly resistant to damage from impact.

Isotropic

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.

Refer to Chapter 7 for a description of isotropic materials.

Anisotropic

Anisotropic materials can have properties with different values when measured in different directions, which is the opposite of isotropy.

Example of anisotropy: Timber.

Refer to Chapter 7 for a description of anisotropic materials.

6.2.5 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 that can be accounted for in bridge design 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., 1 in vertical movement per 800 in (67 ft) 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 ft 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 enable 1-1/4 inches of movement for each 100 ft of span length for steel bridges and 1-3/16 inches of movement for each 100 ft 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.

6.2.6 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.

The *SNBI* uses the Design Method, Item B.LR.02 to record the method by which the bridge was designed. The coding options include the three methods to be discussed in this section (*SNBI* Subsection 5.1).

Load and Resistance Factor Design

As per FHWA, all bridges on federally funded projects designed after 2007 should utilize the LRFD method (FHWA *Load and Resistance Factor Design (LRFD) Memo,* 6/28/00). LRFD is a design procedure based on the actual strength, rather than on a 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 Load Factor Design (LFD).)

These design methods utilize 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. For this reason, it is important for inspectors to accurately report any deficiency found in the members.

Load Factor Design

LFD is a method in which the combined effect of the factored loads is limited to the ultimate strength of a material. The factored loads are determined from the applied loadings, which are increased by selected multipliers that provide a safety margin.

Like Allowable Stress Design, LFD is no longer included as a design method in the current AASHTO *LRFD Bridge Design Specifications.* However, bridge repair or retrofit may still use LFD in the 17^{th} Edition of AASHTO *Standard Specifications for Highway Bridges* where ten possible load combination groups are defined.

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.

ASD is no longer an accepted method for design of new structures but may be used in certain situations when analyzing or load rating structures that were designed with this method. The 17^{th} Edition (2002) of the AASHTO *Standard Specifications for Highway Bridges* included load combinations for ten possible ASD group loadings.

6.2.7 Bridge Load Ratings

One of the primary functions of a bridge inspection is to collect any information necessary for a bridge load capacity rating. Therefore, it is imperative to understand the principles of bridge load ratings. Bridge load rating methods and guidelines are provided by AASHTO in the AASHTO *Manual for Bridge Evaluation (MBE)*.

A bridge load rating is the analysis used to determine the safe vehicular live load carrying capacity of a bridge using bridge plans and supplemented by measurements and other information gathered from an inspection (23 CFR 650.305). Load ratings must be performed by, or under the direct supervision of, a registered professional engineer (23 CFR 650.309 (d)). Load rating engineers should oversee the load rating of bridge structures.

Each member of a bridge has a unique load rating, and the overall bridge load rating represents the rating for the most critical, or governing, member. Load ratings are presented as a load rating factor or weight in tons. The rating factor multiplied by the rating vehicle weight, expressed in units of tons, gives the load rating of the structure in tons. It is important to note that the load rating or rating factor of a bridge is dependent on the type of vehicles rated. That is, a bridge will most likely have different rating factors or load ratings for different rating vehicles.

Load ratings are performed for live load vehicles as specified by each state. Common load rating vehicles, including AASHTO recommended vehicles, are described in Section 6.2.1. The state may choose to develop their own load rating vehicles to encompass multiple AASHTO vehicles.

As with bridge design, load ratings may be calculated using Load and Resistance Factor, Load Factor, or Allowable Stress methods. The *MBE* explains when each method may be used. *SNBI* Item B.LR.04, Load Rating Method, reports the method used for load rating calculations. The Load Rating Date is also reported with Item B.LR.03. This item should reflect the most recent load rating analysis and may not coincide with the inspection date (*SNBI* Subsection 5.1).

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 enable 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. Inventory ratings have been refined to reflect the various material and load types. Reference the AASHTO *MBE* (Section 6B.5.2 for Allowable Stress inventory ratings, Section 6B.5.3 for Load Factor inventory ratings, and Section 6A.4 for LRFR Load Rating Procedures).

The LRFR design load rating 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 generally State legal loads within the AASHTO exclusion limits described in the *LRFD Bridge Design Specifications*.

SNBI Item B.LR.05, Inventory Load Rating Factor, is used to report the inventory rating for the standard AASHTO HS-20 or HL-93 loadings, depending on load rating method (*SNBI* Subsection 5.1).

Operating Rating

Load ratings based on the operating rating level generally describe the maximum permissible live load to which the structure may be subjected for the load configuration used in the load rating. Allowing unlimited numbers of vehicles use the bridge at operating level may shorten the life of the bridge (23 CFR 650.305). Operating ratings have been refined to reflect the various material and load types. Reference the AASHTO *MBE* (Section 6B.5.2 for Allowable Stress operating ratings and Section 6B.5.3 for Load Factor operating ratings).

LRFR does not use the term operating rating. The operating rating, if calculated, is under the design-load rating, operating rating evaluation level. (Reference *MBE Section 6A.4.3.*)

SNBI Item B.LR.06, Operating Load Rating Factor, is used to report the operating rating for the standard AASHTO HS-20 or HL-93 loadings, depending on load rating method (*SNBI* Subsection 5.1).

Legal Rating

In LRFR, the second level rating is a legal load rating. The legal load rating is the maximum permissible legal load to which the structure may be subjected with the unlimited numbers of passages over the duration of a specified bridge evaluation period (23 CFR 650.305). 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 governing rating factor for the State and AASHTO legal loads are reported via *SNBI* Item B.LR.07, Controlling Legal Load Rating Factor (*SNBI* Subsection 5.1).

Permit Rating

For LRFR, the third level rating is used to check the serviceability and safety of bridges in the review of permit applications. States issue permits for vehicles above legal loads to ensure safe travel and minimize damage to bridges. 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 in the AASHTO *MBE*. Guidance on checking serviceability criteria is also given in the *MBE*.

SNBI Item B.LR.08, Routine Permit Loads, is used to identify bridges that carry routine permit loads. It also indicates whether permit loads are restricted from the bridge (*SNBI* Subsection 5.1).

Bridge Posting

If the results of a load rating indicate the bridge is insufficient to carry legal loads, the bridge should be load posted or restricted. Load posting is defined as regulatory signs installed in accordance with 23 CFR 655.601 and State or local law which represent the maximum vehicular live load which the bridge may safely carry (23 CFR 650.305). The posting serves to warn the public of the load capacity of a bridge, to avoid safety hazards, and to adhere to Federal law. Bridges should be posted or restricted as per the AASHTO *MBE* and 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 legal load rating factor. The load posting should be implemented as soon as possible, but no later than 30 days after a load rating analysis determines the need for posting (23 CFR 650.31(l)(2)).

It may be the inspector's responsibility to gather and provide information that the structural engineer can use to analyze and load rate the bridge. The load rater may provide guidance on what field data is necessary, or may gather the information themselves. At the bridge site, the inspector should verify the proper posting signs are in place and properly installed.

Bridge postings show the maximum allowable load by law for single vehicles and combinations (see Figure 6.2.22).

Figure 6.2.22 Bridge Weight Limit Posting

Failure to comply with weight limits may result in fines, tort suits/financial liabilities, accidents, or even death. In addition, bridges may be damaged when postings are ignored (see Figure 6.2.23).

Figure 6.2.23 Damaged Bridge Due to Failure to Comply with Bridge Posting

6.2.8 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.

Span Continuity is reported to FHWA using *SNBI* Item B.SP.05. This item includes the basic classification for simple, continuous, and cantilever, but also includes codes for bridges that are continuous for live loads only and differentiates between cantilevers and cantilevers with pin and hanger assemblies. Frame and buried configurations are also available for this item (*SNBI* Subsection 2.1).

Simple

A simple span is a span with only two supports, each of which is at or near the end of the span. A simple span bridge can have a single span supported at the ends by two abutments (see Figure 6.2.24) or multiple spans with each span behaving independently of the others (see Figure 6.2.25).

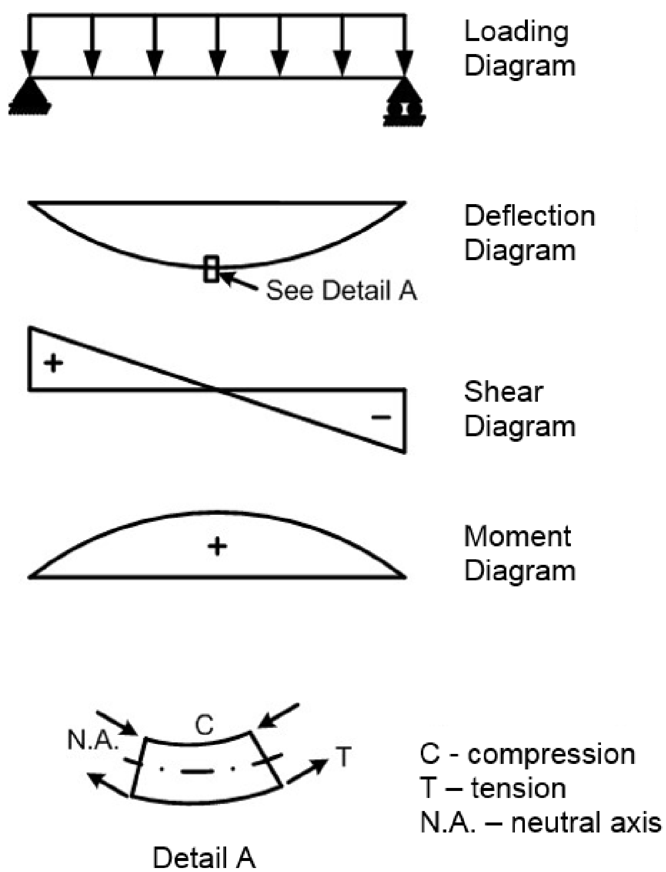


Figure 6.2.24 Simple Span

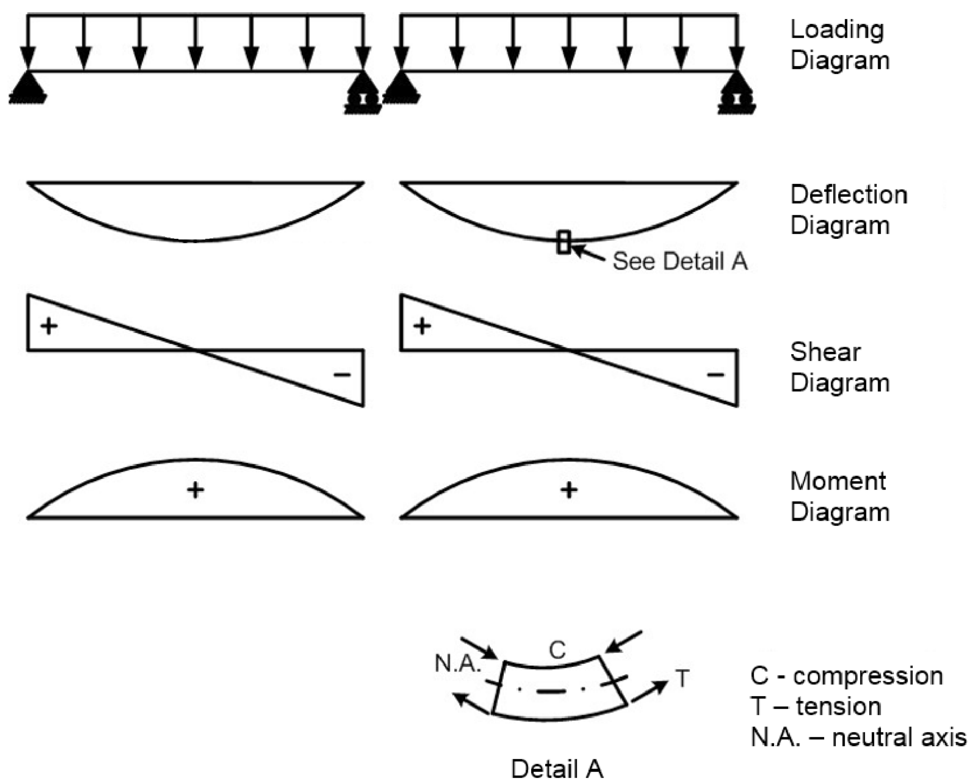


Figure 6.2.25 Multiple Simple Spans

Typical characteristics of simple span bridges include:

- 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 and 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 the behavior of each individual span is dependent on its adjacent span(s) (see Figure 6.2.26).

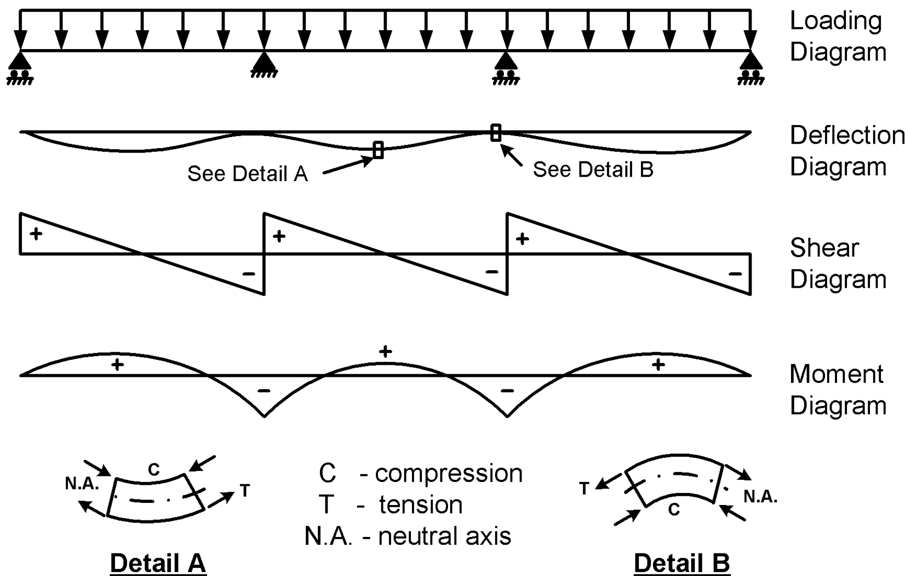


Figure 6.2.26 Continuous Spans

A continuous span bridge is one that is supported at the ends by two abutments and that spans uninterrupted over one or more intermediate supports. Typical characteristics of continuous span bridges include:

- 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 above the neutral axis and tension occurs below the neutral axis.
- For negative bending moments, tension occurs above the neutral axis and compression occurs below the neutral axis.

A continuous span bridge can have 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. Additional safety is provided in continuous bridges due to system redundancy. Refer to Section 6.2.10 and Chapter 7 for more information on redundancy. Another benefit of continuous spans is reduced water infiltration, as there are fewer expansion joints through which drainage can permeate. 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 6.2.27). The restrained end is also known as a fixed support.

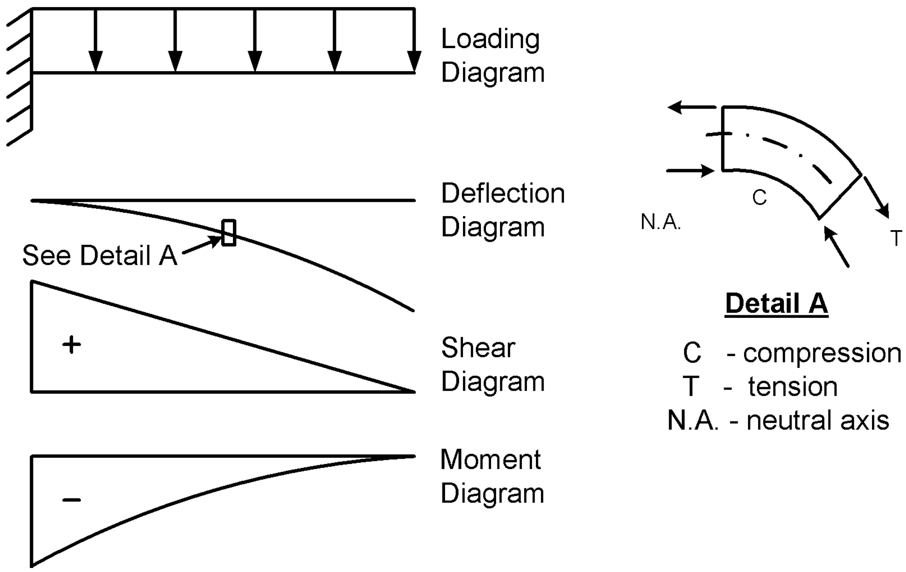


Figure 6.2.27 Cantilever Span

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 most likely be dependent on the adjacent span (see Figure 6.2.28).

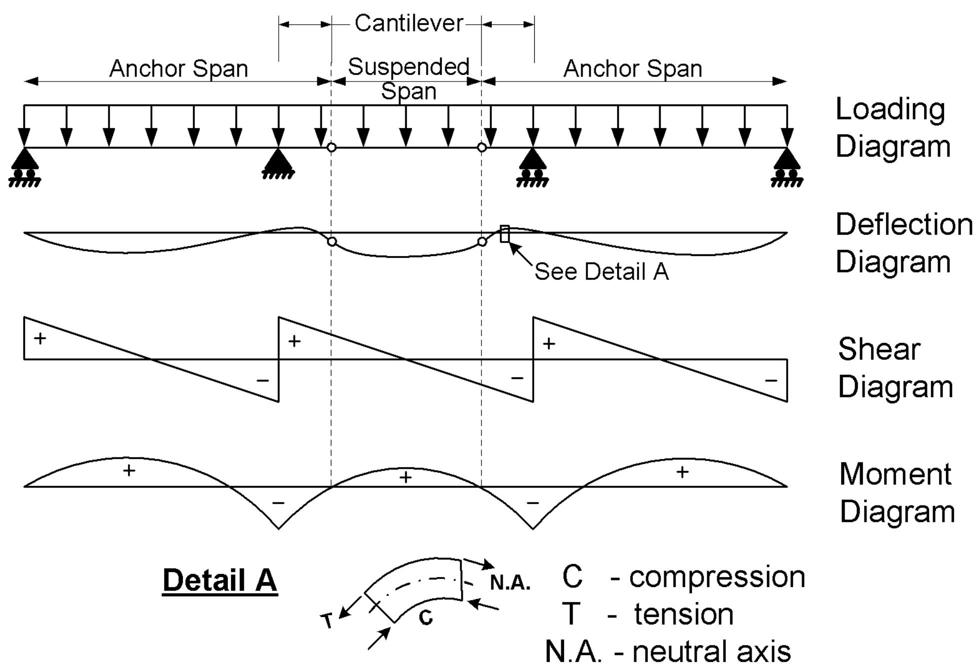


Figure 6.2.28 Cantilever Span Extension of a Continuous Span

Though a cantilever generally does not form an entire bridge, portions of a bridge can behave as a cantilever (e.g., ferry transfer bridges and bascule bridges). Typical characteristics of cantilevers include:

- When loaded, the span deflects downward, but there is no rotation or deflection at the fixed support.
- The fixed support reaction consists of a vertical shear 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.
- Tension occurs above the neutral axis and compression occurs below the neutral axis.

6.2.9 Bridge Deck Interaction

Bridges also have three classifications that are based on the relationship between the deck and the superstructure. These classifications include:

- Non-composite.
- Composite.
- Integral or monolithic.

SNBI Item B.SP.08, Deck Interaction, is used to report the type of interaction between the superstructure and deck for the span configuration based on the classifications noted above. For composite decks, this item differentiates between shored and unshored construction.

Non-composite

A non-composite structure is one in which the superstructure acts independently of the deck (see Figure 6.2.29). Therefore, the superstructure alone resists all of the loads applied, including the permanent loads and the transient loads.

Composite

A composite structure is one in which the deck acts with the superstructure to resist the loads (see Figure 6.2.29). 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 regularly reinforced concrete deck on prestressed concrete superstructure. The properties of the two concrete types are significantly different. Refer to Chapter 7 for detailed information on materials. 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 may act as a unit by preventing slippage between the two when a load is applied.

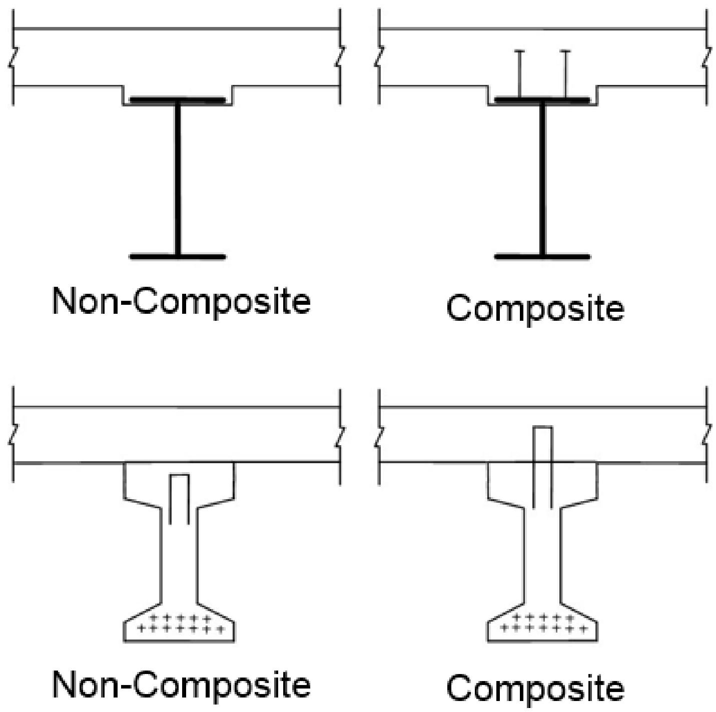


Figure 6.2.29 Non-Composite vs. Composite Deck Configurations

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 that 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 overlays.
- Parapets.
- Railings.
- Any other loads that 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 cannot identify the relationship between the deck and the superstructure at the bridge site. Therefore, they should review the bridge plans to determine whether a structure is non-composite or composite.

Integral

On an integral or monolithic bridge deck, the deck portion of the beam is constructed to act integrally with the stem, providing greater stiffness and increased span lengths (see Figure 6.2.30 and Figure 6.2.31).

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 as the superstructure (stem). An example of an integral bridge is a conventionally reinforced T-beam and is described in detail in Chapter 9.

Figure 6.2.30 Integral Bridge

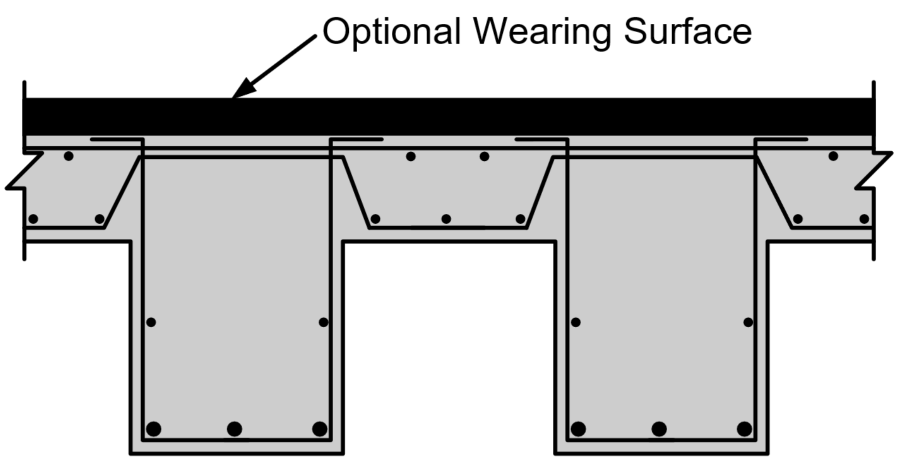


Figure 6.2.31 Cross Section: Integral Bridge

6.2.10 Redundancy

According to AASHTO *MBE*, 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, system redundancy, and internal redundancy.

Refer to Chapter 7 for detailed information on redundancy and system redundant members.

6.2.11 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) and deep foundations.

SNBI Item B.SB.06, Foundation Type, is used to report the type of foundation for the bridge (*SNBI* Subsection 2.2).

Refer to Chapter 14 for detailed foundation information for piers, bents, and abutments.

Shallow Foundations

A spread footing is used when the bedrock layers or when appropriate soil conditions are close to the ground surface and can support the bridge. A spread footing is typically a rectangular slab made of reinforced concrete (see Figure 6.2.32). This type of foundation "spreads out" the loads from the bridge to the underlying rock or well-compacted soil. Though 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 should be below the recognized maximum frost line depth for that area.

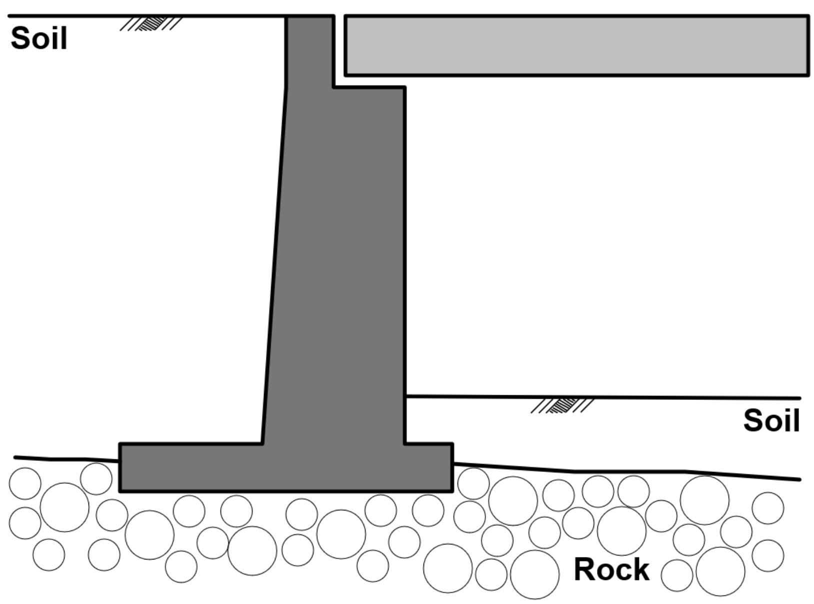


Figure 6.2.32 Spread Footing

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 (see Figure 6.2.33). 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. Caissons, drilled caissons, and drilled shafts are frequently used to transmit loads to bedrock in a manner similar to piles.

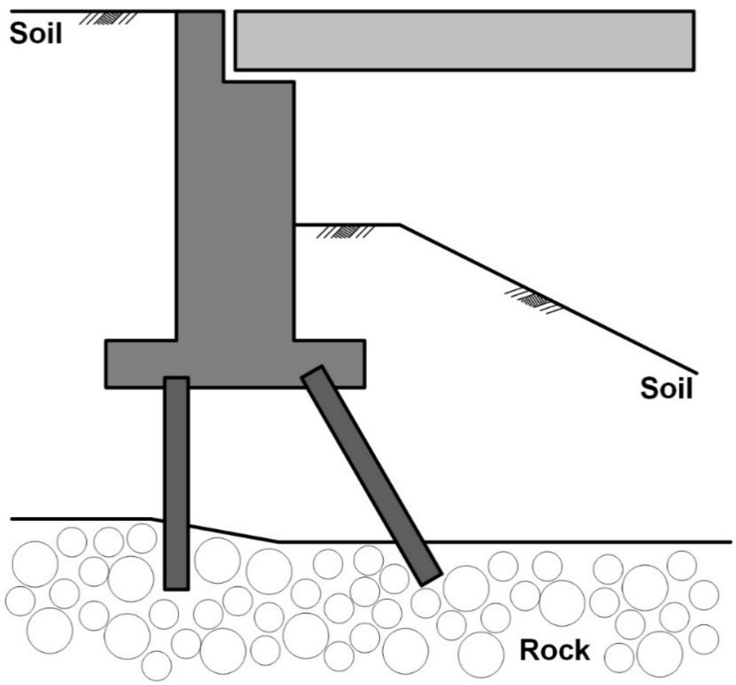


Figure 6.2.33 Deep Foundation

Section 6.3 Culvert Mechanics

6.3.1 Culvert Loadings

As described in Chapter 4, culverts not only have a hydraulic function, they also support the weight of the embankment or fill covering the culvert and any load on the embankment. The two general types of loads that are carried by culverts, like typical bridges, are permanent loads and transient loads.

Permanent Loadings

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 lbs./ft^3 is generally assumed. Self-weight of the culvert is also considered a permanent load.

Transient Loadings

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. Live load surcharge is a vertical load that can increase the magnitude of the horizontal load caused by the weight of the earth. The effect of live loads decreases as the height of cover over the culvert increases. When the cover is less than 2 ft, 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 (see Table 6.3.1). This concept is illustrated in Figure 6.3.1. In addition to the truck load, the HL-93 is also comprised of a 640-pound lane load, which converts into an additional 64 lbs./ft^2. As per AASHTO *MBE* single-span culverts with fill heights exceeding 8 ft, live load effects are negligible relative to the earth loads. The typical highway wheel load contact surface area used for design and analysis are shown in Figure 6.3.2.

Table 6.3.1 Spread Load Area (by Soil Type)

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

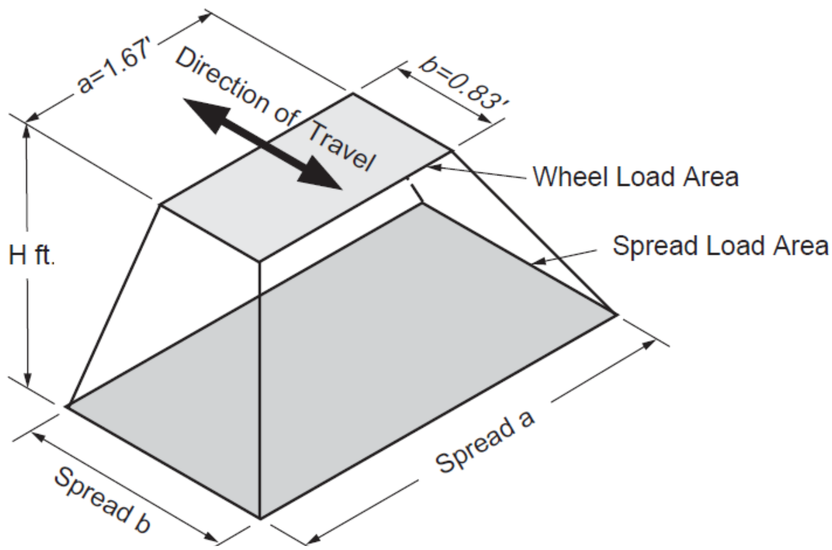


Figure 6.3.1 Spread Load Area (Single Dual Wheel)

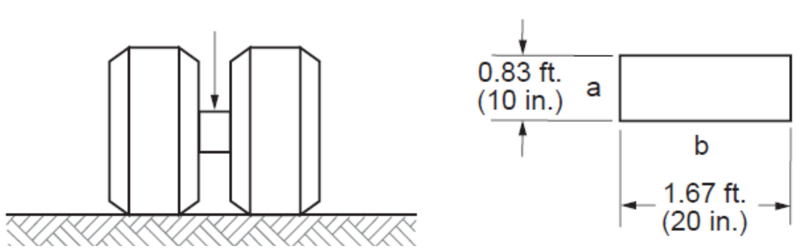


Figure 6.3.2 AASHTO Wheel Load Surface Contact Area (Footprint)

6.3.2 Categories of Structural Materials

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

Rigid Culverts

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 6.3.3).

Figure 6.3.3 Rigid Culvert

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).

See Figure 6.3.4 for the application of these basic loads.

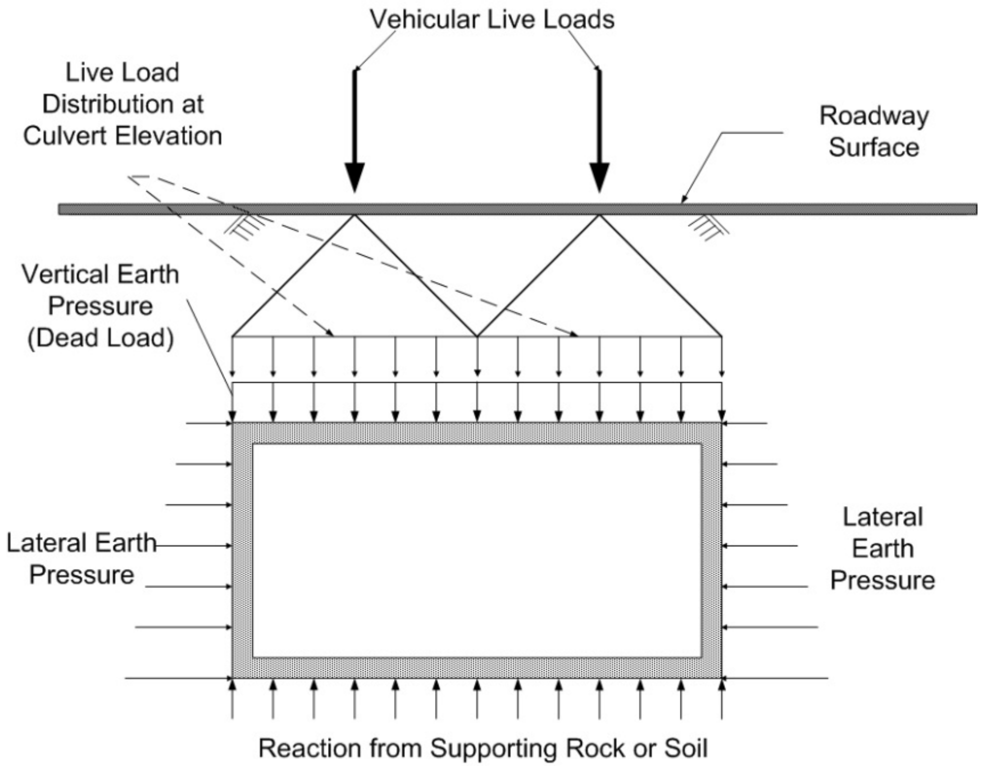


Figure 6.3.4 Rigid Culvert Loads

Pipe, arch, and frame culverts have the same types of loads as box culverts. Steel reinforcement is placed within concrete culverts and specifically situated to resist the loadings applied. More detail on primary and secondary reinforcement in culverts can be found in Chapter 15.

Flexible Culverts

A flexible culvert is a hydraulic structure made up of the culvert barrel and supported by the surrounding soil. The barrel and the soil are both vital features in 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 6.3.5). 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. The compressive thrust is approximately equal to the vertical pressure times one-half the span length (see Figure 6.3.6).

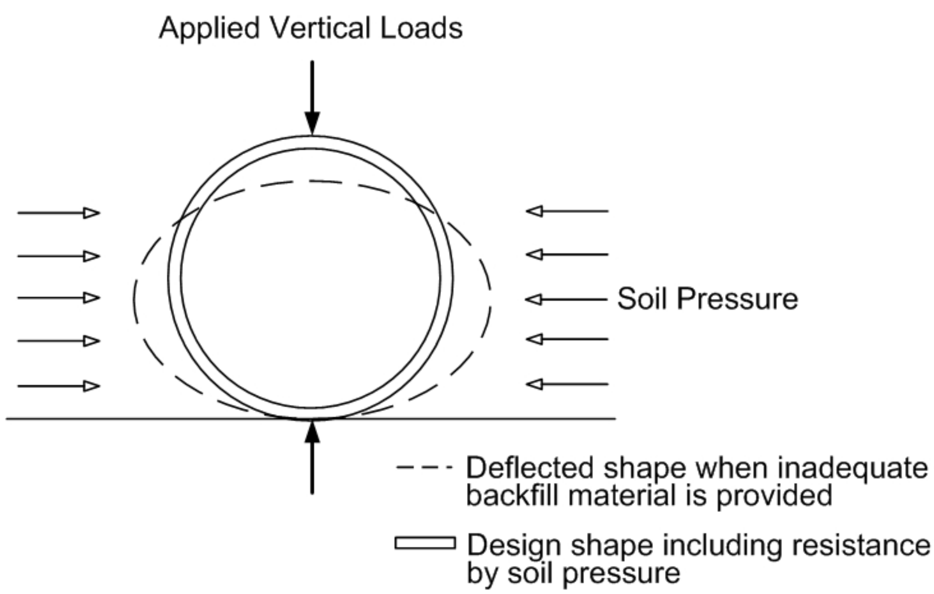
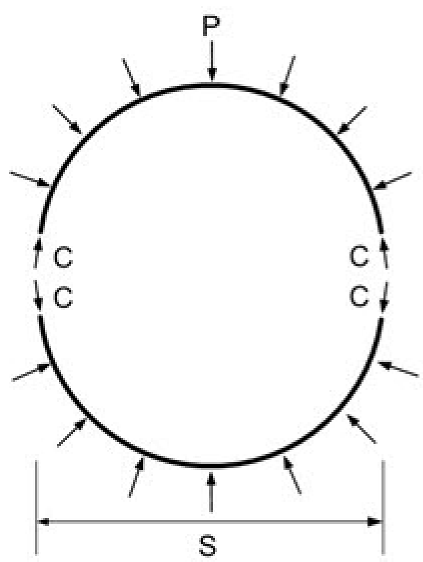


Figure 6.3.5 Flexible Culvert – Load vs. Shape



Summing the vertical forces on half of the pipe at a time shows that

C = P x S/2

Where
C = Compressive thrust in the culvert wall

P = Sum of soil pressure acting on the culvert

S = The span or diameter

S/2 = The radius (R)

Figure 6.3.6 Formula Ring for Compression

An arc of a flexible round pipe, or other shape should remain stable if 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 before the 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.

More detail on flexible culverts can be found in Chapter 15.

6.3.3 Basic Terminology

The basic terminology is similar in a culvert and a bridge. Reference Section 6.2.2.

6.3.4 Culvert Response to Loadings

Culverts experience the same four basic types of forces as bridge members and the equations of equilibrium apply. Reference Section 6.2.3.

Although axial, bending, shear, and torsional forces affect culverts, there are different areas of concern. For example, bending forces should be considered in the wall of a box culvert and torsional distortion may apply to an entire culvert or rigid frame.

6.3.5 Culvert Movements

The same factors play a role in the movement of culverts and bridges. Reference Section 6.2.5.

6.3.6 Design Methods

The same design methods apply to culverts and bridges. Reference Section 6.2.6.

AASHTO *LRFD Bridge Design Specifications, 9th Edition, Article 12.5.5* specifies limit states and resistance factors for buried structures. Of the group loadings mentioned in the AASHTO *Standard Specifications for Highway Bridges, 17th Edition*, only a few apply to culverts.

6.3.7 Culvert Load Ratings

The same load rating equations and rating vehicles apply to culverts and bridges. Reference Section 6.2.7 and reference the guidelines in the AASHTO *MBE.*

This page intentionally left blank.

PART II – BRIDGE, CULVERT, AND WATERWAY CHARACTERISTICS

CHAPTER 7 TABLE OF CONTENTS

CHAPTER 7 LIST OF FIGURES

CHAPTER 7 LIST OF TABLES

This page intentionally left blank.

Chapter 7 Materials, Material Deficiencies, and Inspection Methods

Section 7.1 Concrete

7.1.1 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. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

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

- Portland cement.
- Water.
- 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.

Cement is typically made from limestone and clay or shale. These raw materials are extracted from the quarry crushed to a very fine powder and then blended in the correct proportions. This blended raw material is called the 'raw feed' or 'kiln feed' and is heated in a rotary kiln where it reaches a temperature of about 275 degrees F. In its simplest form, the rotary kiln is a tube up to 700 ft long and perhaps 20 ft in diameter, with a long flame at one end. The raw feed enters the kiln at the cool end and gradually passes down to the hot end, then falls out of the kiln and cools down. 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 that are typically composed of rounded nodules between 1/32 inch to 1 inch across. After cooling, the clinker may be stored temporarily in a clinker store, or it may pass directly to the cement mill. The cement mill grinds the clinker to a fine powder. A small amount of gypsum - a form of calcium sulfate - is typically ground up with the clinker. The gypsum controls the setting properties of the cement when water is added.

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.

Aggregates

Aggregates make up more than 60 percent of a concrete mix and up to 80 percent in some cases. Aggregate is also less expensive than cement, so a higher percentage can lower the cost. Thus, the concrete mix designer can often save money by selecting the maximum aggregate size that can be used in the design. Using larger coarse aggregate typically lowers the cost of a concrete mix by reducing the necessary amount of cement, the most expensive ingredient. Less cement (within reasonable limits for durability) will most likely mean less water if the water-cement (w/c) ratio is kept constant. A lower water content should reduce the potential for shrinkage and for cracking associated with restrained volume change. Generally speaking, a good aggregate has a combination of rocks of many different sizes, graded, with a specific average and maximum size; the aggregate should be clean and durable, and should not contain clay or other minerals that can absorb water.

The high rock content of concrete makes concrete extremely durable, and it often is used in roadways, bridges, and airport runways. The ingredients in both concrete and cement are among the most abundant on earth, and both can be recycled. Cement production does demand a large amount of energy, however, because of the high temperatures necessary.

The characteristics of aggregates strongly influence the properties of the concrete. To produce high quality concrete, the aggregate should consist of clean, hard, strong, and durable particles free of chemicals, coatings of clay, or any other fine materials that may affect the hydration and the final bond of the cement paste. Weak, pliable, or laminated aggregate particles are undesirable. Aggregates containing natural shale or shale particles, soft and porous particles, and certain types of chert (a form of microcrystalline quartz) should be especially avoided since they have poor resistances to weathering.

Generally, the following characteristics are expected of aggregate:

- Freeze-thaw resistance.
- Chemical compatibility with cement.
- Particle shape and surface texture.
- Gradation and size.

Each of these characteristics is further described below.

In exposed concrete, aggregate should be resistant to repeated cycles of freezing and thawing. In the presence of absorbed water, repeated cycles of freezing and thawing can deteriorate the aggregate and reduce the strength of concrete with time.

Aggregate should chemically be compatible with cement. Some aggregate may react chemically with cement and cause expansion of concrete and subsequent severe cracking of concrete. This is called "alkali-aggregate" reaction, and the aggregate is called "alkali-reactive" aggregate. Tests are available to identify alkali-reactive aggregates. If alkali-reactive aggregate is the only aggregate available, cements with certain chemical composition or admixtures may be selected to prevent the alkali-aggregate reaction.

These characteristics of an aggregate mainly affect the properties of the plastic or formable (before hardened) concrete. Aggregates with rough texture or flat and elongated particles necessitate more water to produce workable concrete than round or cubical aggregates. In turn this can result in a higher water-cement ratio and lower strength. However, aggregates with rough and irregular shapes tend to lock to each other to produce a stronger matrix than smooth rounded shapes.

Generally, an aggregate gradation with minimum void content is desirable, since the cement paste demand for concrete increases as the void content in the aggregate is increased. Aggregate with uniform particle size increases void content. To provide the minimum void content, coarse and fine aggregate are combined. Coarse aggregates are particles that generally range between 3/8 and 1.5 inches in diameter, and fine aggregates generally consist of natural sand or crushed stone with most particles passing through a 3/8-inch sieve. Gradation limits for coarse and fine aggregate are specified to ensure minimum void content.

The maximum size of coarse aggregate in the mix depends on the size of the concrete member or size of repair, and on the spacing of the reinforcing steel. The maximum size of coarse aggregate should be small enough to place the mix in the form and in the spaces between the reinforcing steel with no difficulties. Specifications give the maximum size of aggregate on the basis of the minimum dimension of the form and clear space between the reinforcing bars. Also, the smaller the maximum size of aggregate, the greater the amount of mixing water will likely be necessary to produce the same workability. Therefore, it is advantageous to use the largest practicable maximum size of coarse aggregate

Admixtures

The fourth ingredient of most concrete mixes is one or more admixtures to change the consistency, setting time, or strength of concrete. Admixtures may be used primarily to reduce the cost of concrete construction; to modify the properties of hardened concrete; to ensure the quality of concrete during mixing, transporting, placing, and curing; and to overcome certain emergencies during concrete operations.

Admixtures can 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, superplasticizers, corrosion inhibitors, accelerators, and shrinkage reducers. Chemical admixtures are the ingredients in concrete other than Portland cement, water, and aggregate that are added to the mix immediately before or during mixing.

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.

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 1930s. This turned out to be a viable way to dispose of fly ash and positively affect 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 limitations, 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 necessary for concrete mixes using fly ash.

Silica fume (microsilica) results from the reduction of high purity quartz with coal in electric furnaces and producing silicon and ferrosilicon alloys at the same time. 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 may be used. These properties have contributed to the increased use of high performance concrete in recent bridge design and construction. Some limitations 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 might be exposed to deicing salts and to 50 percent in areas that do not demand the use of deicing salts.

Corrosion inhibiting admixtures are commonly used to attempt to prevent corrosion of any steel embedded in the concrete. Integral corrosion inhibitors utilize a chemical compound such as calcium nitrate to retard the onset of the corrosive action of chlorides. This admixture is advantageous particularly in cold weather exposure applications.

Water reducing admixtures and plasticizers may be 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 hydroxylated 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.

Superplasticizers are generally used in high strength concrete applications. Also known as high range water reducers, superplasticizers are synthetic polymers as opposed to the lignosulphonates in traditional plasticizers. Whereas plasticizers can produce a water content of 15 percent less than normal, superplasticizers can reduce the water content by 30 percent or more.

Retarding admixtures may be used to slow down the hydration process and not change the long-term mechanical properties of concrete. This type of admixture is necessary 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. Conversely, accelerating admixtures are utilized to speed up the curing process. Accelerators, one of the most common kinds of chemical admixtures, increase the rate of hydration and enable the concrete strength to develop faster than normal. Accelerators are commonly used in cold weather or time sensitive projects. These admixtures should be used sensibly, as the additional calcium could lead to steel corrosion and minimal setting times can create unwanted cracking in the finished concrete.

Shrinkage reduction admixtures are sometimes added to lower the risk of cracking. Shrinkage reducers can decrease both early and long-term drying shrinkage by regulating the water content. Where new concrete is used to repair existing structures, these admixtures may help to control cracking and provide durable construction joints.

The typical air entrainment additive is a vinsol resin. Air entrainment also increases durability against freeze/thaw effects, reduces cracking, improves workability during construction and reduces water segregation. Many tiny air bubbles introduced into the plastic concrete naturally create lighter weight concrete. 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.

7.1.2 Properties of Concrete

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

Physical Properties

The major physical properties of concrete are:

- Thermal expansion - concrete expands as temperature increases and contracts as temperature decreases.
- Porosity - because of entrapped air, the cement paste never completely fills the spaces between the aggregate particles, permitting absorption of water and the passage of water under pressure.
- Volume changes due to moisture - concrete expands with an increase in moisture and contracts with a decrease in moisture.
- Fire resistance - quality concrete is highly resistant to the effects of heat; however, temperatures over 700 degrees Fahrenheit may cause damage.
- Formability - concrete can be cast to almost any shape before 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 7.1.1).
- Six principal factors that increase concrete strength are:
  - o Increased cement content.
  - o Increased aggregate strength.
  - o Decreased water-to-cement ratio.
  - o Decreased entrapped air.
  - o Increased curing time (extent of hydration).
  - o 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. Refer to Chapter 6 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.

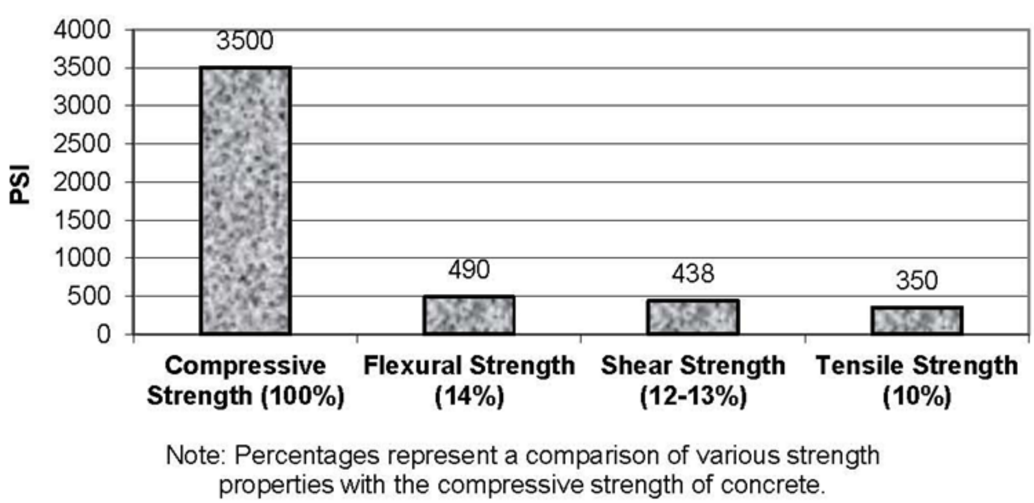


Figure 7.1.1 Strength Properties of Concrete (3500 psi Concrete)

7.1.3 Reinforced Concrete

Concrete is commonly used in bridge applications due, in part, 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

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.

Lightweight concrete typically 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.

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 7.1.2). The gripping pattern improves bond with the surrounding concrete. Modern reinforced concrete bridges are constructed with "deformed" reinforcing steel. Older bridges (1930s and older) used square reinforcement bars. Sometimes they were twisted to produce a gripping pattern similar to "deformed" rebars discussed above. Reinforcement bars are commonly identified by Grade. For instance, Grade 60 rebar has a tensile strength of 60 ksi.

Figure 7.1.2 Standard Deformed Reinforcing Bar

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 (see Table 7.1.1). 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.

Table 7.1.1 Table of Standard Reinforcing Bar Sizes

inch-lb. Bar Size	Diameter (inches)	Area (inches^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

Reinforcing bars can be protected, with epoxy for example, or unprotected from corrosion. Unprotected reinforcement is referred to as “black” steel because only mill scale is present on the surface.

Current steel reinforcement has approximately 100 times the tensile strength of commonly used concrete. Typical new steel reinforcement has a tensile yield strength of 60 ksi or 75 ksi. Bridges built in or before the 1950s utilized 40 ksi reinforcement. Stainless steel reinforcement (40 ksi to 75 ksi) is becoming more common due to manufacturers estimating a service life of 100 years. Stainless steel rebars may be solid stainless steel or coated in stainless steel and referred to as stainless steel clad rebars. MMFX rebar (75 ksi to 100 ksi) is also becoming more prevalent. These uncoated bars are corrosion resistant and provide a high strength reinforcement option.

Due to the differentiation in tensile strengths, in conventionally reinforced concrete members, the concrete is more equipped to resist 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 situated close to the tension face of a structural member to maximize its efficiency (see Figure 7.1.3).

Figure 7.1.3 Spalled Concrete Member with Exposed Tensile Steel Reinforcement

Shear reinforcement is also necessary in flexure members to resist diagonal tension. 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 resist crack initiation and propagation.

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, combined, provide a strong, durable construction material.

Reinforcing bars are also placed uniformly around the perimeter of a flexure member to resist stresses resulting from temperature changes and volumetric changes of concrete (see Figure 7.1.4). This steel is referred to as temperature and shrinkage steel.

Figure 7.1.4 Rebar Cage for Concrete Member

7.1.4 Prestressed Concrete

Another type of concrete used in bridge applications is prestressed concrete, which uses high tensile strength steel strands as reinforcement. (see Figure 7.1.5). 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 7.1.6).

Figure 7.1.5 Fabrication of a Prestressed Concrete Beam

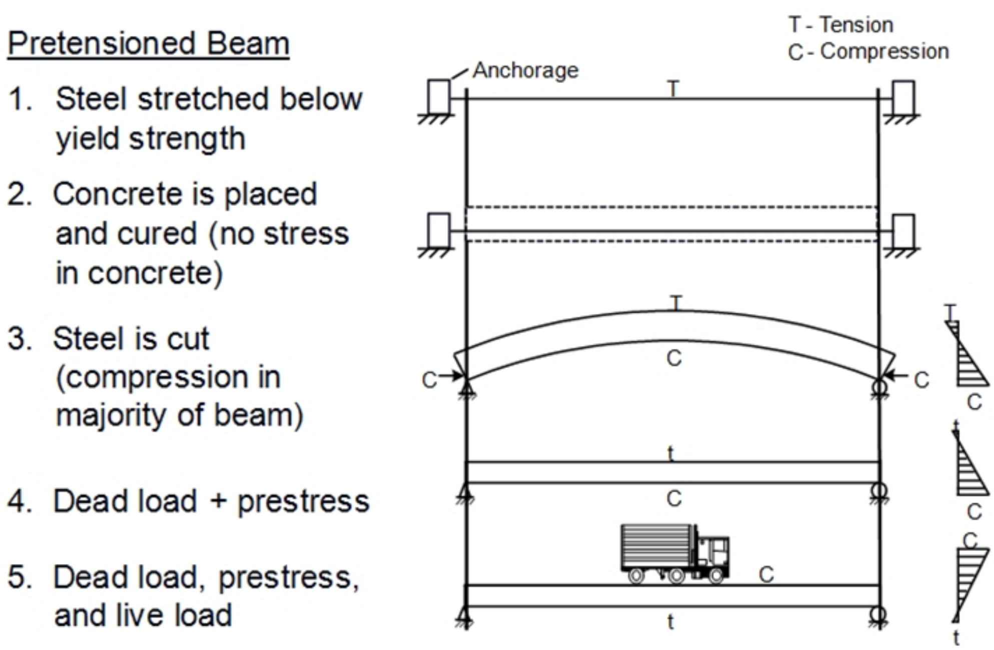


Figure 7.1.6 Prestressed Concrete Beam

Prestressing Methods

There are three methods of prestressing concrete:

- Pre-tensioning.
- Post-tensioning.
- Combination method.

Pre-tensioning is a method in that steel is tensioned before concrete placement; during fabrication of the member, prestressing steel is placed and tensioned before casting and curing of the concrete (see Figure 7.1.7) The prestressing steel is bonded by the concrete.

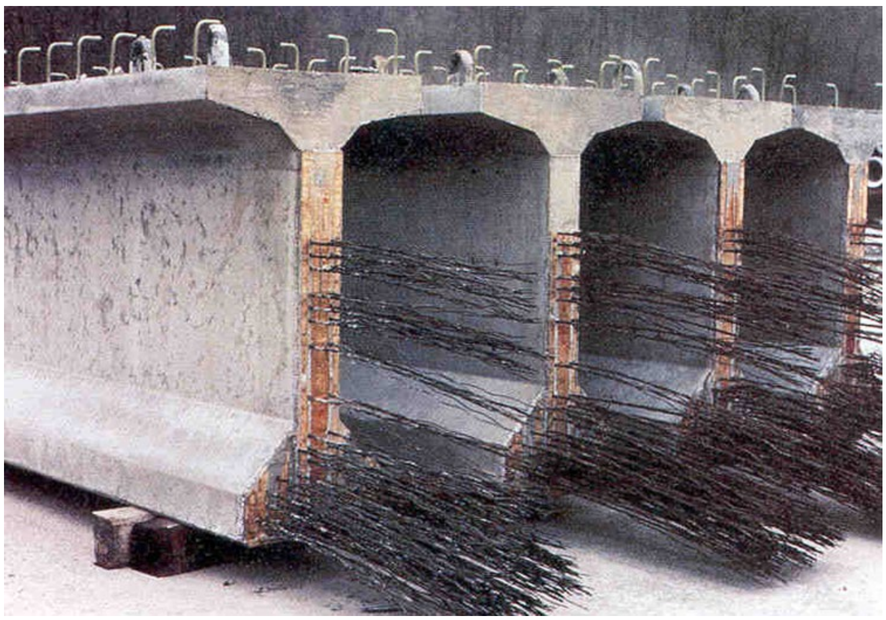


Figure 7.1.7 Pretensioned Concrete I-Beams

Post-tensioning is the term used for steel that is tensioned after concrete placement; during fabrication of the member, concrete is cast around ducts (typically plastic but occasionally steel) so that after curing, the prestressing steel can be passed through the ducts and tensioned (see Figure 7.1.8). The prestressing steel has no bond with the member concrete. Sometimes these ducts are exposed, usually inside of box girders, to provide for visual inspection.

A combination method is generally used for longer members for which the necessary pretensioned prestressing force alone is not adequate or construction methods such as segmental girders demand post-tensioning.

Figure 7.1.8 Post-tensioned Concrete Segmental 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 common 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 around each other; 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 seldom used 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 tend 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 tends to melt, this type of alternative is not used in pretensioned members 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 addition, mild steel reinforcement is incorporated at the end of beams for crack control.

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 typically used that are pressure injected with grout after the tendons are tensioned and locked off. The grout also provides an additional phase of corrosion protection. If the ducts crack or grout system breaks down, the tendons are very susceptible to corrosion and should be closely inspected.

For post-tensioned members, when bonding is not desirable, grouting of tendon ducts is not performed and corrosion protection is in the form of galvanizing, greasing, and sheathing, or some other means may be provided. For some bridge types, the post-tensioning tendons are not embedded in the concrete at all. Refer to Chapter 9 for types of post-tensioned concrete bridges.

In prestressed concrete beams, shear strength is enhanced by the local compressive stress present. However, mild shear reinforcement is still necessary. Similar to reinforced concrete, prestressed concrete also necessitates mild steel temperature and shrinkage reinforcement.

7.1.5 Types of Concrete

In addition to the most common type of concrete, Portland cement concrete, there are other types of concrete in use or being researched at the present time for bridge construction. These include:

- Low slump dense concrete (LSDC).
- Latex modified concrete (LMC).
- Internally sealed concrete.
- Lightweight concrete (LWC).
- Fiber reinforced concrete (FRC).
- Polyester Polymer Concrete (PPC).
- Self-Consolidating Concrete (SCC).
- Silica Fume Concrete.
- High Performance Concrete (HPC).
- Ultra-High Performance Concrete (UHPC).

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 1960s for patches and overlays on bridges in Iowa and Kansas (hence the common term "Iowa Method"). This type of concrete is generally used because it cures rapidly and has a low permeability. The low permeability resists chloride penetration, and the fast curing decreases the closure period. Low slump dense concrete is placed mainly in locations where deicing salts may be 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 likely demand resurfacing after about 25 years of service, regardless of the concrete deck deterioration caused by steel reinforcement corrosion.

Latex modified concrete involves the incorporation of polymer emulsions into the fresh concrete. The emulsions have been polymerized before 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 overlays.

The primary difference between the LSDC and the LMC is that low slump concrete uses inexpensive materials but is difficult to place and demands special finishing equipment. Conversely, latex-modified concrete utilizes expensive materials but demands less labor and is placed by conventional equipment. The performance of LMC has generally been satisfactory, although in some cases, extensive map cracking and debonding have been reported. The causes for this are likely the improper application of the curing method, application under high temperature, or shrinkage due to high slump.

Internally sealed concrete uses wax beads to reduce the intrusion of chlorides for the protection of the steel reinforcing After the concrete has cured, the wax beads are heated. The melted wax seeps into concrete voids to prevent water from penetrating into the concrete and corroding the steel reinforcement.

Lightweight concrete (LWC) is concrete with lightweight aggregates and a higher entrained air content. This produces a concrete mix of approximately 80 to 100 pcf compared to 140 to 150 pcf for conventional concrete. This type of concrete has a reduced dead load compared to a traditional concrete. Lightweight concrete is also used for cast-in-place and precast decks, as well as overlay wearing surfaces.

Deterioration in concrete members is primarily caused by corrosion of conventional reinforcement. Fiber-reinforced polymer (FRP) composite reinforcement is sometimes used as an alternative, as it does not corrode like conventional reinforcement.

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 7.1.9). The most common type of discontinuous fiber reinforcement is polypropylene, though organic timber fibers are currently being researched with promising results.

The fiber minimizes shrinkage cracking of fresh concrete and increases the impact strength of cured concrete. This type of concrete is used in bridge decks (refer to Chapter 8 for more information).

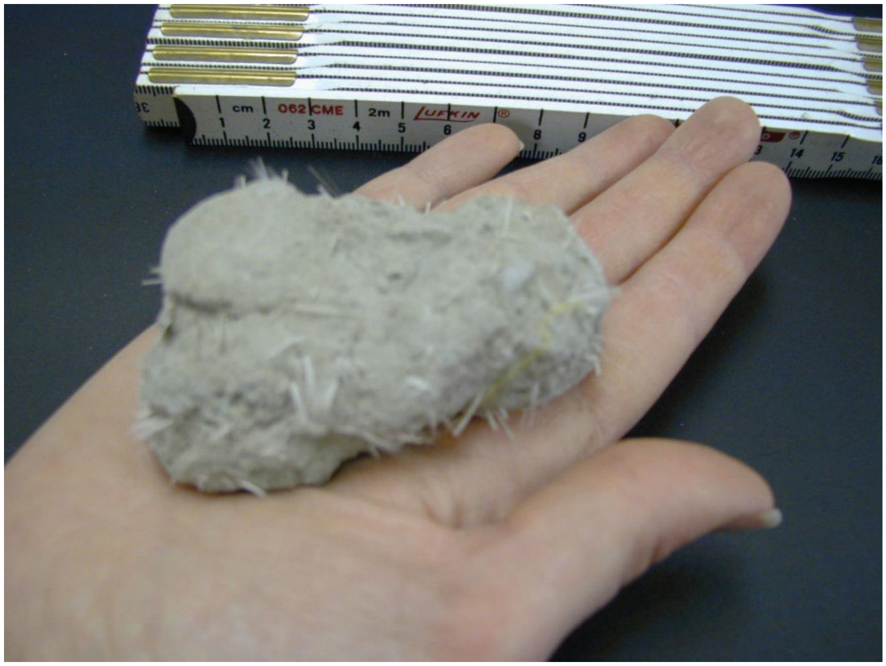


Figure 7.1.9 Fiber added to Fiber Reinforced Concrete (FRC)

Polyester Polymer Concrete is a mix of sand, stone, and polyester resin. It does not include cement or water. This type of concrete is generally used for overlays and only placed in limited thicknesses as protection for concrete decks.

Self-consolidating concrete is a very fluid mix of concrete that flows easily within any formwork and does not necessitate tamping or vibration. SCC is fluid due to the high proportion of fine aggregates combined with superplasticizers and viscosity-enhancing admixtures. It does not have a higher water content than normal concrete mixtures. Because of its composition, SCC generally has a low yield stress.

Silica fume is added to Portland cement concrete to improve its properties, such as compressive strength, bond strength, and abrasion resistance. Silica fume is a very effective pozzolanic material and also reduces the permeability of concrete to chloride ions, which protects the reinforcing steel from corrosion, especially in chloride-rich environments such as coastal regions.

High performance concrete (HPC) has been used for many years in the building industry. Under the FHWA's Strategic Highway Research Program (SHRP) Implementation Program in the late 1980s, four types of high performance concrete mix designs were developed (see Table 7.1.2). High performance concrete is distinguished from regular concrete by its curing conditions and proportions of the ingredients in the mix design. The use of fly ash and high range water reducers play an important role in the design of HPC, as well as optimizing all ingredients 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 members can be found in the bridge file under design specifications or in construction plans and specifications.

Table 7.1.2 FHWA's Strategic Highway Research Program (SHRP) Implemented HPC Mix Design

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			

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 may include Portland cement, silica fume, quartz flour, fine silica sand, high-range water-reducer, water, and 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.

7.1.6 Anticipated Modes of Concrete Deficiencies

In order to properly inspect a concrete bridge, the inspection team should be able to recognize the various types of deficiencies associated with concrete. The inspection team also should understand the causes of the deficiencies and how to examine them. There are many common deficiencies that occur on reinforced concrete bridges:

- Cracking.
- Scaling.
- Delamination.
- Spalling.
- Exposed rebar.
- 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, concrete sand, and form streaking).

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. Cracks caused by dimensional changes due to shrinkage or temperature are considered non-structural cracks.

Larger structural cracks may indicate potentially serious problems, because they may be 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. 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, caused by frozen bearings for example, induce significant forces which result in cracks. Forces due to the expansion of an approach slab or failure of a backwall can also cause unexpected cracking.

There are two main types of structural cracks: flexure and shear (see Figure 7.1.10).

Structural Flexure Cracks

Flexure cracks are considered structural cracks and are caused by tensile forces and therefore develop in the tension zones. Tension zones can be present at the bottom or the top of a member as a result of a bending moment, depending on the span configuration. Tension zones can also occur in substructure units. Tension cracks terminate when they approach the neutral axis of the member. However, the neutral axis may change when a flexural member is cracked. The crack may propagate to this “revised” neutral axis. Refer to Chapter 6 for further discussion of neutral axis. If a beam is a simple span structure, 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 7.1.11). If the beams are continuous span structures, flexure cracks can also occur at the top of members at or near their interior supports. Flexural cracks propagate perpendicular to primary reinforcement.

Structural 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. Typically, 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 7.1.12). Shear structural cracks can also occur in abutment backwalls, stems and footings, pier caps, columns, and footings.

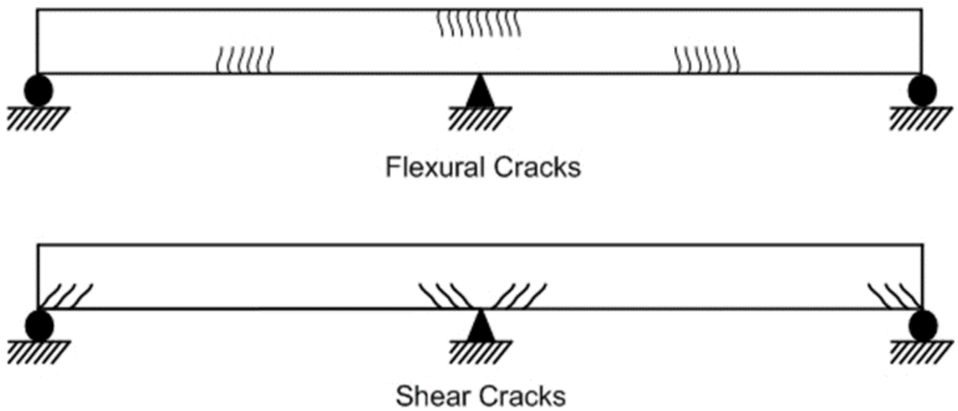


Figure 7.1.10 Structural Cracks

Figure 7.1.11 Flexural Cracks on a Tee Beam

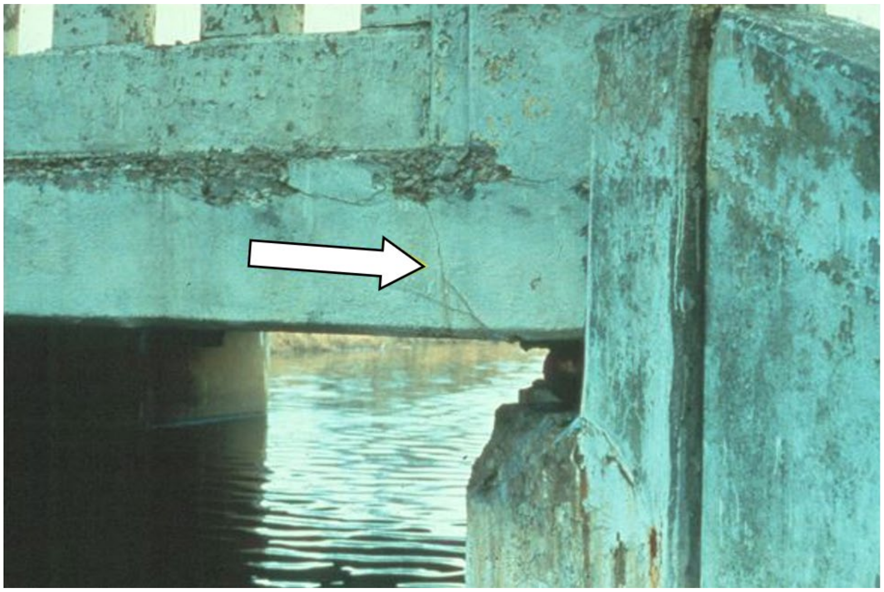


Figure 7.1.12 Shear Crack on a Slab

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 7.1.13).

Figure 7.1.13 Crack Comparator Card

According to the AASHTO Manual for Bridge Element Inspection, cracks in reinforced concrete less than 0.012 inches can be considered insignificant. Cracks ranging from 0.012 to 0.05 inches can be considered moderate, and cracks greater than 0.05 inches can be considered wide.

According to AASHTO Manual for Bridge Element Inspection, cracks in prestressed concrete less than 0.004 inches can be considered insignificant. Cracks ranging from 0.004 to 0.009 inches can be considered moderate, and cracks greater than 0.009 inches can be considered wide.

Typically, cracking in prestressed elements is more problematic than cracking in reinforced concrete elements. Reinforced Concrete elements are expected to have some cracking under tension forces due to flexure. Prestressed concrete elements are designed to primarily be in compression and not expected to experience tension cracks due to flexure.

In cases where flexural cracking is minor or appropriate assessment has indicated that the cracking has 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.

Inspectors should record the length, width, location, and orientation (horizontal, vertical, or diagonal) when reporting cracks. Cracks in main members or primary members should be carefully recorded. It should be documented if the crack extends partially or completely through the member. Inspectors should 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 7.1.14).
- Shrinkage cracks (see Figure 7.1.15).
- 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, shrinkage, and mass concrete cracks typically do not significantly affect the structural strength of a concrete member.

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 most likely crack under tension. Inoperative bearing devices and clogged/frozen expansion joints can also cause this to occur. Temperature cracks are typically more parallel than shrinkage cracks. Temperature cracks typically occur perpendicular to the longitudinal axis of a member in an area where cracking is not anticipated to be in bending/flexure.

Shrinkage cracks are due to the shrinkage of concrete caused by the curing process (loss of moisture). Volume reduction due to curing is also referred to as plastic shrinkage. Plastic shrinkage cracks occur when 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. Placed in small quantities and thin slabs, the heat of hydration dissipates evenly and quickly, but with mass concrete members the temperature in concrete can increase faster than it escapes. High temperatures and differences between the center of the mass and the surfaces can cause thermal cracks to develop.

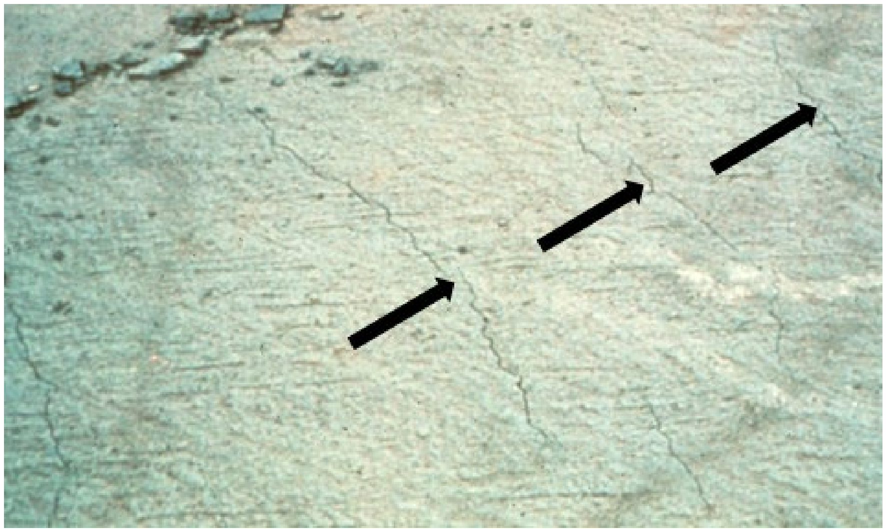


Figure 7.1.14 Temperature Cracks

Figure 7.1.15 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. Delaminations and spalls are discussed later in the chapter.

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 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 member (see Figure 7.1.16).
- Longitudinal cracks - These are fairly straight cracks that run parallel to the primary reinforcement of the bridge or a bridge member (see Figure 7.1.17).
- Diagonal cracks - These cracks are skewed (at an angle) to the centerline of the bridge or a bridge member, vertically or horizontally.
- Pattern or map cracking - These are inter-connected cracks that form networks of varying size. (see Figure 7.1.18).
- 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 7.1.16 Transverse Cracks

Figure 7.1.17 Longitudinal Cracks

Figure 7.1.18 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 also occurs more frequently when the concrete is not properly air entrained or if the concrete surface is not properly finished. 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 7.1.19).
- 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 7.1.20).
- Heavy scale - loss of surface mortar from 1/2-inch to 1-inch deep; coarse aggregates are clearly exposed (see Figure 7.1.21).
- 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 7.1.22).

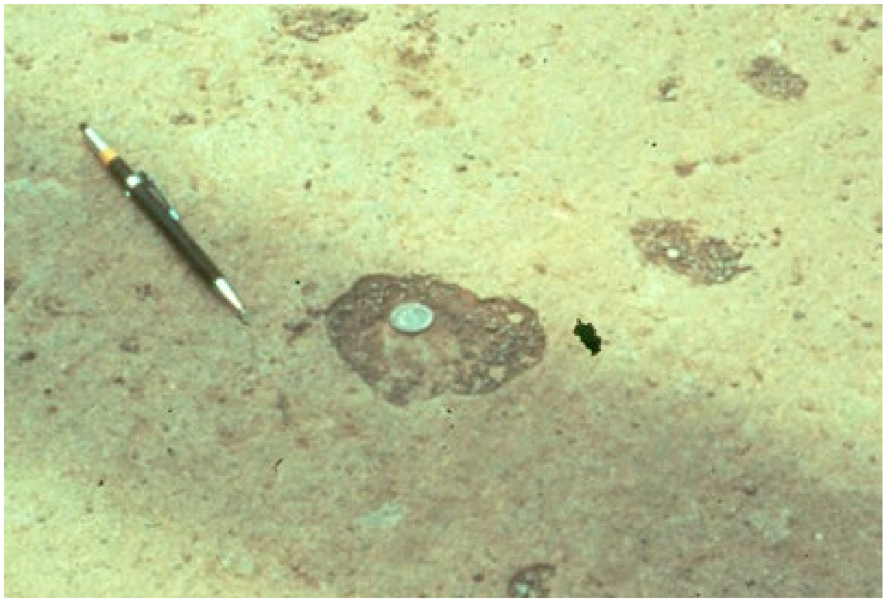


Figure 7.1.19 Light or Minor Scaling

Figure 7.1.20 Medium or Moderate Scaling

Figure 7.1.21 Heavy Scaling

Figure 7.1.22 Severe Scaling

When reporting scaling, inspectors should 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 should also be recorded.

Delamination

Delamination or delaminated area occurs when layers of concrete separate at or near the level of the outermost layer of reinforcing steel (see Figure 7.1.23). 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.

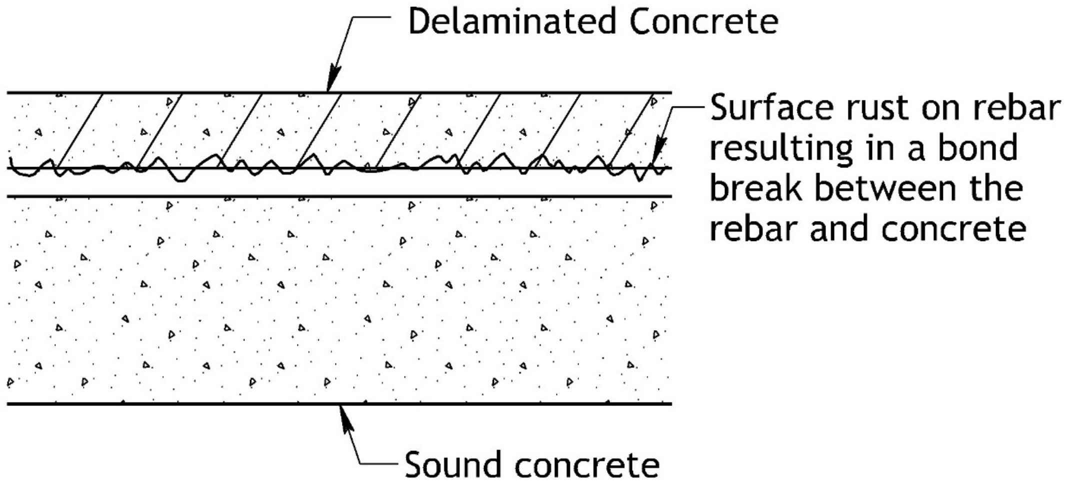


Figure 7.1.23 Concrete Delamination

Spalling

A spall is a depression in the concrete (see Figure 7.1.24). 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 7.1.24 Spalling on a Concrete Deck

When concrete is overstressed, it deflects or fractures. Over time, the fracture opens wider from debris, freeze/thaw cycles, or more overstress. Cracks are presented earlier in this section. This cycle continues until a spall is formed. Spalls caused from overstress are very serious and are to be brought to the observation of the Chief Bridge Engineer (CBE). Most spalls are caused from corroding reinforcement, but if the spall is 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.

Exposed Rebar

Exposed rebar is generally a defect that coincides with concrete spalling, but may be a result of insufficient concrete cover. In most cases, the reinforcement has begun corroding prior to becoming exposed, which causes the concrete to be displaced.

Chloride Contamination

Chloride contamination in concrete is the presence of recrystallized 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. Free and bound chlorides are sourced from soluble salts present in the concrete materials or on the surface of aggregates. Chloride ions that are dissolved in the pore water are commonly referred to as free chlorides. Bound chlorides are ions that are chemically bound to hydrated cement paste. Chloride contamination can also be caused by aggregates with high chloride content. During the 1960s, salt was added to water to prevent it from freezing during mixing and fabrication. Practices like these are no longer tolerable. This practice causes accelerated reinforcement corrosion that leads to cracking of the concrete. Various admixtures are incorporated to account for adverse weather condition.

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, spalling, 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 recrystallized carbonate and chloride compounds (see Figure 7.1.25). When cement is mixed with sand and water to form concrete, the concrete sets because the calcium oxide reacts with carbon dioxide, especially at and near the surface where there is abundant carbon dioxide. However, some unreacted calcium oxide remains in the body of the concrete. The calcium oxide reacts to form hydroxide and then into a hydroxide solution as water slowly seeps through the concrete. The calcium hydroxide solution reacts when exposed to the air, forming calcium carbonate which is the stalactites seen in Figure 7.1.25. Therefore, concrete stalactites may eventually form where rainwater can percolate through the concrete. In order to estimate the percent of concrete contaminated by chloride, nondestructive testing is necessary (refer to Chapter 17).

Figure 7.1.25 Efflorescence

Alkali-Silica Reaction

The Alkali-Silica Reaction (ASR) process involves a reaction between potassium (K) and sodium (Na) alkalis (common in cement) and silica (common in aggregates) (see Figure 7.1.26). 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.

ASR is an expansive reaction, forming a gel, that may result in the swelling and expansion of concrete. (See Figure 7.1.27).

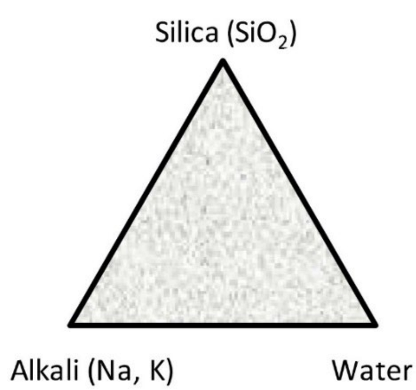


Figure 7.1.26 Conditions for ASR

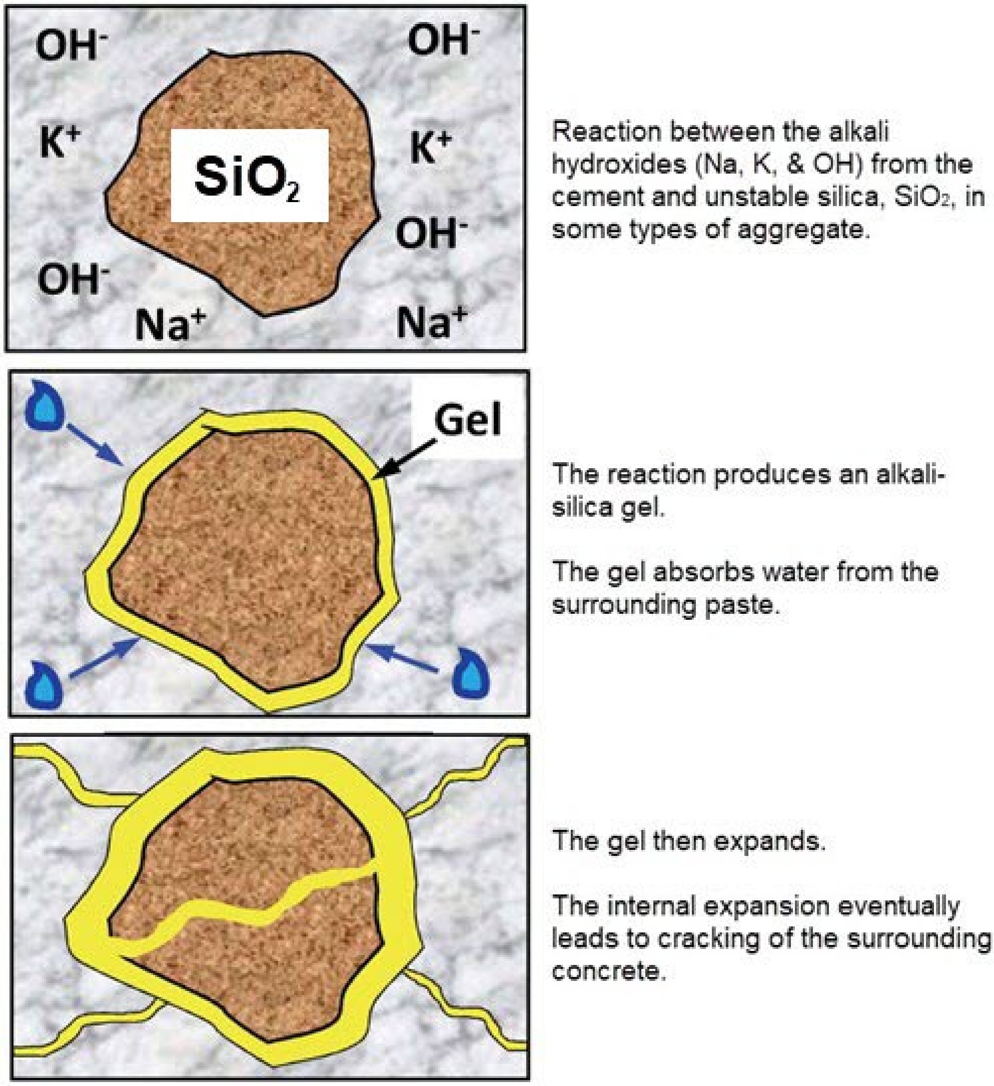


Figure 7.1.27 Sequence of ASR

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 (see Figure 7.1.28). There is no early visible indication of ASR, so lab testing may be necessary to confirm presence of ASR.

Figure 7.1.28 Alkali-Silica Reaction (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 necessary, a Level 2 investigation should be conducted, which consists of documenting information, measuring the Cracking Index (measurement and summation of widths within a set area), obtaining samples, and conducting a petrographic examination. After the first two levels of investigation are complete and further investigation is necessary, a Level 3 investigation is performed. Level 3 investigations should include determining the expansion to date, the current rate of expansion, and the potential for future expansion. It will likely be necessary to perform the test of the structure at the location of the ASR in order to gather all the information necessary 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)

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 (C3A). Ettringite formation is initially formed when water is added to the cement but before 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. Delayed ettringite formation is caused when the temperature is too high during the curing process. 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 necessary. Over time, ettringite formation can compromise the structurally integrity of the concrete as corrosion to the reinforcement is accelerated.

Honeycombs

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. 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 that may be present within the concrete (see Figure 7.1.29). In some cases, honeycombs are the result of insufficient vibration, where the entire concrete mix does not physically reach the formwork surface.

Figure 7.1.29 Honeycombing

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 likely 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 that 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 (see Figure 7.1.30). 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, and parapets.

Figure 7.1.30 Wear in Wheel Paths

Collision Damage

Trucks, derailed railroad cars, or marine traffic may strike and damage concrete bridge members (see Figure 7.1.31). 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 7.1.31 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 7.1.32). 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. Fair condition of abrasion consists of exposed coarse aggregate but the aggregate remains secure in the concrete. Poor condition of abrasion has loose or has popped out aggregate. 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 (refer to Chapter 16).

Figure 7.1.32 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 or a complete failure (see Figure 7.1.33). 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.

Figure 7.1.33 Overload Damage

Reinforcing Steel Corrosion

Due to the chemistry of the concrete mix, reinforcing steel embedded in concrete is typically 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 find 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 will likely cause the concrete to yield, resulting in wider cracks, delaminations, and spalls (see Figure 7.1.34).

Figure 7.1.34 Corroded Reinforcing Bars with Severely Spalled Concrete

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 in prestress members 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 before any measurable section loss due to environmentally induced cracking (EIC).

Environmentally induced cracking in steel prestressing strands can occur when the strands are subject to high tensile stresses in a corrosive environment. Rust stains may be present. The strands, which are typically ductile, undergo a brittle failure due to the combination of the corrosive environment along with the tensile stresses.

A PennDOT forensic study after a 2005 prestress beam failure in Pennsylvania determined that nominal tensile stress can be reduced to approximately 20 to 30 percent based on the condition of the prestress strands (Lehigh). Lightly corroded strands experienced a 20 percent reduction and heavily 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 of EIC is called hydrogen-induced cracking (HIC) and occurs due to hydrogen diffusing into the prestressing steel, during the manufacturing process or from the service environment. 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 enables corrosion of the tendons and thereby reduces the bond between tendon and concrete, 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. The carbon dioxide starts to carbonate the moment the concrete member is fabricated. Carbonation will likely start at the surface and move slowly deeper into the concrete, eventually reaching the reinforcement. The rate of penetration in concrete is proportional to the square root of time. Once it reaches the reinforcement, corrosion may begin to occur. Carbon dioxide also reacts with the alkali in the cement which makes the pore water become more acidic, lowering the pH thus making concrete environment more conducive to corrosion.

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. One example of this is referred to as ettringite formation.

Moisture Absorption

Concrete is porous and absorbs water to some degree. As water is absorbed, the concrete swells slightly. If restrained, the material will likely burst, or the concrete will crack. This type of deterioration is typically 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 typically 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 most likely 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 may 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 likely 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.
- Approach roadway expansion can cause abutment and backwall to crack and spall.

Concrete Sand and Form Streaking

In both cases, the condition is caused by a loss of water through or along the concrete formwork. Concrete sand streaking results from excessive water bleeding along the face of the formwork and may leave a heavy concentration of fine aggregate in vertical streaks. These areas are low in cement content and are susceptible to erode quicker.

Concrete form streaking is caused by water bleeding through the formwork joints. This typically results in a heavier concentration of course aggregate at the surface of the concrete with insufficient cement content.

Unintended Objects in Concrete

Some items that have been 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 most likely damage concrete. Once the temperature is above 212°, moisture in the concrete evaporates and spalls will most likely result. Significant strength reduction (up to 40 percent) occurs at temperatures greater than 550°. High temperatures (above 700°) cause a weakening in the cement paste and degradation of the aggregates which can reduce the strength up to 70 percent.

7.1.7 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 before covering concrete to determine the best possible paint based upon the expected weather/exposure conditions and concrete type.

Several classes of paint can be used to protect concrete. Coatings should not be placed until concrete has properly cured. If applied prematurely or without proper surface preparation, coatings may fail. Common paint types on concrete surfaces typically include:

- Oil-based paint.
- Latex paint.
- Epoxy paint.
- Urethanes.

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 consists of a resin emulsion. Latexes can contain a variety of synthetic polymer binding agents. Acrylic or vinyl latexes provide improved overall performance, in that they are more resistant to alkaline attack than oil-based paint. Latex paints, however, are susceptible to efflorescence. 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 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 before painting.

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 demand 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 7.1.35). These are smooth 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.

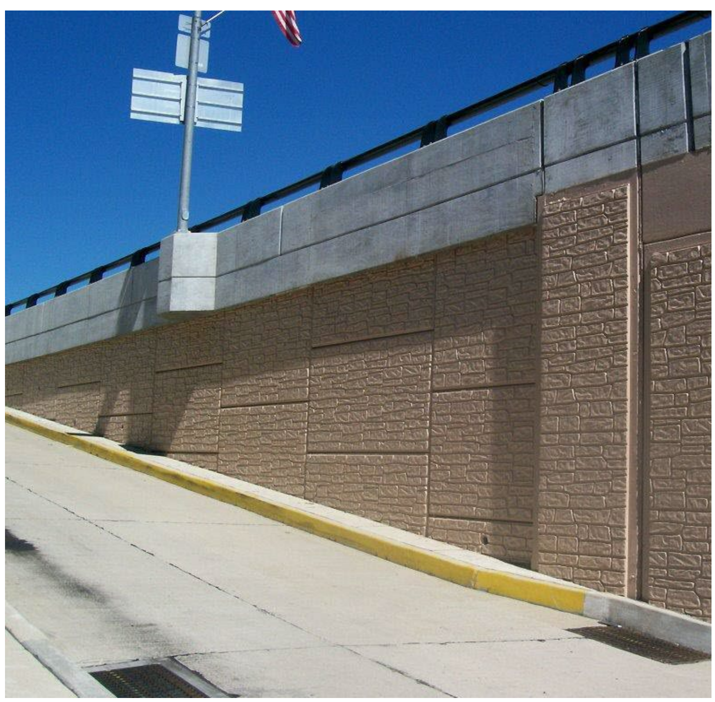


Figure 7.1.35 Anti-Graffiti Coating on Wingwall

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 and Protective Systems

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.
- Stainless steel cladding.
- MMFX steel.
- Fiber Reinforced Polymers.
- 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, and 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 before top- coating with another layer of epoxy or another material. If not removed, the chalking may 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 and the other accelerates due to the electrical current. In this situation, the steel stops corroding, and the zinc has accelerated corrosion.

Stainless Steel and Stainless Steel Cladding

Solid stainless steel and stainless steel cladding (coating) is another form of steel reinforcement protection that is resistant to corrosion. The chromium (Cr) in the coating will likely form a passivation layer chromium oxide (Cr_2O_3) when exposed to oxygen which is not visible to the naked eye. The coating should protect the reinforcement from water and air and the coating should quickly reform if the surface is scratched.

MMFX Steel

MMFX Steel is a corrosion resistant alloy steel reinforcement option. MMFX rebar is comprised of a different chemical composition than tradition steels. The production process modifies the microcomposite microstructure, with a goal of creating a carbide free steel. The steel itself is therefore more resistant to corrosion caused by chlorides, even without a coating.

Fiber Reinforced Polymers

Fiber Reinforced Plastic or Polymer (FRP) rebar is a non-steel reinforcement option. FRP is made from a combination of fiberglass roving and resin. It is a very lightweight material with non-corrosive and non-conductive properties, as it is nonmetallic.

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 enabled 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 and the rectifier is powered on (see Figure 7.1.36).

Figure 7.1.36 Cathodic Protection: Deck Wires Connected to Direct Current

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 in extremely corrosive environments. However, this demands careful monitoring and control, otherwise, it could speed up the corrosion process.

7.1.8 Anticipated Modes of Protective System 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) may 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.

7.1.9 Inspection Methods for Concrete Members

There are three basic methods used to inspect prestressed and reinforced concrete members. Depending on the type of inspection and deficiency, it may be necessary for the inspection team to use only one individual method or all methods. They include:

- Visual inspection methods.
- Physical inspection methods.
- Advanced inspection methods.

Visual Inspection Methods

All concrete surfaces should receive a thorough visual assessment to identify obvious surface deficiencies during a routine inspection or in-depth inspection. The visual inspection methods may be supplemented by physical or advanced inspection methods.

Physical Inspection Methods

Some concrete deficiencies may necessitate physical inspection methods in addition to visual inspection. Inspectors should physically examine the concrete using a measuring tape, crack comparator card, inspection hammer, chain drag, or rotary percussion tool.

Suspect areas should be sounded 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 7.1.37). The inspection team drags this across a deck and makes note of the resonating sounds. A delaminated area will most likely have a distinctive hollow “clacking” sound when tapped with a hammer, rotary percussion tool or revealed with a chain drag. Striking sound concrete should result in a solid “pinging” type sound. Delaminated areas can be delineated with marking keel.

The inspection team should give special observation to document the length and width of cracks found during the physical inspection methods. For typical reinforced concrete members, 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. A ruler or measuring tape should be used to determine the crack length. For prestressed members, crack widths are usually narrower in width. For this reason, a crack gauge should be used, which is a more accurate crack width-measuring device.

Figure 7.1.37 Inspector Using a Chain Drag looking for Delaminated Areas

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.

There are several advanced methods for concrete inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Electrical Resistivity (ER).
- Galvanostatic Pulse Measurement (GPM).
- Ground Penetrating Radar (GPR).
- Half-Cell Potential (HCP).
- Impact Echo (IE).
- Infrared Thermography (IR).
- Linear Polarization (LPR).
- Magnetic Flux Leakage (MFL).
- Magnetometer (MM).
- Rebound and penetration methods.
- Ultrasonic Pulse Velocity (UPV).
- Ultrasonic Pulse Echo (UPE).
- Ultrasonic Surface Waves (USW).

Other methods that may be considered destructive testing methods include the following.

- Concrete cores.
- Borescopes.
- Moisture content.
- Petrographic examination.
- Alkali-Silica Reaction (ASR) evaluation.
- Reinforcing steel sample.
- Rapid chloride permeability testing.

All advanced methods are described in detail in Chapter 17.

Section 7.2 Steel/Metal

7.2.1 Introduction

Steel is a widely used construction material for bridges due to its strength, relative ductility, and reliability. It is found in a variety of members on a large number of bridges. Therefore, the bridge inspector should be familiar with the various properties and types of steel.

7.2.2 Common Methods of Steel Member Fabrication

Rolled Shape Beams

Rolled beams are manufactured in structural rolling mills. The flanges and web are one piece of steel. Rolled beams in the past were generally available up to 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 have bearing stiffeners but no intermediate stiffeners since they are compact. Although rolled beams may not incorporate intermediate stiffeners, they may have connection plates for diaphragms or cross-frames.

Refer to Chapter 4 for additional rolled shapes including rods, bars, plates, angles, channels, and structural tees. Also refer to the AISC *Manual of Steel Construction* for available rolled beam shapes and their properties.

Built-Up Shape Girders

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 typically constructed from welded members and high strength bolts in the field.

7.2.3 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.
- 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 and suspenders (see Figure 7.2.1).
- Steel plates have a wide variety of uses. They are primarily used to construct built-up shapes (see Figure 7.2.2 and Figure 7.2.3).
- Steel bars can be used as primary members such as eyebars in older trusses and arches, or in secondary tension members.
- Rolled shapes are generally 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 hot rolled or cold rolled. The typical rolled shape is an "I" shape. The "I" shape comes in many sizes and weights (see Figure 7.2.4). Other rolled shapes are channel or "C" shapes, angles ("L" shaped), 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. Built-up shapes may be used when an individual rolled shape cannot carry the necessary load or when a unique shape is desired. Built-up shapes may be connected by riveting, bolting, or welding. Common built-up shapes include I-girders, box girders, and truss members (see Figure 7.2.5).

Figure 7.2.1 Steel Cables with Close-up of Cable Cross-Section showing Individual Wires

Figure 7.2.2 Steel Plate Welded to Girder

Figure 7.2.3 Welded I-Girder

Figure 7.2.4 Rolled Beams

Figure 7.2.5 Built-up Girders

7.2.4 Properties of Steel

Physical Properties

When compared with iron, steel has greater strength characteristics, is more elastic, and exhibits greater resistance to the effects of impact and vibration. Iron consists of small amounts of carbon. However, when the carbon content is between 0.1 percent and 2.1 percent, the material is classified as steel. Steel has a unit weight of about 490 pcf.

ASTM and AASHTO define the properties for various steel types. ASTM classifies each type with an "A" designation. AASHTO uses an "M" designation.

Low carbon steel, steel with carbon content less than approximately 0.3 percent, defines some of the most common steel types:

- A7 steel - the most widely used bridge steel up until 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 identified with a "W" for weathering steel.

In 1996 High Performance Steel (HPS), was introduced to bridge construction. Before 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). 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 shallower girder which helps with low clearance problems.

Structural nickel steel (A8) was used widely before the 1960s 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 before the development of low alloy steels in the 1950s. 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 necessary, 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 A441 in 1989).
- A709 - 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.
- A328 steel - used for steel sheet piling for sea walls, cofferdams, and similar applications.
- A690 steel - used for sheet piling or H-piles for sea walls, cofferdams, and similar marine applications.

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.

Mechanical Properties

Some of the mechanical properties of steel include:

- Strength - Steel is an isotropic material in which properties are very consistent and possesses great compressive and tensile strength. There are a wide variety of strengths available depending on the type and grade of steel specified.
- 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 typically 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.
- Corrosion resistance - unprotected carbon steel corrodes (i.e., rusts) readily; however, steel can be protected by coating, metalizing, 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 - steel is made to have a higher toughness for increased resistance to crack propagation.

7.2.5 Protective Systems

Protective systems for steel bridge members include paint, metalizing, galvanizing, weathering steel patina and protection of suspension and stayed cables.

Function of Protective Systems

Protective systems, when applied properly, provide protection against rust or corrosion by providing a barrier, electrical methods, or both.

The following are necessary 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, and zinc chromate), reinforcement of the dry paint film, stabilization against deficiency by sunlight, color, and hardness. Red lead paint is considered hazardous and is not permitted on new bridges or current bridge painting projects. The paint may still be on older bridges. Pigments are generally powder before being mixed into paint. The resin also remains in the dry-cured paint layer. It binds the pigment particles to each other 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. Though 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

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 necessary maintenance can be chosen to provide long lasting results. Oil/alkyd 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 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 limitation of oil/alkyd paints 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. Lead Alkyds may be found on older bridges but is not used for current coating applications. Calcium Sulfonate Alkyds are still in use.

Types of Paint: Vinyl Paint

Vinyl paints are based on various vinyl polymer binding agents dissolved in a strong solvent. These paints cure by solvent evaporation. Vinyl's 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. Vinyl's 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. Vinyl paints are not commonly used now and will mainly be found on older bridges.

Types of Paint: Epoxies

Epoxies utilize an epoxy polymer binder, which forms a tough, resilient film upon drying and curing. Drying is by solvent evaporation. 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 before top coating with another layer of epoxy or another material. If not removed, the chalking may compromise subsequent adhesion.

Types of Paint: 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 necessary in one application. They can be formulated with wetting and penetrating agents, which permit application on minimally prepared steel surfaces.

Types of Paint: 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.

Types of Paint: Zinc-rich Primers

Zinc-rich primers contain finely divided zinc powder and 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 before application, or some coated areas may be deficient in zinc, lowering the substrate protection.

Types of Paint: 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 (enabling 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.

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 likely be accelerated, and the corrosion of the other (e.g., steel) will likely 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, but generally demands 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 necessitate 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 may 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 that 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.

Weathering steel patina generally displays a number of the following characteristics:

- 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.

The color of the surface of weathering steel is an indicator of the protective oxide film. The color changes as the oxide film matures to a fully protective coating.

A yellow orange, for new steel with initial exposure, is tolerable (see Figure 7.2.6). For bridges that have been in service for several years, purple/brown color is tolerable (see Figure 7.2.7 and Figure 7.2.8). However, flaking steel or black color indicates the improper formation of the protective oxide film (see Figure 7.2.9).

Figure 7.2.6 Yellow Orange - Early Development of the Oxide Film (Patina)

Figure 7.2.7 Light Brown - Early Development of the Oxide Film (Patina)

Figure 7.2.8 Chocolate Brown to Purple Brown - Fully Developed Oxide Film

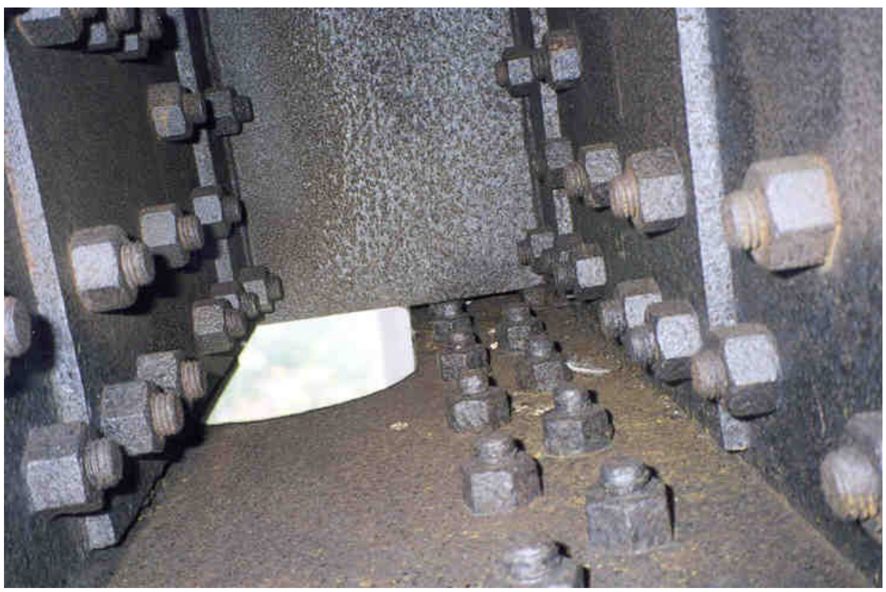


Figure 7.2.9 Black - Non-protective Oxide

An area of steel that 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 a desirable way to record the changing condition of the weathering steel over time. A color coupon may be included in each photograph to enable comparison.

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 can determine the adhesion of the oxide film to the steel substrate. Inspectors should take thickness measurements to monitor remaining section during subsequent inspections. Surfaces, which have granules, flakes, or laminar sheets are examples of non-adhesion.

Table 7.2.3 presents a correlation between the texture of the weathering steel and the degree of protection.

Table 7.2.3 Correlation Between Weathering Steel Texture and Condition

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, ¼ inch in diameter	Initial indication of non-protective oxide
Large flakes, ½ inch in diameter or greater	Non-protective oxide
Laminar sheets or nodules	Non-protective oxide, severe conditions

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 typically used. Red lead pigment was commonly used in the past. Lead constitutes a significant health hazard, and care should be exercised 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 top-coated 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 enables the cable to “breathe” and still provide 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 enable 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 percent.

Cathodic Protection

Another type of protective system is cathodic protection. These systems incorporate an external current source and artificial anode mesh or coating, 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. Cathodic protection systems are to be inspected by a specialist rather than a typical bridge inspector. The inspection team should be trained to verify if the system is running and determine whether the specialist should supplement the inspection.

A conductive solution (water) or electrolyte should 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 typically 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 percent at normal or lower temperatures. The presence of chlorides in the water may accelerate corrosion by increasing the conductivity of the water.

7.2.6 Anticipated Modes of Steel and Protective Systems Deficiencies

In order to properly inspect a steel bridge, the inspector should be able to recognize the various types of deficiencies associated with steel. The inspector also should understand the causes of the deficiencies and how to examine them. There are several common deficiencies that occur on steel bridges:

- Corrosion.
- Fatigue cracking.
- Collision damage.
- Overload damage.
- Heat damage.
- Coating failures.

Corrosion

Corrosion can be described 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.

To properly inspect a steel bridge, the inspector should be able to recognize the various types of steel deficiencies and deterioration. The inspector should also understand the causes of the deficiencies and how to examine them. The most recognizable type of steel deficiency is corrosion (see Figure 7.2.10). Bridge inspectors should 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 wet-dry cycles of exposed steel. When deicing chemicals are present, the effect of corrosion is accelerated.

Figure 7.2.10 Steel Corrosion and Complete Section Loss on Girder Webs

Stress Corrosion

This type of corrosion results from the combine action of a mechanical stress (bending, tension) and a corrosive environment. Each of these parameters alone would not have such a significant effect on the resistance of the metal or would have no effect at all. Stress corrosion occurs when tensile forces expose an increased portion of the metal at the grain boundaries, leading to corrosion and ultimately fracture.

Environmental Corrosion

Environmental corrosion primarily affects metal in contact with soil or water and is caused by formation of a corrosion cell due to salt water i.e., sea water, deicing chemical concentrations, moisture content, oxygen content, and accumulated foreign matter such as roadway debris and bird droppings This type of corrosion can be uniform. Corrosion develops as pits of very small diameter, in the order of a micrometer, and results in a uniform and continuous decrease in thickness over the entire surface area of the metal. The rate of corrosion can be determined by measuring the section loss over time.

Pitting Corrosion

This localized form of corrosion is characterized by the formation of irregularly shaped cavities on the surface of the metal. The diameter and depth depend on several parameters related to the metal, the medium and service conditions. Unlike uniform corrosion, the intensity and rate of pitting corrosion cannot be assessed neither by determining the mass loss. Pitting corrosion can be assessed using three criteria: the density, i.e., the number of pits per unit area, the rate of deepening and the probability of pitting.

Transgranular and Intergranular (Intercrystalline) Corrosion

Within the metal, at the level of the grain, corrosion may propagate in two different ways. It may spread in all directions, corrosion affecting all the metallurgical constituents. This is called Transgranular or Transcrystalline corrosion because it propagates within the grains. It could also follow a preferential pat: corrosion propagates at grain boundaries. This form of intercrystalline corrosion consumes a very small amount of metal, that is why mass loss is not a significant parameter for assessment of this type of corrosion. It is not detectable by a naked eye. When penetrating into the bulk of the metal, it may impact mechanical properties and lead to rupture of members. Stray current corrosion is 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.

Fretting Corrosion

Fretting corrosion takes place on closely fitted parts that 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

Pack rust occurs between two mating surfaces; an increase in volume of rust over the original steel may create localized distortion and possibly cracking.

Crevice Corrosion

Crevice corrosion is a localized corrosion in recesses: overlapping zone for riveting, bolting, or welding, zone under joints and under various deposits. These zones also called crevices and are very tiny and difficult to access for aqueous liquid that is covering the rest of the readily accessible surface.

Exfoliation Corrosion

This is a type of selective corrosion that propagates along a large number of planes running parallel to the direction of rolling or extrusion. Between these planes are very thin sheets of sound metal that are not attacked, but gradually pushed away by the swelling of the corrosion products, peeling off like pages in a book, hence the term exfoliation corrosion. The metal will most likely swell which results in the spectacular aspect of this form of corrosion.

Galvanic Corrosion

When two dissimilar metals are in direct contact in a conducting liquid, experience shows one of the two may corrode. This is called galvanic corrosion. The other metal may not corrode; it may even be protected in this way. Galvanic corrosion does not depend upon metal’s texture or temper.

It will likely occur as soon as the two metals are in contact. The appearance of galvanic corrosion is very characteristic. It is not dispersing like pitting corrosion, but highly localized in contact zone with the other metal. The zone affect by galvanic corrosion often has a shinier aspect than the rest of the surface.

Erosion Corrosion

Corrosion by erosion occurs in moving media. This type of corrosion is related to flow speed of liquid. It leads to local thinning of the metal, which results in scratches, gullies, and undulations, which are always oriented in the same direction, namely the flow direction.

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 7.2.11). This type of cracking can lead to sudden and catastrophic failure on certain bridge types. Therefore, the bridge inspector should know where to look and how to recognize early stages of fatigue crack development.

Figure 7.2.11 Fatigue Crack (entirely through bottom flange and web)

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. Under a cyclic loading condition, repeated in-plane bending can result in load-induced fatigue. 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. 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 diagram and floorbeams, connect. This can lead to fatigue cracking (see Figure 7.2.12).

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 Section 7.3.

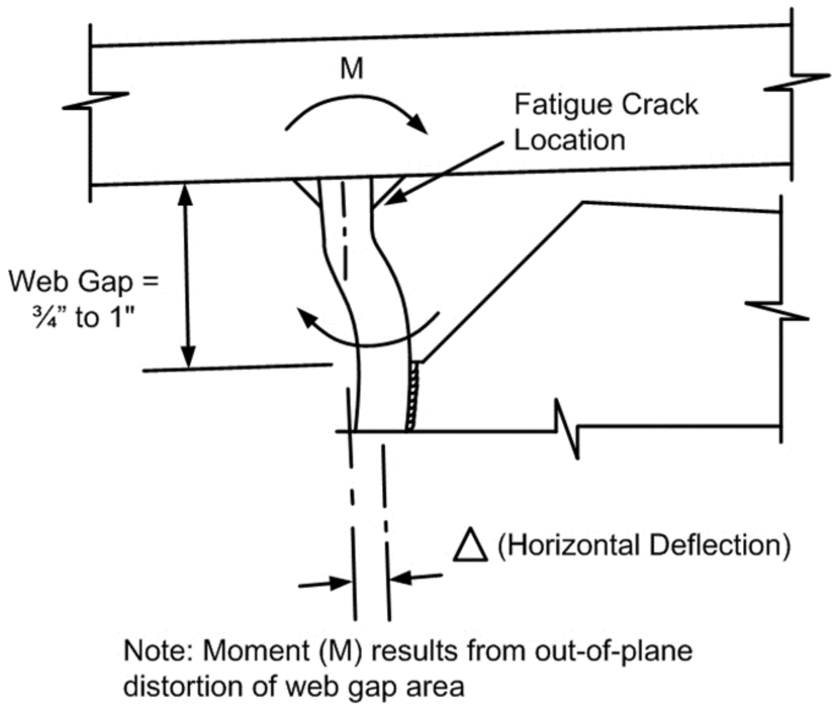


Figure 7.2.12 Distortion Induced Fatigue

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 (refer to Chapter 6). When yielding occurs, steel bends or elongates and remains 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 include elongation and a decrease in cross section, commonly called "necking down".

The primary symptom of plastic deformation in compression members includes buckling, which can manifest in two forms. Buckling can be in the form of a single bow or 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.

The symptoms of plastic deformation for shear members include web buckling and diagonal cracking.

Any excessive vibration or deflection that may occur under traffic should be noted, which can include an overstress.

Collision Damage

Structural members of a bridge that are adjacent to a roadway or waterway traffic are susceptible to collision damage. Indications of collision damage may include scratches, broken, dislocated or distorted members (see Figure 7.2.13).

Figure 7.2.13 Collision Damage on a Steel Bridge

Heat Damage

Steel members undergo serious deformation when exposed to extreme heat (see Figure 7.2.14). 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 7.2.14 Heat Damage

Temperatures affecting steel strength commonly used in bridges are as follows:

- 600 degrees-1000 degrees Fahrenheit (F) - starts to affect strength.
- Above 1000 degrees 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 percent of its value up to 600 degrees F. Between 600 degrees F and 1000 degrees F, the yield strength then further reduces to approximately 75 percent of its yield strength. The modulus of elasticity for steel reduces to 75 percent at 1000 degrees F. Temperatures above 1000 degrees F significantly reduce the strength properties of steel. Diesel fuel and gasoline can produce fires with temperatures in excess of 1500 degrees F and wood burning fires can reach temperatures up to 2000 degrees F.

Coating Failures

The following coating failures are common on steel protective systems:

- Chalking, erosion, checking, cracking, and wrinkling caused by too much paint (see Figure 7.2.15).
- 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 7.2.16). 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 7.2.17). 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 attaching 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. 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.
- Mud cracking can be considered a widely spaced cracking failure, where the breaks in the coating extend to the steel substrate, enabling rapid corrosion (see Figure 7.2.18). Mud cracking is often a phenomenon of inorganic zinc-rich primers that have been applied too thick or an applied on a hot surface. Rapid curing causes the shrinkage that yields the alligatoring, and ultimately, mud cracks.
- Bleeding occurs when soluble colored pigment from an undercoat penetrates the topcoat, causing discoloration.

Figure 7.2.15 Paint Wrinkling

Figure 7.2.16 Rust Undercutting at Scratched Area

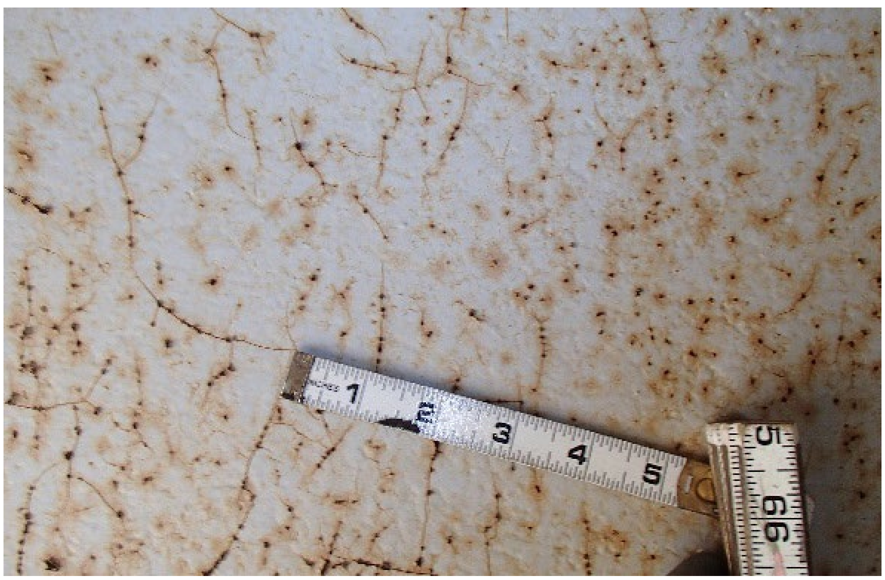


Figure 7.2.17 Pinpoint Rusting

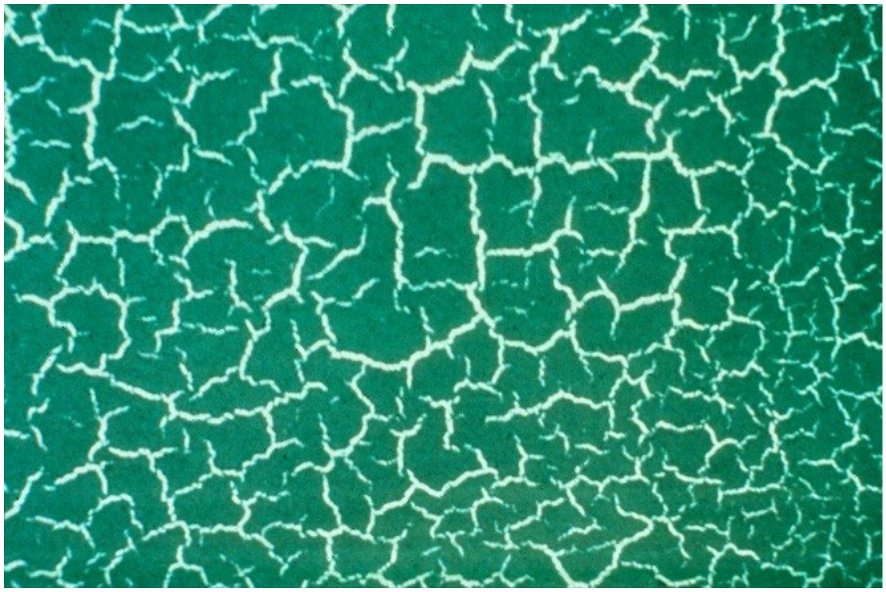


Figure 7.2.18 Mud Cracking Paint

7.2.7 Inspection Methods for Steel and Protective Coatings

There are three basic methods used to inspect steel member and their coatings. Depending on the type of inspection and deficiencies, it may be necessary for the inspector to use only one individual method or all methods. They include:

- Visual inspection methods.
- Physical inspection methods.
- Advanced inspection methods.

Visual Inspection Methods

All steel surfaces and protective coatings should receive a thorough visual assessment to identify obvious surface deficiencies during a routine inspection or in-depth inspection. Nonredundant steel tension members (see Section 7.3) require a hands-on inspection (23 CFR 650.305). A member of the inspection team is to be within arm's reach of NSTMs. Redundant members may have a normal visual inspection. Visual inspection methods may be supplemented by physical or advanced inspection methods.

Physical Inspection Methods

Steel

Some steel deficiencies may necessitate physical inspection methods in addition to visual inspection. For steel members, the main physical inspection methods involve the measurement of deficiencies identified visually. An inspection hammer or wire brush or grinder is used to remove loose paint or rust flakes so accurate measurements of remaining section can be made. Excessive hammering, brushing, or grinding may close surface cracks and make the cracks difficult to find.

Steel members should be physically examined using a measuring tape, calipers, thickness meter (D-meter), to measure the amount of steel remaining. It is important to measure the actual thickness of remaining steel instead of trying to estimate the percentage of section loss.

The inspector should use the inspection hammer, wire brush, rotary percussion hammer or grinder to remove the loose or scaling rust including weathering steel. (See Figure 7.2.19) Once sound metal is reached, the inspector should use the measuring tape, calipers, or thickness meter to measure sound steel. Special observation should be given to document the length of cracks found during the physical inspection methods. A ruler or measuring tape should be used to determine the crack length.

Figure 7.2.19 Inspector using a Hammer to Remove Loose or Scaling Rust

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.

Incomplete removal of mill scale can provide a starting point for corrosion. When mill scale cracks, it enables 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. The exposed surface should be inspected for mill scale, intact or rusted. Inspectors should probe with a knife or other sharp object at weld spatter to check for rusting. Inspectors should also have awareness that red lead may have been used in the past as part of a coating but is no longer satisfactory and is in fact hazardous (OSHA). Areas where paint is removed should be re-coated.

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. The tape test is still qualitative since there is not an actual measurement of adhesion (psi) such as with a pull-off test.

Cathodic Protective System

Cathodic Protective Systems are typically inspected by specialists such as electrical engineers or electricians. Some agencies ask their bridge inspectors to check for tripped circuit breakers after a rainstorm. The moisture of the deck or in the panel box can cause circuits to trip.

Advanced Inspection Methods

If the extent of the deficiency cannot be determined by visual and/or physical inspection methods, advanced inspection methods may be used.

There are several advanced methods for steel inspection that may be performed in the field. They can be done by a bridge inspector or certified NDE technician and include the following.

- Acoustic Emission (AE).
- Crack Propagation Gage (CPG).
- Dye Penetrant Testing (PT).
- Eddy Current Testing (ET).
- Magnetic Particle Testing (MT).
- Magnetic Flux Leakage (MFL).
- Ultrasonic Testing (UT).
- Phased Array Ultrasonic Testing (PAUT).

Other methods that may be considered destructive testing methods include the following.

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

All advanced methods are described in detail in Chapter 17.

7.2.8 Other Metals

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. It is not used in current new bridge construction, but it may, however, be found on 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

Some of the mechanical properties of cast iron include:

- Strength - tensile strength varies from 25,000 psi to 50,000 psi, and 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.

Deficiencies

The primary forms of deficiencies 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

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, and 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 that is less likely to progress to flaking and scaling than 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.

Deficiencies

The primary forms of deficiencies 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, but some have been constructed in recent years. Also, aluminum has been used to replace iron or steel members for rehabilitation projects. Aluminum weighs less than steel and may enable greater live loads on the rehabilitated structures.

Properties

The properties of aluminum are generally similar to those of steel. However, a few notable differences exist:

- Light 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 necessitates special procedures.
- Durability - aluminum is durable.
- Expense - aluminum is more expensive than steel.

Deficiencies

The primary forms of deficiencies in aluminum are similar to those in steel and:

- 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. When two dissimilar metals are in contact, one typically acts as the anode and the other the cathode. The result will most likely be an accelerated corrosion rate.

Section 7.3 Nonredundant Steel Tension Members

7.3.1 Introduction

Since the 1960s, three notable bridge collapses highlighted the importance of rigorous inspection of nonredundant steel tension members that would identify localized defects that could lead to partial or total collapse. 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 7.3.1). The collapse was due to stress corrosion and corrosion fatigue that enabled 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 typically 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 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 7.3.1 Silver Bridge Collapse

The Silver Bridge collapse is discussed further in Chapter 10. More information on the collapse can be found here: https://transportation.wv.gov/highways/bridge_facts/Modern-Bridges/Pages/Silver.aspx

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 7.3.2). The pin-and-hanger assembly failed at one location on one of the two girders. The forces were redistributed and caused an overstress that led to the bridge collapse.

Figure 7.3.2 Mianus River Bridge Collapse

The Mianus River Bridge collapse is discussed further in Chapter 10. More information on the collapse can be found here: https://connecticuthistory.org/mianus-river-bridge-collapses-today-in-history/

On August 1, 2007, the I-35W Mississippi River Bridge in Minneapolis, Minnesota collapsed (see Figure 7.3.3). The cause of this deck truss collapse was due to a failed gusset plate.

Figure 7.3.3 I-35W Mississippi River Bridge Collapse

The I-35W Bridge collapse is discussed further in Chapter 10. More information on the collapse can be found in the National Transportation Safety Board (NTSB) report, *Highway Accident Report – Collapse of I-35W Highway* Bridge as well as the MNDOT website here: https://www.dot.state.mn.us/i35wbridge/index.html.

The above catastrophes resulted due to a failure of bridges with nonredundant steel tension members. 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 nonredundant steel tension member by sight or based on previous reports and drawings. The National Bridge Inspection Standards (NBIS) require that all nonredundant steel tension members on a bridge be identified, an inspection interval be described, and the inspection methods be listed before an inspection (23 CFR 650.313).

Nonredundant Steel Tension Member

According to the NBIS, a nonredundant steel tension member (NSTM) is a primary steel member fully or partially in tension, and without load path redundancy, system redundancy, or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse (CFR 650.305). Bridges that contain NSTMs require an NSTM Inspection.

Reviewing Member Forces

Two criterions exist for a bridge member to be classified as an NSTM. The first criterion deals with the forces in the member. Members or elements that are in tension meet the first criterion. The four types of member forces are presented in Chapter 6 and include:

- Axial forces (compression and tension) - Compression acts along the longitudinal axis of a member and tends to "push" the member from both ends. Tension acts along the longitudinal axis of a member and tends to "pull" the member apart.
- Shear - Equal but opposite transverse forces that 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 an NSTM is that its failure may cause a total or partial collapse of the structure. Therefore, recognition and identification of a bridge's degree of redundancy is crucial.

Redundancy is the quality of the bridge that enables it to perform its design function in a damaged state.

Redundancy means that if a member or element fails, the load previously carried by the failed member is redistributed to other members or elements and prevents failure of the bridge. 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.
- Internal redundancy.
- System 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 7.3.4).

Load path redundancy can usually be determined visually. Definitive determination of load path redundancy requires structural analysis with members eliminated in turn to determine resulting stresses in the remaining members.

Determination of load path redundancy in transverse members, such as floorbeams, cannot be made by a simple count of members, and would be dependent on structural characteristics such as spacing, continuity, and composite action. In this case, determination of load path redundancy necessitates structural analysis.

Figure 7.3.4 Load Path Redundant Girder Bridge

Internal Redundancy

Internal redundancy exists when a bridge member contains three or more elements that are mechanically fastened to each other 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 Figure 7.3.5 and Figure 7.3.6.

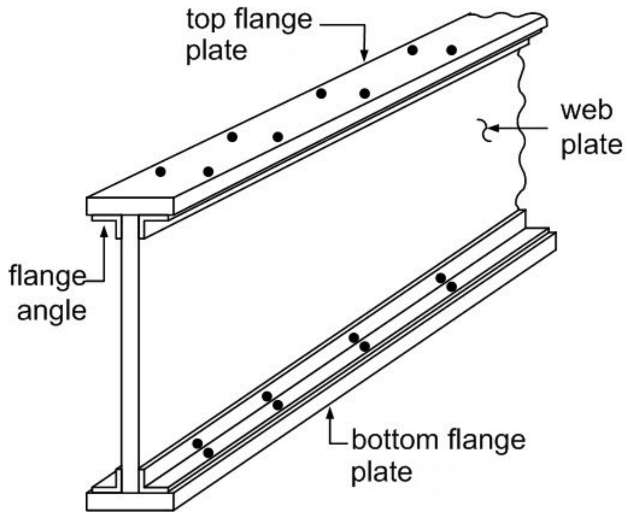


Figure 7.3.5 Internally Redundant Riveted I-Beam

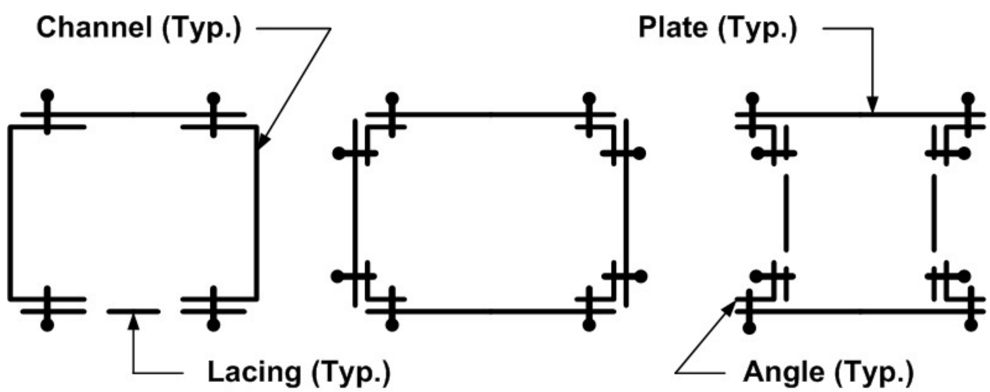


Figure 7.3.6 Internally Redundant Riveted Box Shapes

Internal redundancy of a member can be decreased or eliminated by repairs that involve welding. The welds provide paths for cracks to propagate from one element to another. A truss bottom chord member consisting of multiple eyebars connected at a pin could be internally redundant as well (see Figure 7.3.7).

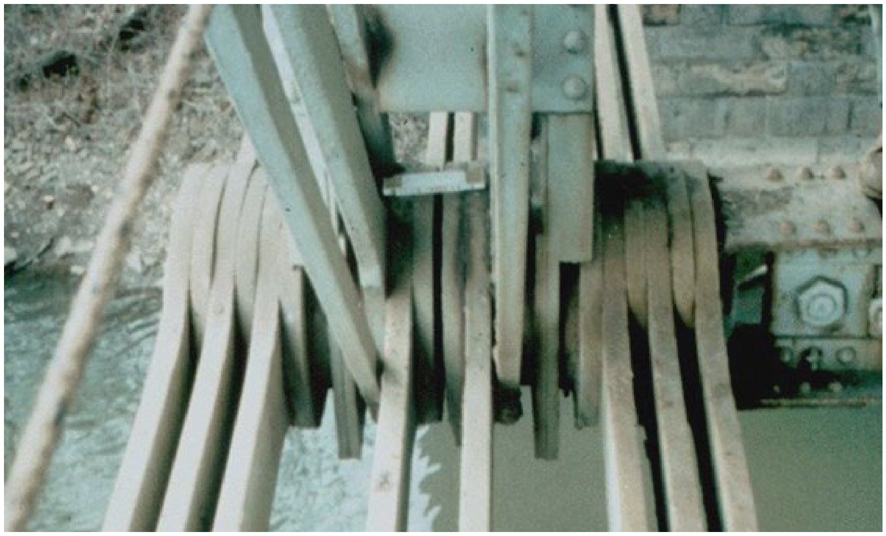


Figure 7.3.7 Internally Redundant Eyebar Connection

System Redundancy

System redundancy is a redundancy that exists in a bridge system without load path redundancy, such that fracture of the cross section at one location of a primary member will not cause a portion of or the entire bridge to collapse (23 CFR 650.305). System redundancy cannot be determined in the field or from bridge plans. A refined analysis, with methodology and evaluation criteria reviewed by FHWA, is necessary to determine whether a bridge exhibits system redundancy.

Continuous bridges are an example of possible system redundancy. Continuity of load path from span to span may provide system redundancy if a member failure occurs in an interior span where the loading can be safely distributed to the adjacent spans, preventing failure. The degree of system redundancy can only be determined through an in-depth analysis. Continuous bridges also may not be entirely system redundant. In the end spans, 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 and system redundancy is not present.

For in-service inspection protocol, owners may go beyond the simple, conservative NBIS definition of NSTMs based on a load path redundancy assessment alone if the member was fabricated to meet the *American Welding Society Fracture Control Plan (AWS FCP)* with an application of refined analysis techniques.

A member that was fabricated according to an AWS FCP and need not be considered an NSTM for in-service inspection would be considered a System Redundant Member (SRM). A System Redundant Member (SRM) should be designated on the design plans with note to fabricate them in accordance with AWS. The criteria, assumptions, and the refined analysis used to determine the system redundancy condition should be retained and included in the inspection records or permanent bridge file. Changes in conditions of bridge elements or loading on the bridge could result in SRMs becoming NSTMs in the future and requiring an NSTM inspection; therefore, it is vitally important to retain the refined analysis records and revise them as necessary to account for these changes over the life of the structure.

Nonredundant Configurations

Bridge inspectors can usually determine through visual cues whether a member has load path redundancy. System and internal member redundancy demand more refined analysis which is not readily apparent to bridge inspectors. Unless specific direction is provided to the bridge inspector regarding system redundancy, the bridge inspector should only consider load path redundancy when determining if a member is an NSTM. The bridge inspector may consider all bridges with less than three load paths to have no load path redundancy and therefore members may be NSTMs (see Figure 7.3.8).

Figure 7.3.8 Nonredundant Two Girder Bridge

7.3.2 Failure Mechanics

Fatigue

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

Fatigue issues often present as small defects and can propagate quickly if unaddressed. Describing the process by which a member fails when subjected to fatigue is called 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 fracture of an NSTM 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 necessary 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 typically considered for fatigue life calculations for primary bridge members.

The number of cycles necessary to initiate a fatigue crack is the fatigue-crack-initiation life. The number of cycles necessary 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 previous plastic deformation (see Figure 7.3.9). Once a brittle fracture occurs, the surface of the fracture is flat.

Figure 7.3.9 Brittle Fracture of Cast Iron Specimen

Ductile fracture is generally preceded by local plastic deformation of the net uncracked section. This plastic deformation results in distortion of the member, providing some visual warning of the impending failure which is the distorted shape that 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 7.3.10).

Figure 7.3.10 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 nonredundant steel tension members, the transition temperature is 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 typically 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 fatigue-prone details may be 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. When it occurs, the failure of a nonredundant steel tension member is very likely 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 is the ability of a material to resist crack propagation 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, along 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 7.3.11). This test measures the amount of energy absorbed by a test specimen before failure. The Charpy V-notch test specifications vary depending on the type of steel, type of construction, whether welded or mechanically fastened, and the applicable minimum service temperature (ASTM E23).

Figure 7.3.11 Charpy V-notch Testing Machine

7.3.3 Factors Affecting Fatigue Crack Initiation

Most critical conditions for fatigue crack initiation are those that 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. Proper 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 enabling the parts to flow 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, which are sometimes referred to as butt welds, are typically used when the members to be connected are lined up edge to edge or are in the same plane (see Figure 7.3.12). Full penetration groove welds extend through the entire thickness of the piece being joined. 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 an improved fatigue strength category for descriptions of AASHTO Fatigue Categories.

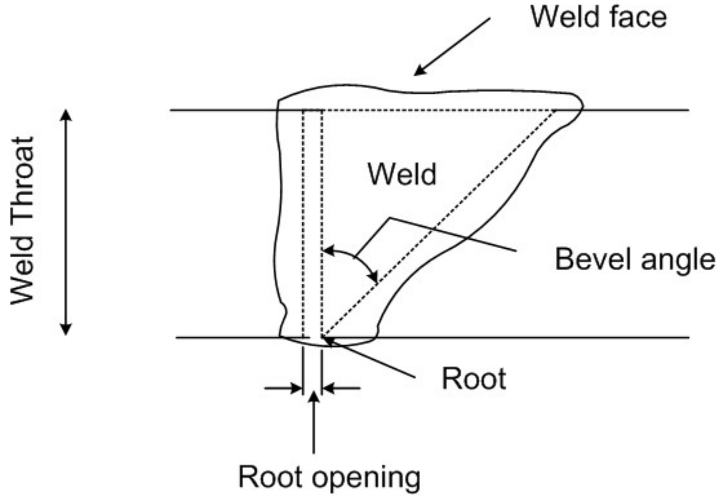


Figure 7.3.12 Groove Weld Nomenclature

Fillet welds connect members that overlap each other or are joined edge to face of plate, as in plate girder assembly of web and flange plates (see Figure 7.3.13). Fillet welds are the most common type of weld because large tolerances in fabrication are satisfactory when members are lapped over each other instead of fitted to each other as in groove welds.

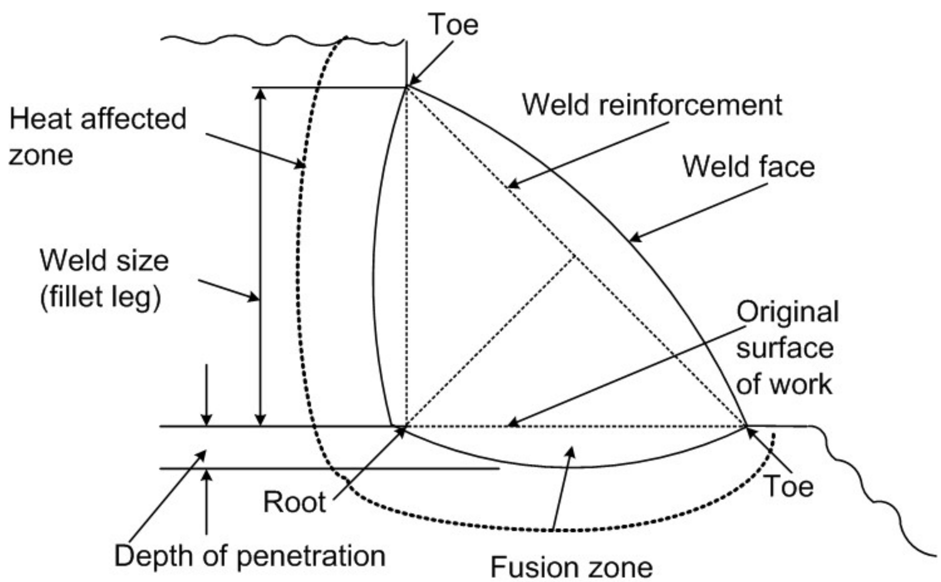


Figure 7.3.13 Fillet Weld Nomenclature

Plug welds pass through holes in one member to another, with weld metal filling the holes and joining the members to each other (see Figure 7.3.14). 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 except for limited use in web reinforcement plates (doubler plates) on girder webs at pin-and-hanger locations because they are fatigue-prone due to the high degree of constraint and the prominence of weld flaws and slag inclusions.

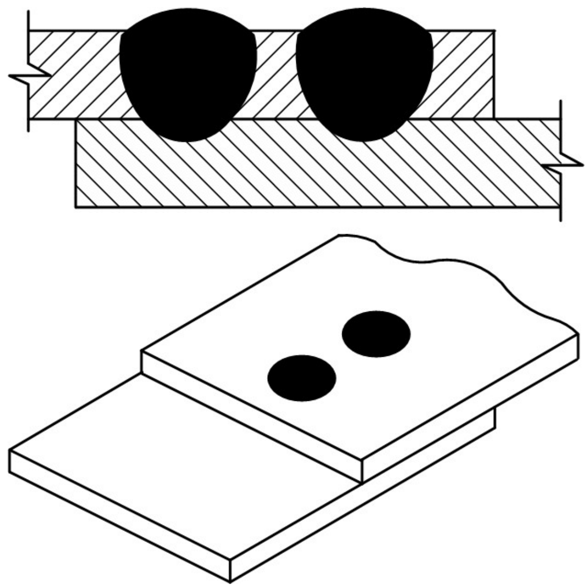


Figure 7.3.14 Plug Weld Schematic

Tack welds are small welds commonly used to temporarily hold pieces in position during fabrication or construction (see Figure 7.3.15). They are often made carelessly, without proper procedures or preheating, and can be a fatigue-prone detail when found on a tension member.

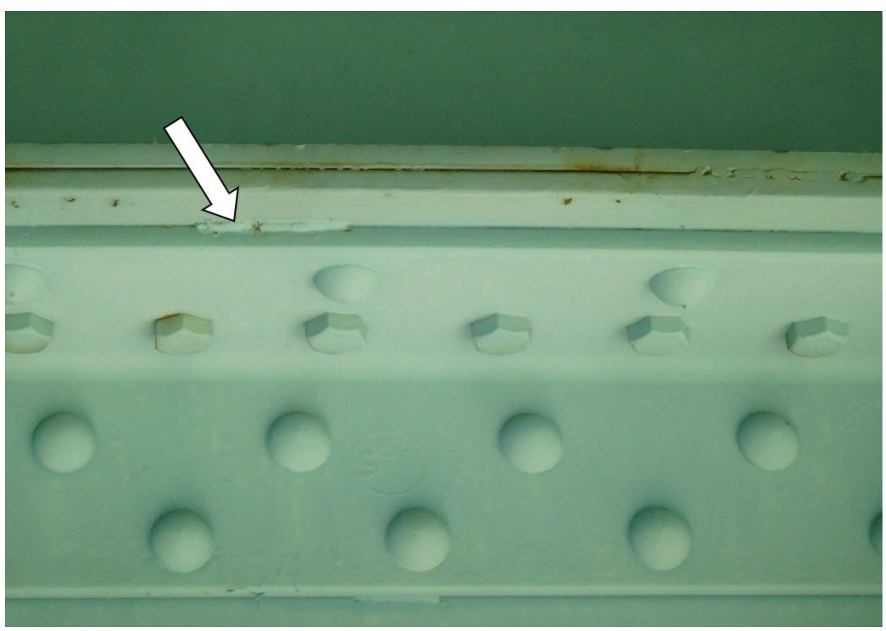


Figure 7.3.15 Tack Weld

Both plug and tack welds are smaller than fillet and groove welds, but they can be a major cracking source to bridges. They tend to be more fatigue-prone than groove and fillet welds. Cracks typically 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 7.3.16):

- Butt.
- Lap.
- Tee
- Edge.
- Corner.

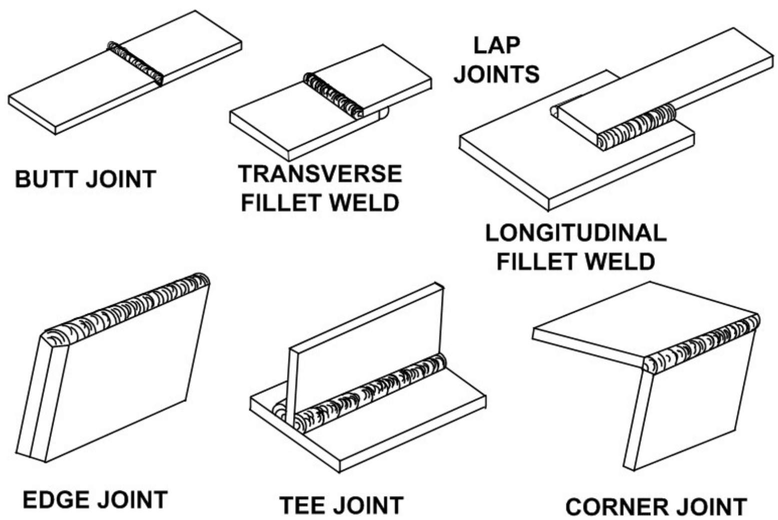


Figure 7.3.16 Types of Welded Joints

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 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 can vary in size from 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 Figure 7.3.17 and Figure 7.3.18).

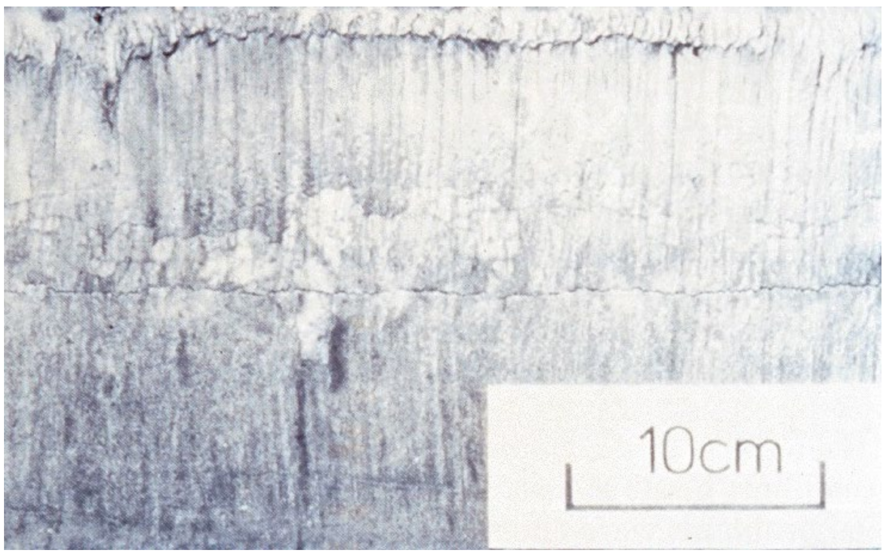


Figure 7.3.17 Exposed Lamination in Steel Slab

The centerline crack in Figure 7.3.17 may have resulted from a shrinkage cavity like that shown in Figure 7.3.18 which was not forged and melded completely in the hot rolling process.

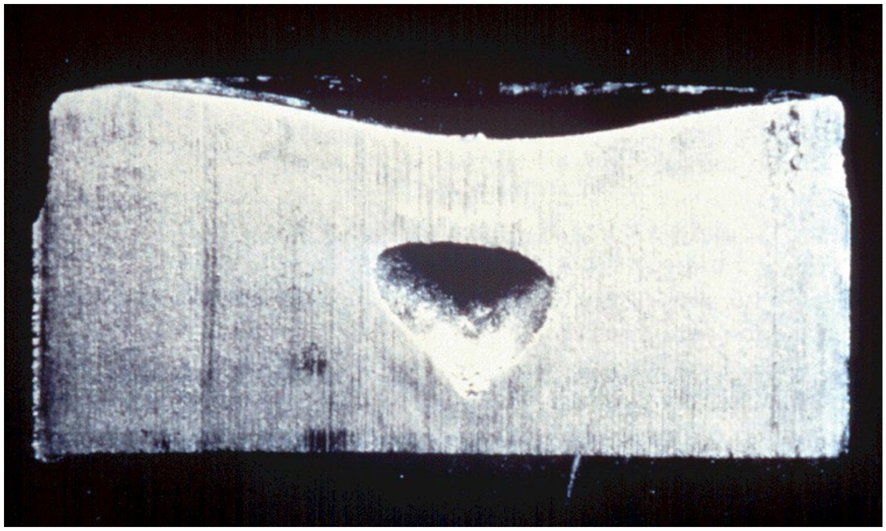


Figure 7.3.18 Shrinkage Cavity in Steel Billet

Fabrication Flaws

Fabrication flaws occur when members are joined to each other 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 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 7.3.19). Incomplete penetration welds are 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.

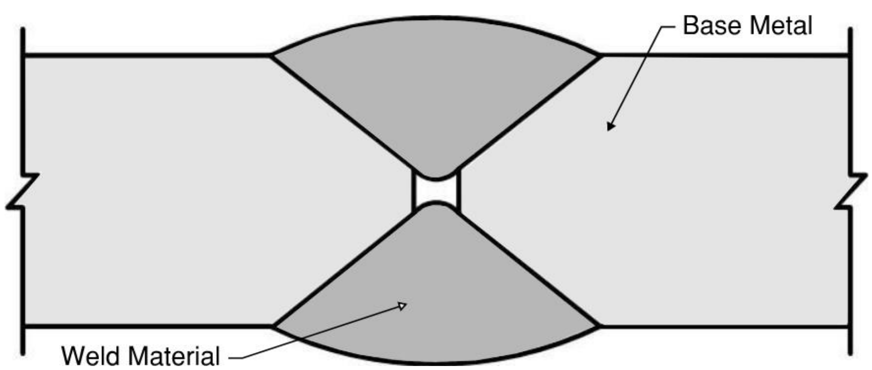


Figure 7.3.19 Incomplete Penetration of a Double V-Groove Weld

Lack of fusion is a condition in which boundaries of unfused metal exist between the base metal and weld metal or between adjacent layers of weld metal. 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 inclusion occurs when nonmetallic matter is inadvertently trapped between the weld metal and the base metal (see Figure 7.3.20). Slag from the welding rod shield may be forced into the weld metal by the arc during the welding operation. If large, irregular inclusions or lengthy lines of inclusions are present, crack initiation at a lower stress could begin and the strength of the weld may be considerably reduced. However, small, isolated globe-shaped inclusions do not seriously affect the strength of a weld, but can be a point of crack initiation.

Porosity is the presence of cavities in the weld metal caused by entrapped gas and takes the form of small spherical cavities, scattered throughout the weld or clustered in local regions. 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.

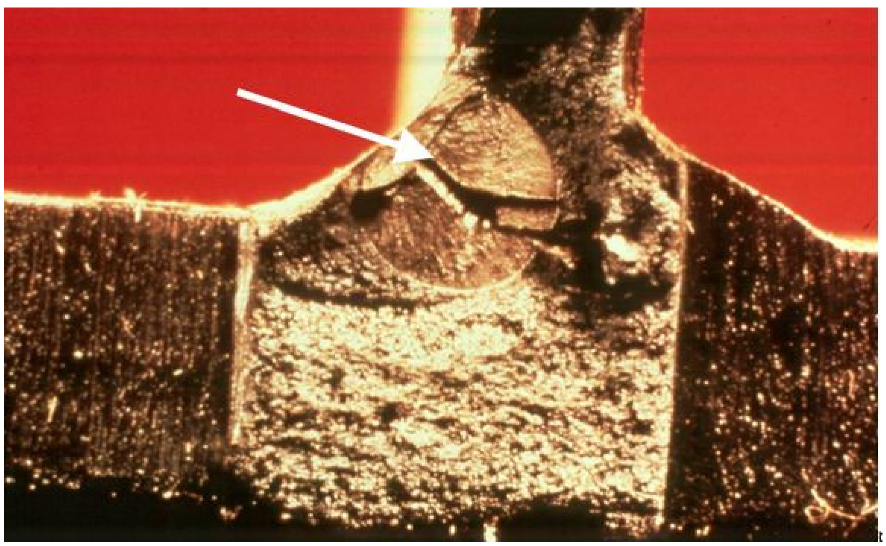


Figure 7.3.20 Web to Flange Crack due to Fillet Weld Slag Inclusion

As presented previously plug welds are sometimes found in bridge members. In some cases, they were made to fill mis-drilled 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 7.3.21).

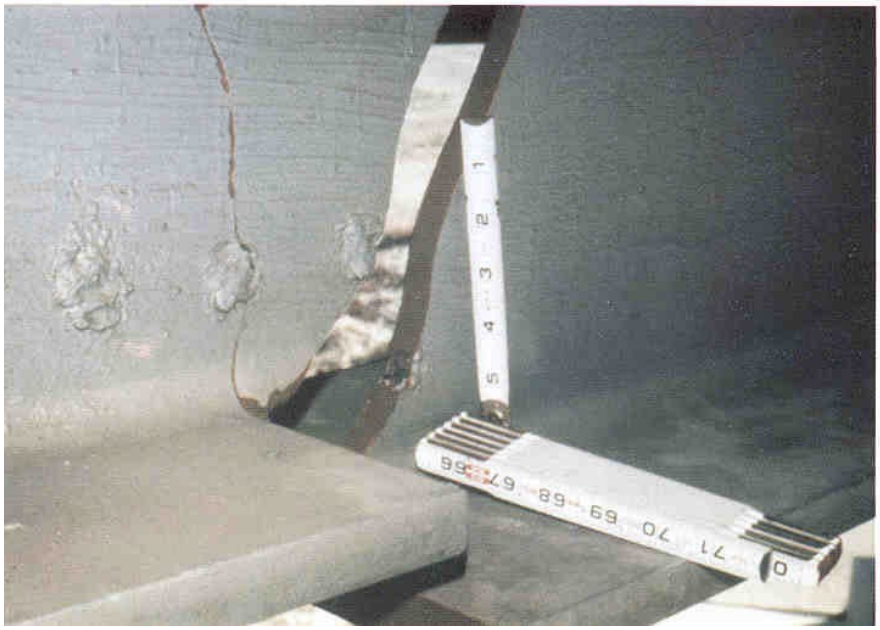


Figure 7.3.21 Crack Resulting from Plug Welded Holes

Visible weld deficiencies may include the following.

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 necessary 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 specifications.

Undercutting is the condition in which a local reduction in a section of base metal occurs alongside the weld deposit. This may happen 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 7.3.22). 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.

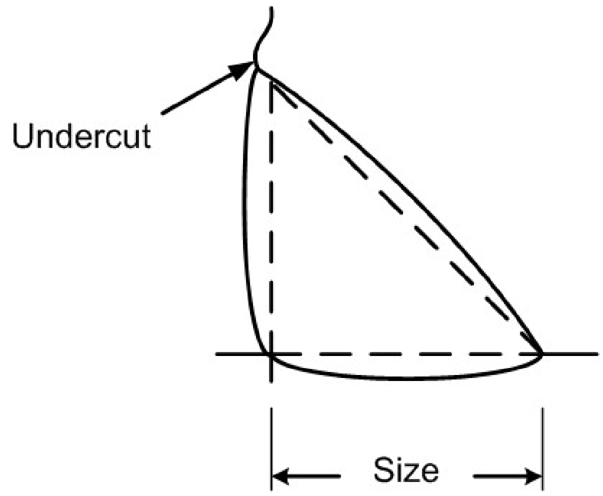


Figure 7.3.22 Undercut of a Fillet Weld

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 7.3.23). 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.

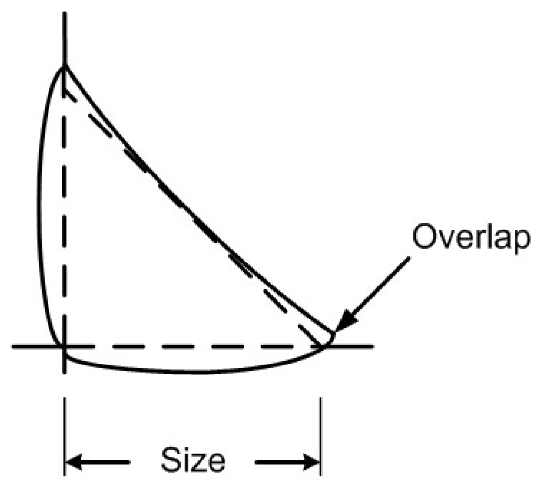


Figure 7.3.23 Overlap of a Fillet Weld

Cleanliness of the joint is important. 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 to each other properly.

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 (see Figure 7.3.24).

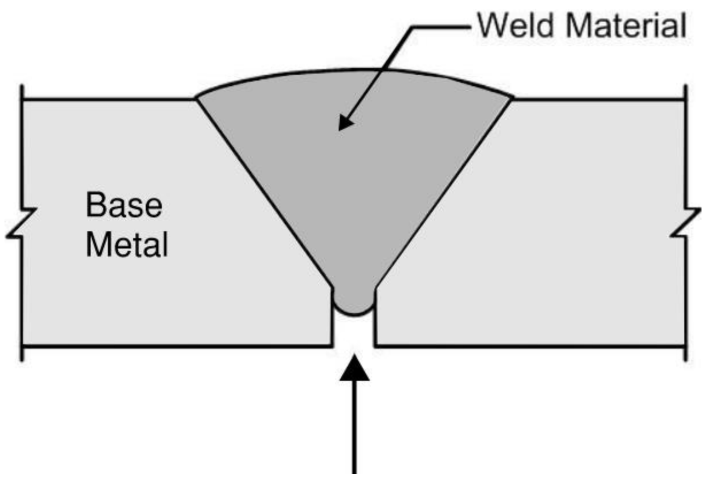


Figure 7.3.24 Incomplete Penetration of a V-Groove Weld

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. Typically, 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, 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 stress 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 7.3.25).
- Flame cuts - Flame cutting, although fast, creates large surface discontinuities that are stress risers. The surfaces of flame cut plates in tension are to be ground smooth in the direction of the tensile stress.
- 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 7.3.26).

Figure 7.3.25 Crack Arrest Hole in Stringer Web at Coped Flange Location

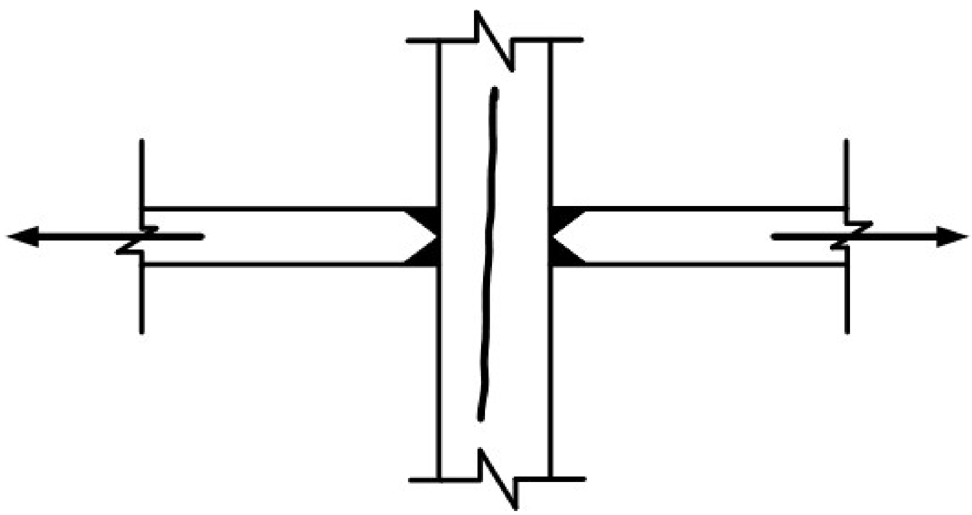


Figure 7.3.26 Thick plate with Two Plates Welded to it and Showing a Lamellar Tears

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.
- Nicks, notches, and indentations.
- Tack welds.

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.

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.

Similar to plug welds, tack welding was a common practice in the mid-1950s and through the early 1960s and was applied to hold members to each other during fabrication and 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 narrow width and long, so they may not interfere with subsequent 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.
- Indiscriminate welds.
- Corrosion.
- Improper heat straightening.

Some members may be prone to collision damage by errant vehicles that may nick, tear, and excessively stress the steel (see Figure 7.3.27).

Figure 7.3.27 Severe Collision Damage on a Fascia Girder

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.

Deep corrosion pits can develop in structures that are improperly detailed for corrosion control, poorly maintained, or left unpainted.

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.

In summary, bridges can contain significant flaws or fatigue-prone details that can be the point of initiation of fatigue cracking and possibly result in fracture. Fatigue-prone details should be identified before a nonredundant steel tension member inspection.

7.3.4 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 observe before serious distress develops in the member. Large magnitudes of cyclic stresses, corresponding to a high frequency of occurrence or to a long exposure time, typically generates fatigue cracks. 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 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 7.3.28). The weight or dead load of the bridge produces a constant stress and not included in this stress range. 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.

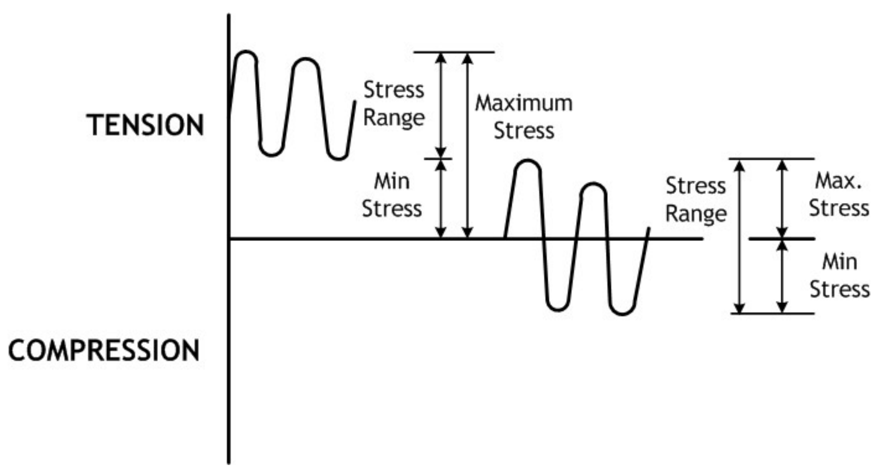


Figure 7.3.28 Applied Tensile and Compressive Stress Cycles

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 fatigue-prone details used in the connections of bridges. AASHTO has chosen some typical details, or illustrative examples (see Table 7.3.4). These illustrative examples are used to help determine AASHTO Fatigue Categories are discussed in Section 7.3.5.

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 provided higher stress ranges than the lower category details). In other words, details of higher fatigue strength categories (A & B), are provided higher stress ranges than those in the lower category details (D through E').

Other factors influencing the initiation of fatigue cracks include material fracture toughness, of both the base metal and weld metal, as well as the ambient temperature. Cracking is more likely in colder temperature.

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. If the thickness of cover plate is greater than 0.80 inches, the fatigue resistance of the weld is of greater concern.

One or more cracks can initiate from microscopic flaws or deficiencies at the weld toe of the transverse end weld (see Figure 7.3.29). Such cracking may then advance in three stages:

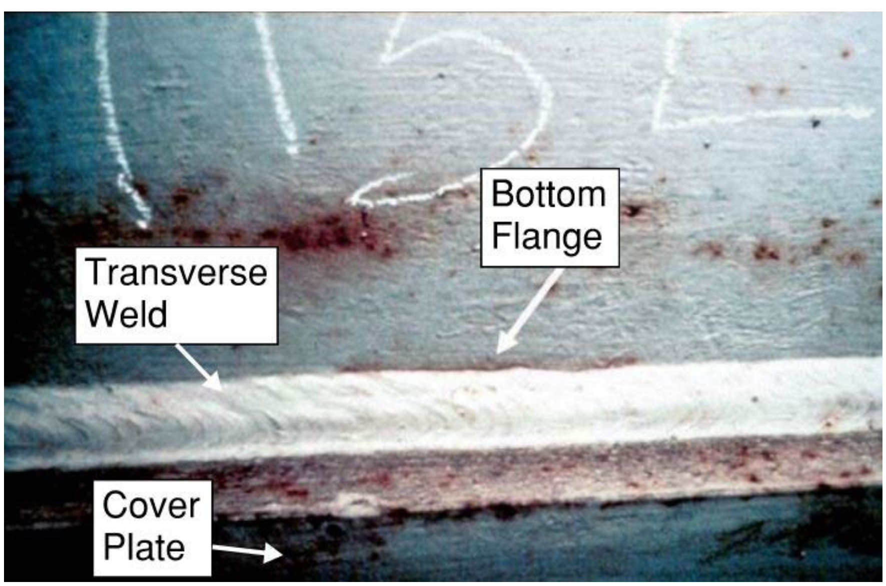


Figure 7.3.29 Part-through Crack at a Cover Plated Flange

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 7.3.30).

Figure 7.3.30 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 percent 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 7.3.31).

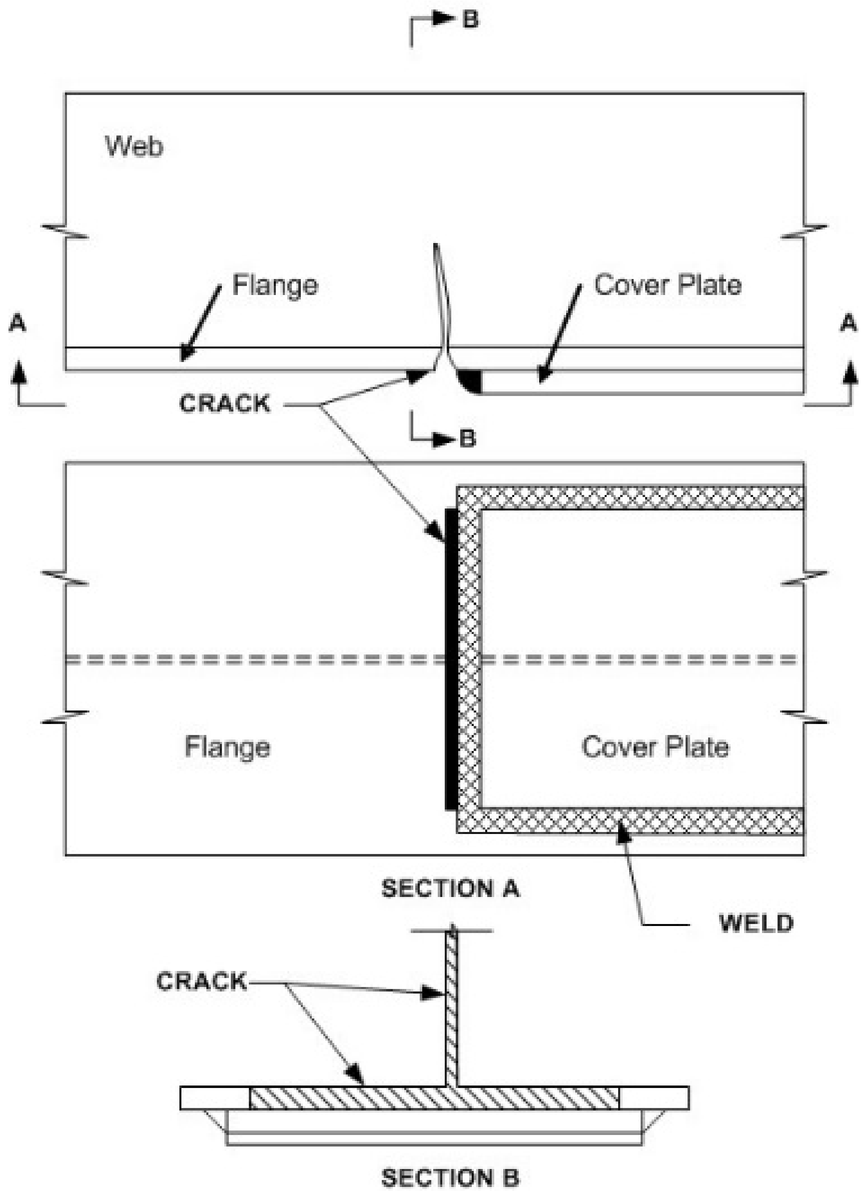


Figure 7.3.31 Through Crack Growth at Cover Plate Welded to Flange

The crack is readily visible as a through-the-thickness crack on both the top and bottom surfaces of the flange (see Figure 7.3.32).

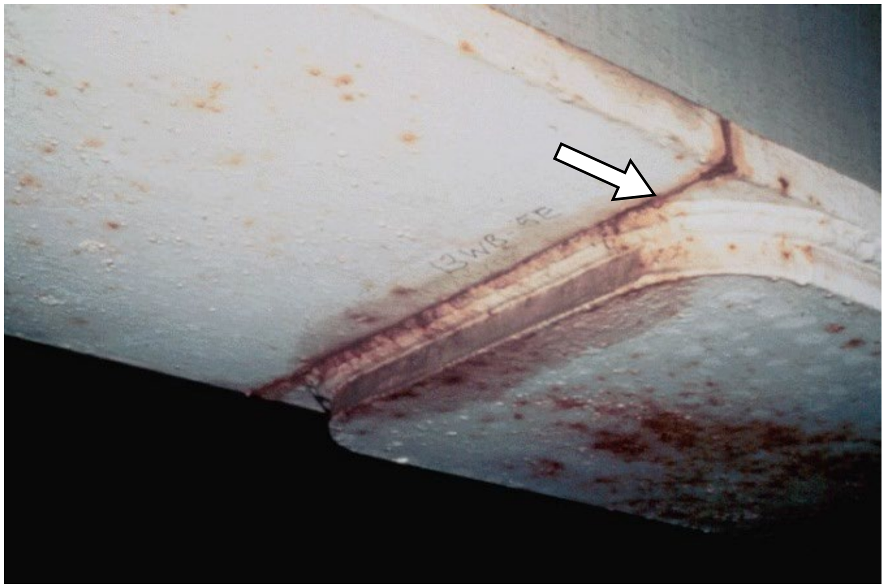


Figure 7.3.32 Through Crack at a Cover Plated Flange

Approximately five percent of the fatigue life is left for growing the Stage 2 through crack (see Figure 7.3.33).

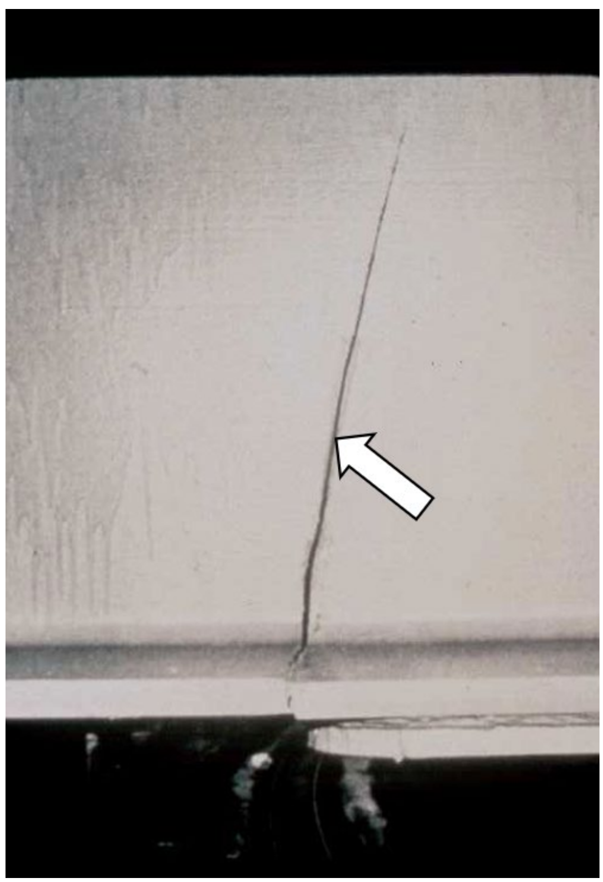


Figure 7.3.33 Through Crack Propagated into the Web

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 steel member fully or partially loaded in tension and is nonredundant, the span, or a portion of it, likely collapses.

The brittle fracture surface appears crystalline or uneven, and often reveals a herringbone pattern oriented toward the point of fracture initiation (see Figure 7.3.34).

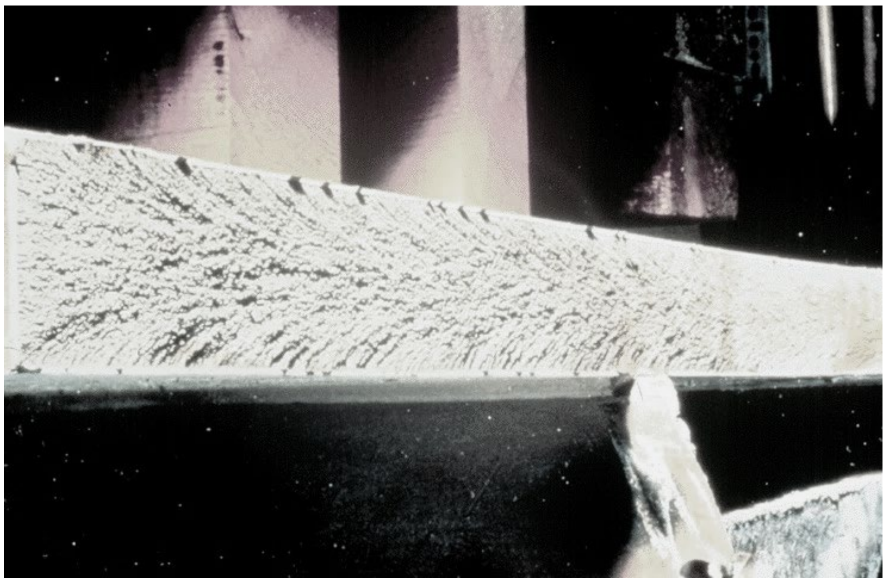


Figure 7.3.34 Brittle Fracture - Herringbone Pattern

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.

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 (see Figure 7.3.35). This type of crack grows in three stages:

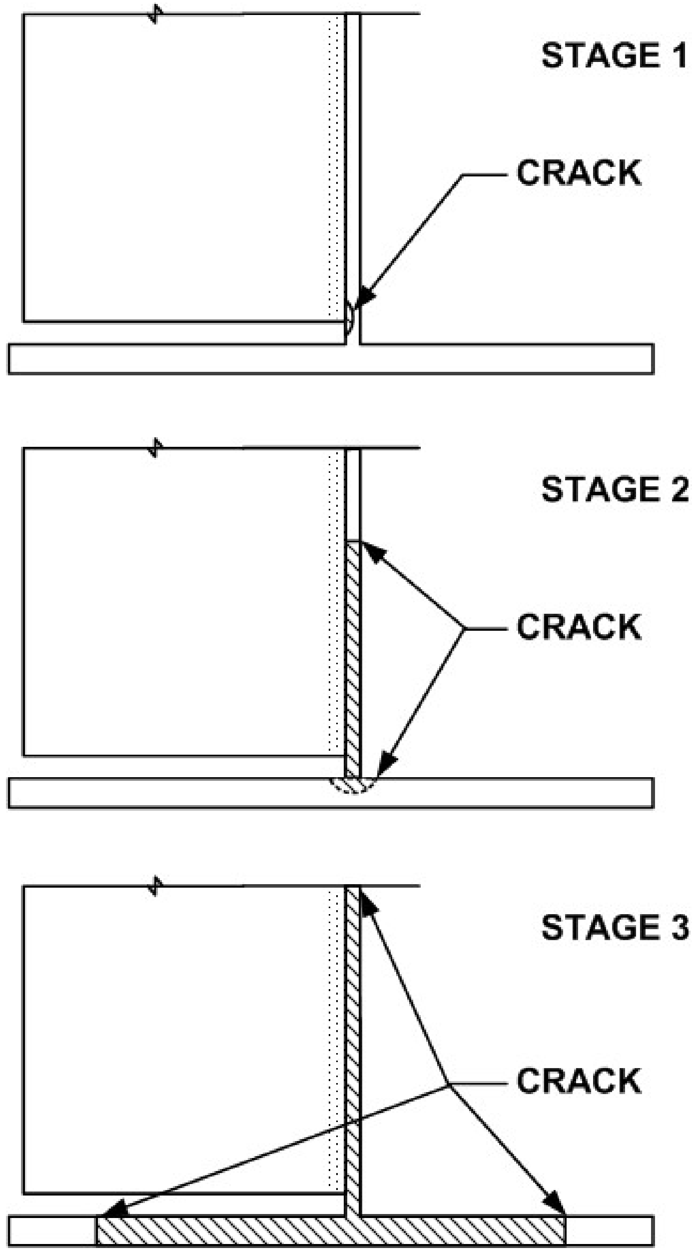


Figure 7.3.35 Crack Growth at Transverse Stiffener to Web and Flange

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 7.3.36).

Figure 7.3.36 Web Gap Crack between Bottom Flange and Vertical Stiffener

This type of stiffener crack expends about 95 percent 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 that propagates up and down the web (see Figure 7.3.37). The through crack can be readily seen on both sides of the web. The stiffener crack expends about five percent of the fatigue life propagating in Stage 2.

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

Bring any web/transverse stiffener cracks discovered to the immediate observation of a bridge owner.

Figure 7.3.37 Through Crack in the Web

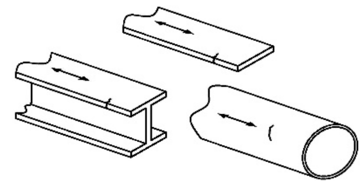
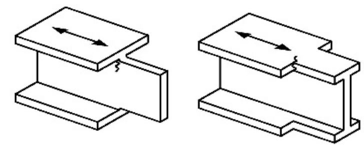
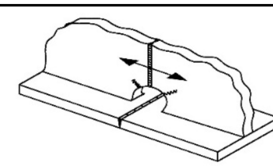
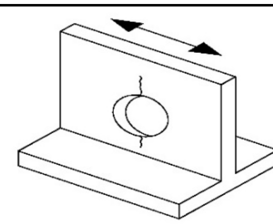
7.3.5 AASHTO Detail Categories for Load-Induced Fatigue

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 – Detail Categories for Load-Induced Fatigue (see Table 7.3.4). 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.

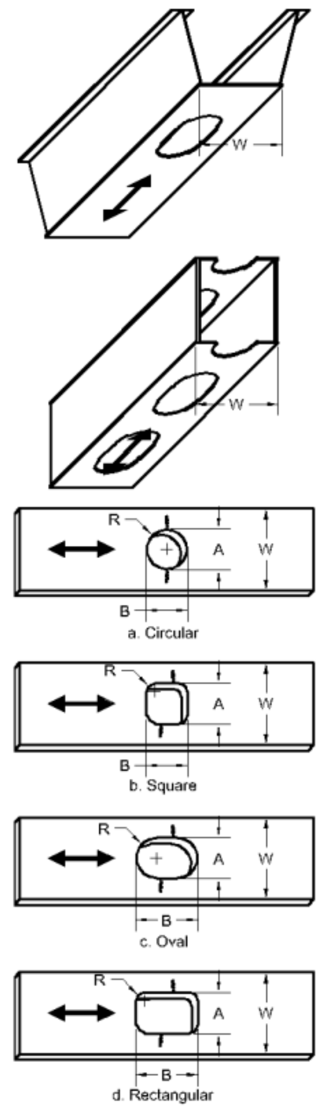
Each letter represents a rating given to a detail that indicates its level of fatigue strength, Category A offering the highest and Category E' having the lowest resistance. Note that the 1998 *AASHTO Bridge Design Specifications* (2nd Edition) eliminated Category F. Category E' can be conservatively 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 for a particular fatigue-prone detail.

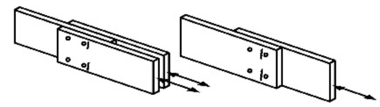
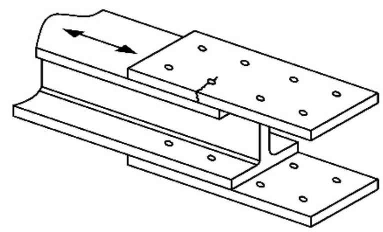
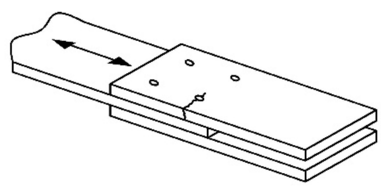
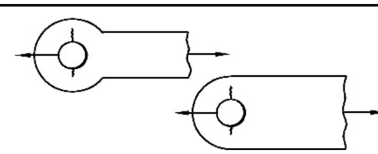
When used in NSTM inspections, these fatigue categories serve as a reminder of which details are more likely prone to fatigue cracking. They also prioritize the level of effort expanded to inspect each detail. The AASHTO Detail Categories are defined as follows.

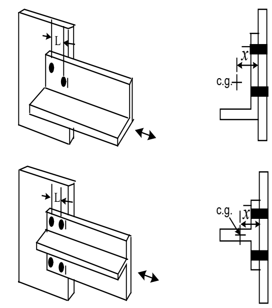
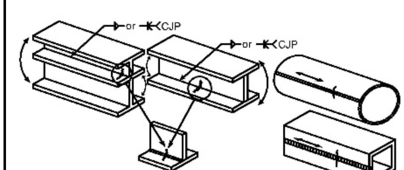
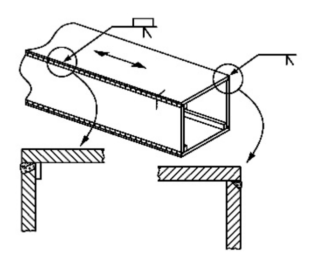
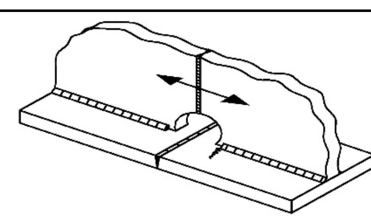
Table 7.3.4 AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020, Table 6.6.1.2.3-1 - Detail Categories for Load-Induced Fatigue

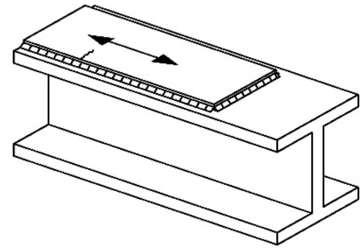
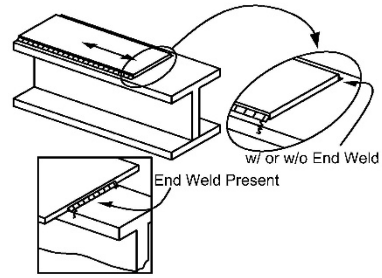
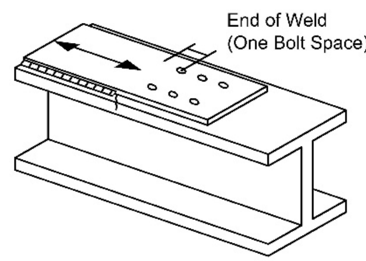
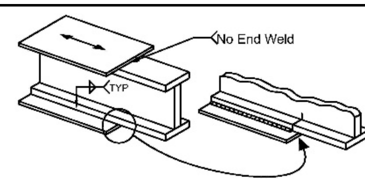
Description	Category	Constant A $(ksi)^3$	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 1—Plain Material away from Any Welding					
1.1 Base metal, except noncoated weathering steel, with rolled or cleaned surfaces. Flame-cut edges with surface roughness 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	

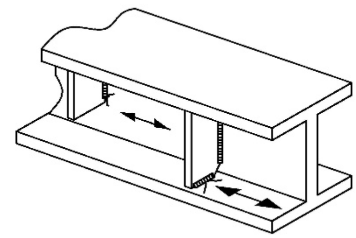
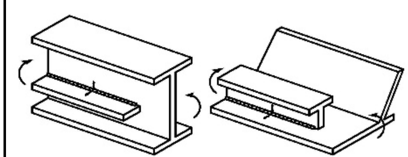
continued on next page

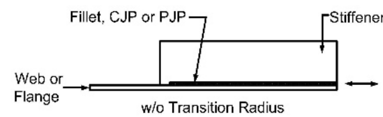
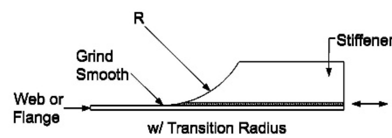
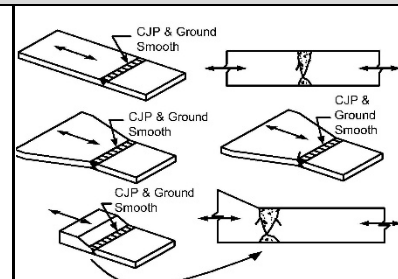
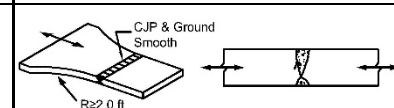
Description	Category	Constant A $(\text{ksi})^3$	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
1.6 Base metal at the net section of manholes or hand holes made to the requirements of AASHTO/AWS D1.5, in which the width of the hole is at least 0.30 times the width of the plate ($A \geq 0.30W$) (Bonachera Martin and Connor, 2017). The geometry of the hole shall be: a. circular; or b. square with corners filleted at a radius at least 0.10 the width of the plate ($R \geq 0.10W$); or c. oval ($B > A$), elongated parallel to the primary stress range; or d. rectangular ($B > A$), elongated parallel to the primary stress range, with corners filleted at a radius at least 0.10 times the width of the plate ($R \geq 0.10W$). All holes shall be centered on the plate under consideration, and all stresses shall be computed on the net section. (Note: Condition 1.5 shall apply for all holes in cross sections in which other smaller open holes or holes with nonpretensioned fasteners are located anywhere within the net section of the larger hole, and minimum edge distance requirements specified in Article 6.13.2.6.6 are satisfied for the smaller holes.)	C	44×10^8	10	In the net section originating at the side of the hole	

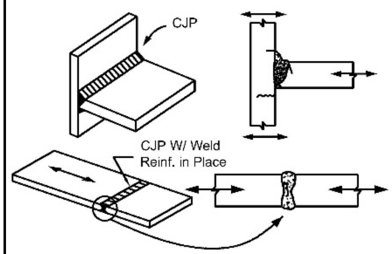
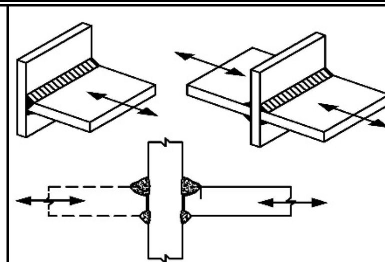
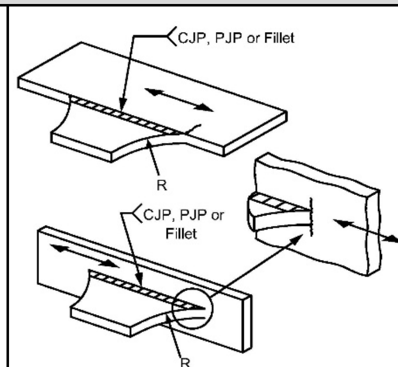
Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 2—Connected Material in Mechanically Fastened Joints					
2.1 Base metal at the gross section of high-strength bolted joints designed as slip-critical connections with pretensioned 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; see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.)	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 pretensioned 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; see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates.)	B	120×10^8	16	In the net section originating at the side of the hole	
2.3 Base metal at the net or gross section 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 ASTM A307 bolts or non-pretensioned high-strength bolts. (Note: see Condition 2.5 for bolted angle or tee section member connections to gusset or connection plates).	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	

Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
2.5 Base metal in angle or tee section members connected to a gusset or connection plate with high-strength bolted slip-critical connections. The fatigue stress range shall be calculated on the effective net area of the member, $A_e = UA_g$, in which $U=(1-\bar{x}/L)$ and where A_g is the gross area of the member. $\bar{x}$ is the distance from the centroid of the member to the surface of the gusset or connection plate and L is the out-to-out distance between the bolts in the connection parallel to the line of force. The effect of the moment due to the eccentricities in the connection shall be ignored in computing the stress range (McDonald and Frank, 2009). The fatigue category shall be taken as that specified for Condition 2.1. For all other types of bolted connections, replace A_g with the net area of the member, A_n, in computing the effective net area according to the preceding equation and use the appropriate fatigue category for that connection type specified for Condition 2.2 or 2.3, as applicable.	See applicable Category above	See applicable Constant above	See applicable Threshold above	Through the gross section near the hole, or in the net section originating at the side of the hole, as applicable	 *continued on next page*
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	

Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
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	
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		*continued on next page*
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	

Description	Category	Constant A $(ksi)^3$	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
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). Base metal adjacent to bearing stiffener-to-flange fillet welds or groove welds.	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	 *continued on next page*

Description	Category	Constant A (ksi)^3	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
Section 4—Welded Stiffener Connections (continued)					
4.3 Base metal at the termination of longitudinal stiffener-to-web or longitudinal stiffener-to-box flange welds:					
With the stiffener attached by 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:				In the primary member near the point of tangency of the radius	
$R \geq 24$ 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		
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		
$F_y \geq 100$ ksi	B′	61×10^8	12		
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	 *continued on next page*

Description	Category	Constant A $(\text{ksi})^3$	Threshold $(\Delta F)_{TH}$ ksi	Potential Crack Initiation Point	Illustrative Examples
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 × 108	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 × 108	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	
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 or at the toe of the weld at the weld termination if not ground smooth	
$R \geq 24$ 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		
For any transition radius with the weld termination not ground smooth.	E	11×10^8	4.5		
(Note: Condition 6.2, 6.3 or 6.4, as applicable, shall also be checked.)					

7.3.6 Superstructure NSTMs

The following is a list of steel bridge superstructure members and connections that are susceptible to fatigue cracking and possible failure:

- Two-girders systems.
- Box beams and girders.
- Trusses.
- Arches.
- Rigid frames.
- Main cables of cable supported bridges.
- Girders or other special features on movable bridges.
- Anchorage systems on floating bridges.
- Pin-and-hanger assemblies.
- Gusset plates.
- Eyebars.

Identification of superstructure members and connections with nonredundant configurations is crucial. It is important to identify all NSTMs present on a bridge within the bridge file, and detail the necessary inspection procedures for each member. With these NSTMs properly identified, inspection teams can focus their efforts on performing the hands-on inspection. Identification of NSTMs and performing hands-on inspection of these members is required by the NBIS (23 CFR 650.313 (f)).

7.3.7 Fatigue Susceptibility

Cracks and fractures have occurred in a large number of steel bridges. A reference manual, *Design and Evaluation of Steel Bridges for Fatigue and Fracture*, was prepared in 2016 under the support of the Federal Highway Administration to explain relevant issues relating to fatigue and fracture. This reference manual accompanies the NHI 130122 Course of the same name. Chapters within the report cover various topics including crack behavior and discontinuities in steel structures, fracture mechanics, factors influencing fatigue, fracture control in design, redundancy, and fatigue design approaches as presented in the *AASHTO LRFD Bridge Design Specifications*. There are many factors that influence the fatigue susceptibility of a bridge with NSTMs, including:

- The degree of redundancy.
- The live load member stress range.
- The propensity of the material to crack or fracture.
- The condition of specific NSTMs.
- The existence of fatigue-prone 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 on NSTM members. For example, a riveted connection is considered a Category D detail per *AASHTO LRFD Bridge Design Specifications* (see Figure 7.3.38). Review the traffic data as higher truck volumes may increase the magnitude and number of stress cycles.

Figure 7.3.38 Riveted Gusset Plate Connection – Category D Fatigue Detail

Initial Deficiencies

Initial deficiencies, in many cases, are cracks resulting from poor quality welds between attachments and base metal (see Figure 7.3.39). 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 welded members can provide a path for the crack to travel between steel members (see Figure 7.3.40).

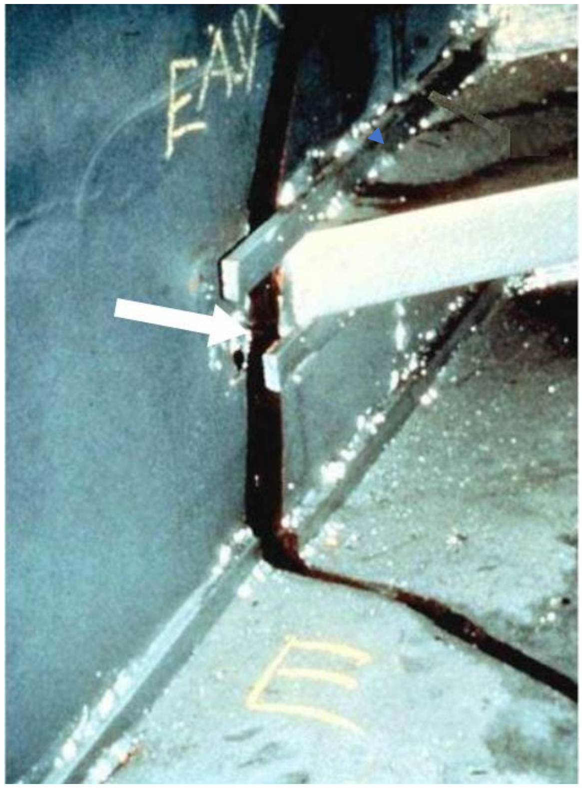


Figure 7.3.39 Poor Quality Welds Inside Cracked Cross Girder

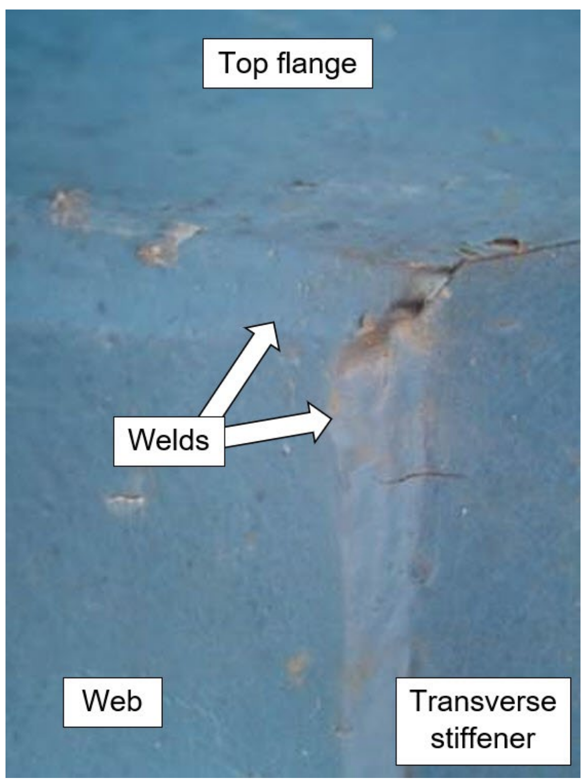


Figure 7.3.40 Intersecting Welded Members

7.3.8 Inspection Methods for NSTMs

Inspection methods for Nonredundant Steel Tension Members are described in Section 7.2.7. Though NSTMs are to be given special observation, the inspector should take care to assure that the remainder of the bridge or non-NSTMs are not ignored and that they are also inspected thoroughly. Bridge plans and shop drawings for bridges designed after 1980 are to have NSTMs clearly identified. If the NSTMs are not clearly identified, use the guidelines previously described in this Chapter, along with the aid of a professional engineer to determine NSTMs.

According to the NBIS, an NSTM inspection is defined as a hands-on inspection of a nonredundant steel tension member. It may include visual and other nondestructive evaluation. *SNBI* Item B.IR.01 records whether an NSTM inspection is required. NSTM inspections are to be performed at intervals not to exceed 24 months unless the member meets the criteria for an extended 48 month interval (23 CFR 650.311(c)(1)(iii)). Certain NSTMs may need a frequency of less than 24 months due to deteriorated condition (23 CFR 650.311(c)). Program managers should establish criteria to determine the level and interval of these inspections based on such factors as structure type, design, materials, age, condition, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle impact damage, loads and safe load capacity, and other known deficiencies.

Section 7.4 Timber

7.4.1 Introduction

Approximately three percent 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 has become more common with the use of engineered wood products (see Figure 7.4.1). 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. For other publications from Forest Products Laboratory in PDF format, access the following link: www.fpl.fs.usda.gov.

Figure 7.4.1 Glued-Laminated Modern Timber Bridge

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 www.awc.org/. 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, Surfaced on 4 Sides (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.

7.4.2 Basic Shapes Used in Bridge Construction

Depending on the necessary 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 a channel (See Figure 7.4.2). 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 due to the flat sides and can be used for decking, superstructure beams, arches, truss elements, and curbs or railings (see Figure 7.4.2).

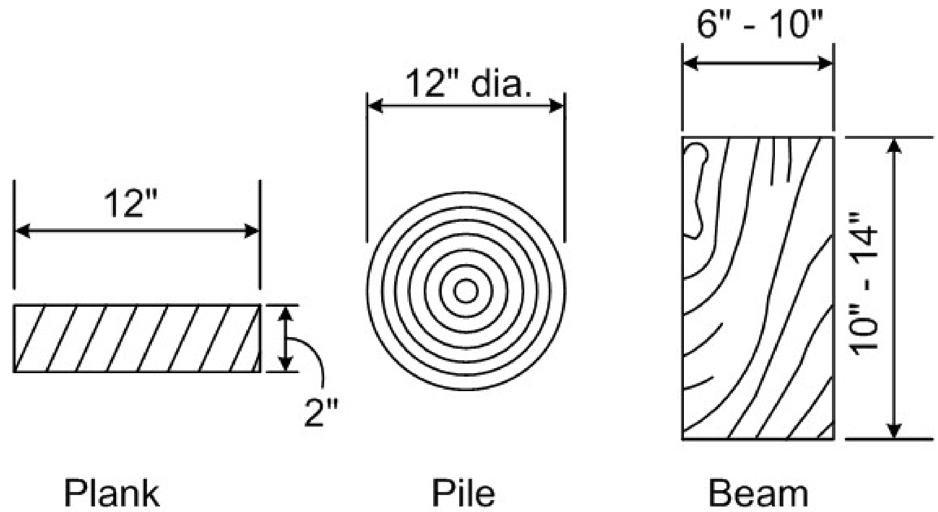


Figure 7.4.2 Timber Shapes

Built-up

Modern timber bridge members are fabricated from basic rectangular shapes to create built-up shapes, which perform at high capacities. Fundamental examples of timber members are rectangular or deck/slab beams. Two other examples are T-shaped and box-shaped beams, trusses and, arches (see Figure 7.4.3). Using glue-laminate technology and stress timber design, these shapes enable modern timber bridges to carry current legal loads.

Refer to Chapter 11 for further information on timber superstructures and Chapter 8 for timber decks.

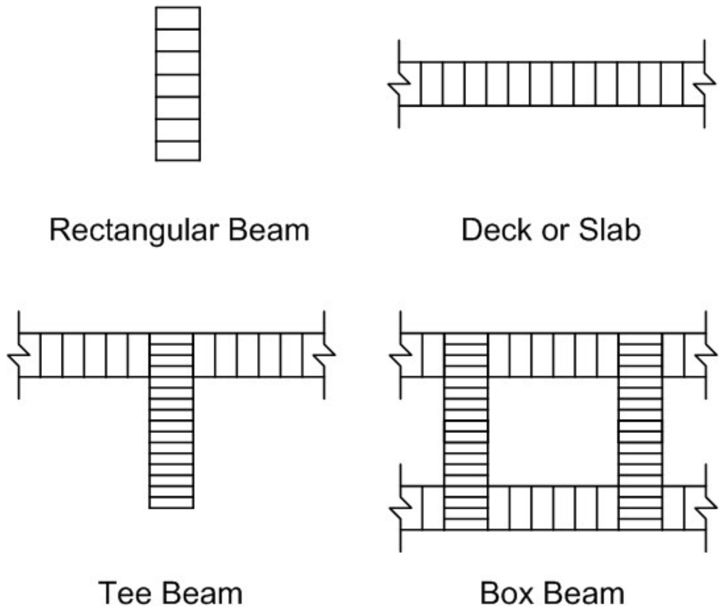


Figure 7.4.3 Built-up Timber Shapes

7.4.3 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.
- Dimensional shrinkage due to moisture loss.

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 describe 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 Figure 7.4.4 and Figure 7.4.5):

- 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 to each other.
- 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, that 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 that transport sugars, and may also protect the tree from contaminants.

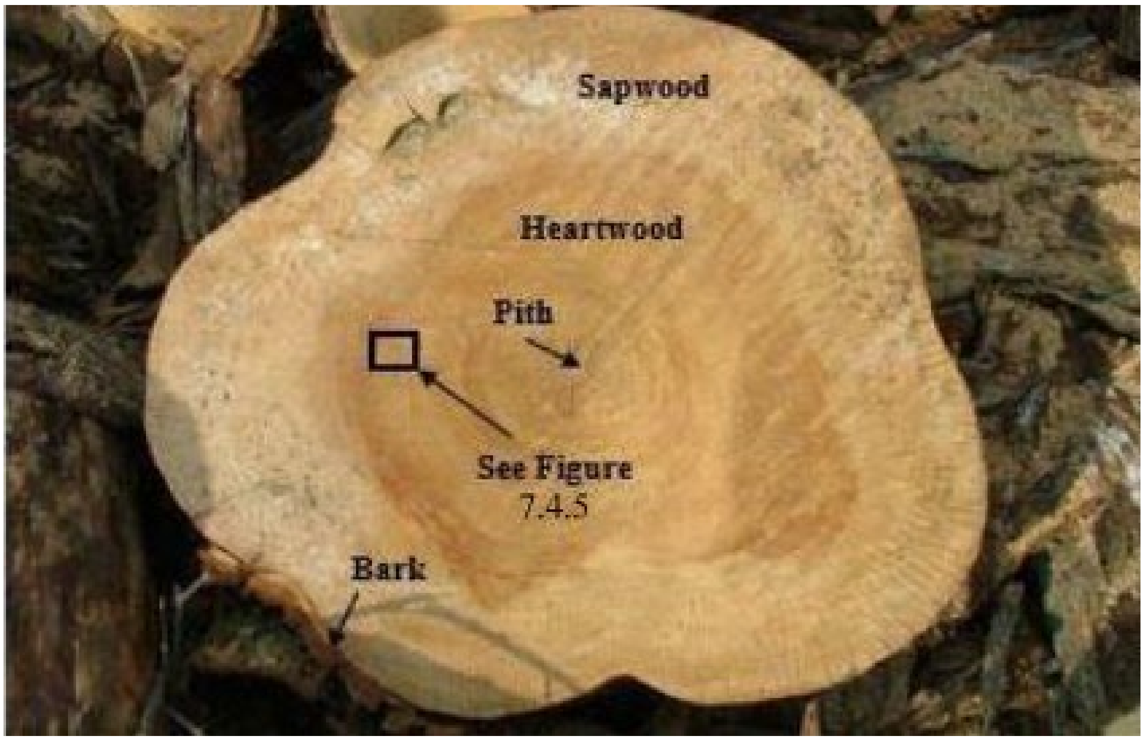


Figure 7.4.4 Anatomy of Timber

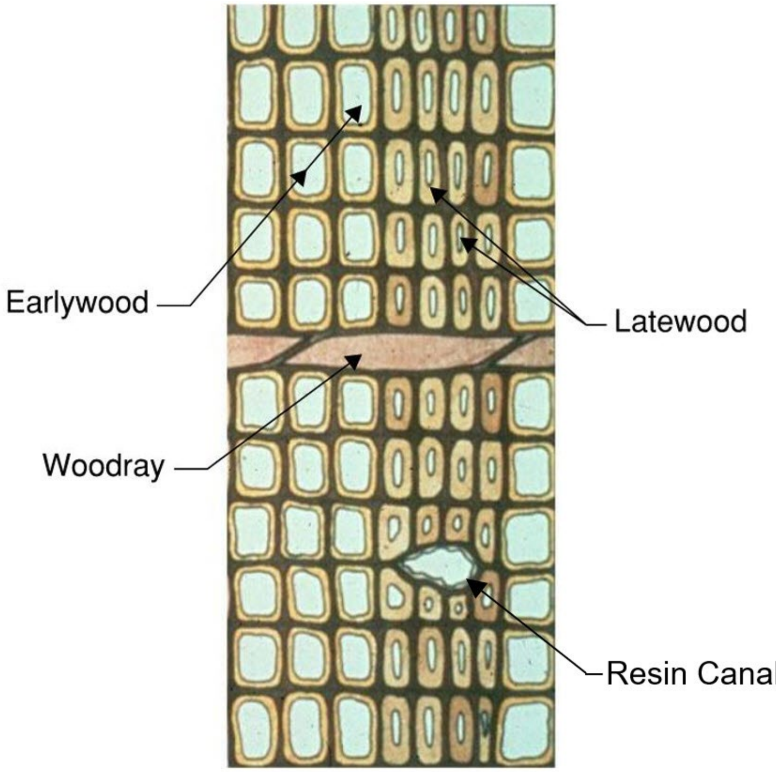


Figure 7.4.5 Close-up of Softwood Timber Anatomy

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 as little moisture content as possible. This is done naturally over time (seasoning) or using kiln drying. Seasoning involves keeping the wood in a dry environment over a long period of time.

Mechanical Properties

There are four basic mechanical properties that describe 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 7.4.6).

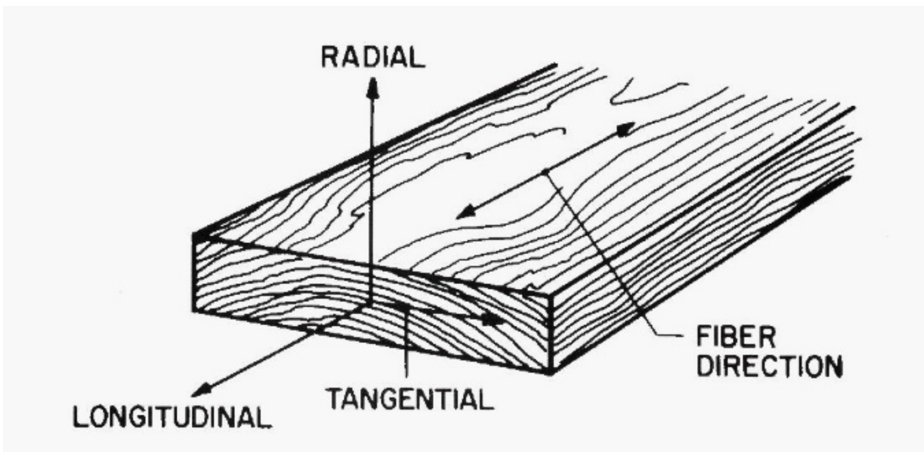


Figure 7.4.6 Three Principal Axes of Wood

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 determined 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 or cyclic 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. Otherwise, impacts can be very detrimental to the cellular structure.

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 enabled 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 may exhibit little permanent increase in deflection with time.

7.4.4 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 are for air-dried wood, which is clear, straight-grained, and free of strength-reducing deficiencies. Reduction factors should be applied to these values based on specific application.

Timber used for outdoor applications should 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, reference the *National Design Specifications for Wood Construction*, American Forest & Paper Association, American Wood Council and *AASHTO LRFD Bridge Design Specifications*, Chapter 8.

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 a visual grading or a mechanical stress grading (MSR).

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 specifications in order to be classified at a certain grade (USDA). If the specifications are not met, the piece of sawn and surfaced lumber is compared to lower grade description specifications 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. 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, in bending as a beam or axial combination as a column or tension member.

Bending combinations are generally 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), and a higher grade lamination is placed on the outside faces where bending stresses are higher.

Axial combinations are generally 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.

7.4.5 Protective Systems

Protective systems are a necessity when using timber for bridge construction. Proper preparation of the timber surface is necessary 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 should 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 ft in diameter and 140 ft 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 to penetrate into the wood for a certain depth. Preservatives are the best means to prevent decay but do not prevent weathering. A paint or water-repellent coating is necessary 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.

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 7.4.7). 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 enable the use of creosote.

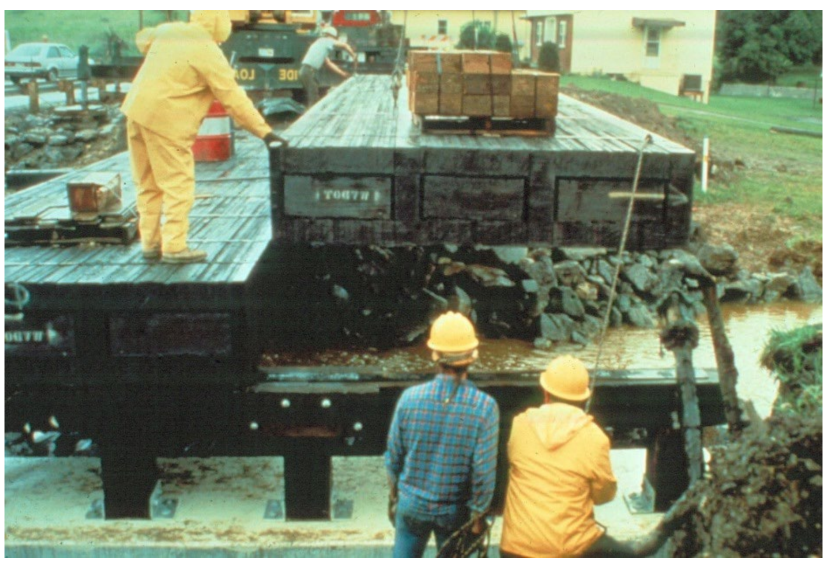


Figure 7.4.7 Coal Tar Creosote Treated Timber Beams

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 saltwater 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 therefore has not been standardized for those applications. Oxine-copper is also used to pressure-treat wood and may be used to control fungi and insects.

Chromated copper arsenate (CCA) was the most common timber preservative from the late 1970s 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.

One alternative to CCA is acid copper chromate (ACC) which is used in industrial and commercial applications. ACC is comprised of approximately two parts chromium trioxide to one part copper oxide. Wood treated with ACC has a light green-brown color with little odor.

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 may be 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 specify 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 been developed and marketed in response to the 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 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 may not indefinitely prevent wood from burning but will most likely 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 that insulates the wood from the intense heat. Application of fire retardants may change some wood properties of glued-laminated timber.

Paint

Wood should be sufficiently dry to permit painting. A few months of seasoning may satisfactorily dry new wood enough to paint. The wood surface should be free of dirt and debris before painting. Old, poorly adherent paint should 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 that 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 may be used to clean small areas.
- Power tool cleaning utilizes powered versions of the hand tools. They are faster than hand tools, but care should 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 should 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.

7.4.6 Anticipated Modes of Timber Deficiencies

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 deficiencies.
- Decay by fungi.
- Damage by insects.
- Damage by borers.
- Chemical attack.
- Other.

Inherent Deficiencies

Defects that form from growth features or from the lumber drying process include (see Figure 7.4.8):

- Checks - separations of the wood fibers, typically 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 “thru check.”
- Shakes - separations of the wood fibers parallel to the grain that 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. Therefore, 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 can form. 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 should be 15 percent 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.

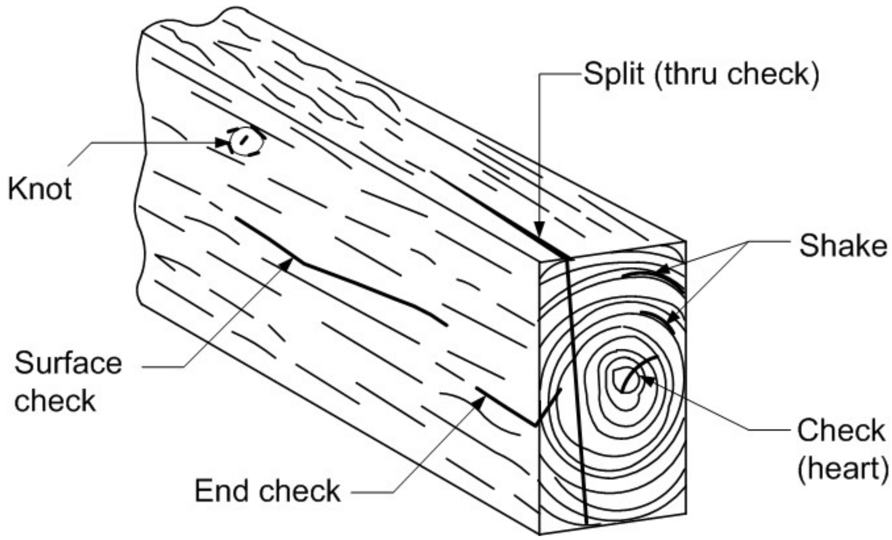


Figure 7.4.8 Inherent Timber Defects

Decay by 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 7.4.9). 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 7.4.9 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 the following four are present:

- Oxygen - Sufficient oxygen should 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 should 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 should be available for the growth of fungi to occur. Below freezing, 32 degrees F, the fungi become dormant but resumes its growth as the temperature rises above freezing to the 75 degrees F to 85 degrees F range, where growth is at its maximum. Above 90 degrees F, growth tapers off rapidly, and temperatures in excess of 120 degrees F 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 should 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 is an adequate supply of moisture. The term "dry-rot" is misleading because dry wood should not rot. Wood should 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 enable moisture to evaporate harmlessly. However, seasoning shakes, checks, and splits, interfaces between timber members, and fastener holes will most likely contribute to 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 fungus 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 7.4.10).
- Stains - specks, spots, streaks, or patches, varying in color, which penetrate the sap wood; sap stain is harmless to wood; it is usually a surface phenomenon and, like molds, implies conditions where harmful fungi can flourish (see Figure 7.4.11).
- 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 7.4.10 Mold and Stains on the Underside of a Timber Bridge

Some fungus 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 7.4.11).
- White rot - feeds upon the cellulose, hemi-cellulose, and the lignin and makes the wood white and stringy (see Figure 7.4.11).

Both brown and white rots can cause structural damage to wood.

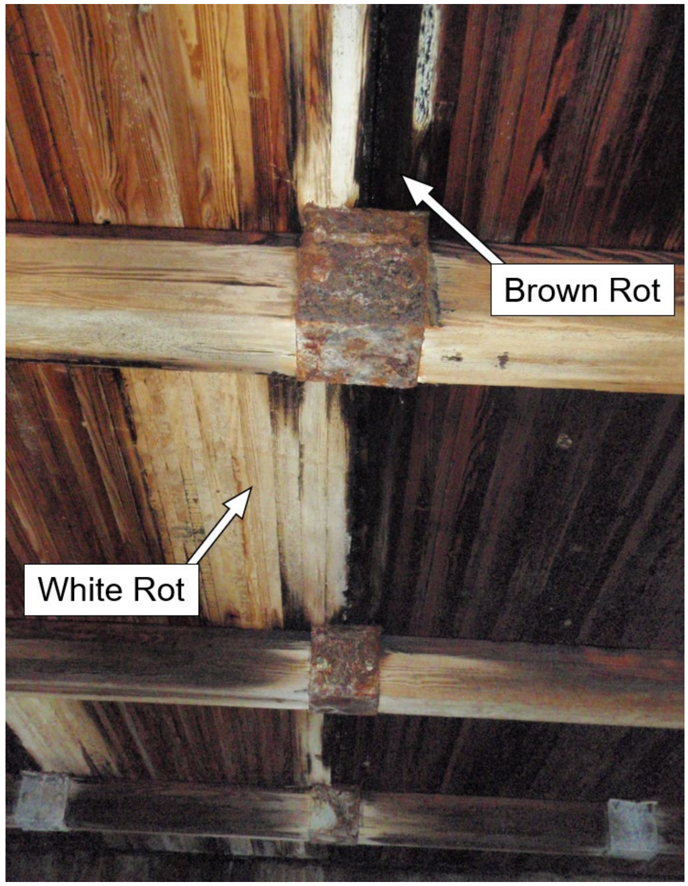


Figure 7.4.11 Brown and White Rot

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. However, 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 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.

Damage by 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 7.4.12). 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 7.4.12 Termites

Powder-post Beetles or Lyctus Beetles

Powder-post beetles (see Figure 7.4.13) 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 7.4.13 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 7.4.14). The ants may be found 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 7.4.14 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 7.4.15). 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 7.4.15 Caddisfly Larva

Damage by Marine Borers

Marine borers are found in seawater and brackish water only and may cause severe damage to timber members in the area between high and low water, although damage may extend to the mud line. 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 7.4.16). 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 ft 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.

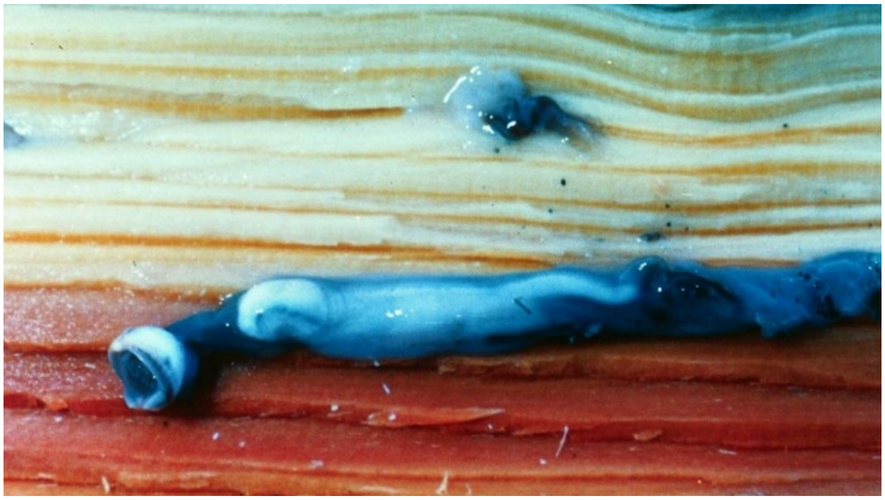


Figure 7.4.16 Shipworm (Mollusk)

Another type of marine borer is the crustacean borer. The most commonly encountered crustacean borer is the limnoria or wood louse (see Figure 7.4.17). 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 likely 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 7.4.17 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 may destroy wood fairly rapidly. Highway bridges are seldom exposed to these substances. Timber structures typically do not come in contact with damaging chemicals unless an accidental spill occurs.

Acids

Wood typically exhibits increased resistance to the effects of certain acids when compared to many other materials and is often used for acid storage tanks. However, strong acids that have oxidizing properties, such as sulfuric and sulfurous acid, may be 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 typically 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 can provide openings for decay to begin and may cause a reduction in strength (see Figure 7.4.18).

Figure 7.4.18 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 7.4.19).

Figure 7.4.19 Loose Hanger Connection Between the Timber Truss and Floorbeam

Surface depressions

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

Fire/Heat Damage

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. Large timbers build a protective coating of char (carbon) after the first 30 minutes of exposure (see Figure 7.4.20). 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 7.4.20 Fire Damaged Timber Members

Impact Damage

Severe damage can occur to truss members, railings, and columns when an errant vehicle strikes them (see Figure 7.4.21).

Figure 7.4.21 Impact/Collision Damage to a Timber Member

Damage from Wear, Abrasion, and Mechanical Wear

Vehicular traffic is the main source of wear on timber decks (see Figure 7.4.22). 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 7.4.22 Wear of a Nail-Laminated Timber Deck

Damage from Overstress

Each timber member has a certain ultimate load capacity. If this load capacity is exceeded, the member will most likely fail. Failure modes include horizontal shear failure, bending moment or flexural failure, and crushing (see Figure 7.4.23, Figure 7.4.24, and Figure 7.4.25).

Figure 7.4.23 Horizontal Shear Failure in Timber Member

Figure 7.4.24 Failed Timber Floorbeam due to Bending Stress Overload

Figure 7.4.25 Decayed Timber Member Subjected to Crushing and Overstress

Damage from Weathering or Warping

Weathering is the effect 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 7.4.26).

Figure 7.4.26 Weathering on Timber Deck

Protective Coating Failure

Paint on timber bridges is typically used on the covered bridge housing. 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.

7.4.7 Inspection Methods for Timber and Protective Systems

There are three basic methods used to inspect timber members. Depending on the type of inspection and deficiency, it may be necessary for the inspector to use only one individual method or all methods. They include:

- Visual inspection methods.
- Physical inspection methods.
- Advanced inspection methods.

Visual Inspection Methods

All timber surfaces should receive a thorough visual assessment to identify obvious surface deficiencies during a routine inspection or in-depth inspection. The visual inspection methods may be supplemented by physical or advanced inspection methods.

Physical Inspection Methods

Some timber deficiencies may necessitate physical inspection methods in addition to visual inspection. For timber members, the main physical inspection methods involve the measurement of deficiencies identified visually. An inspection hammer or pick or small diameter drill or measuring tape may be used to determine the condition of the timber.

Timber

Sounding the wood surface by striking it with a hammer or other object is a common inspection method 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 may 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 may 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 should be verified by advanced inspection methods such as boring or coring.

A pick or penetration test involves lifting a small sliver of wood with a pick, screwdriver, or pocketknife and observing whether or not it splinters or breaks abruptly (see Figure 7.4.27). Sound wood will likely splinter. Decayed wood will likely break abruptly.

Figure 7.4.27 Results of a Pick Test

It may be necessary for an inspector to take samples to determine the condition of the wood. A resistance micro drill is often used for this task. When drilling small diameter holes or boring vertical faces, inspectors should always drill at a slight upward angle so that any drainage flows away from the plugged hole. Repairs should be applied in the form of dowel rods and preservative to any drilled holes once the drilling and boring is complete.

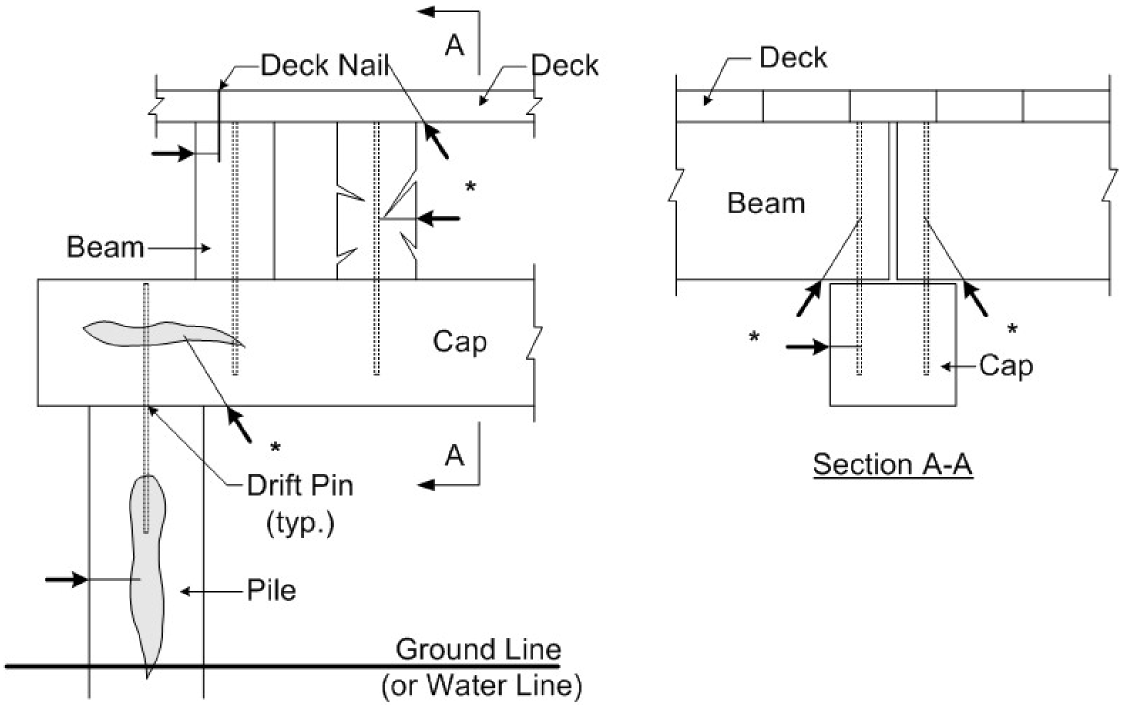


Figure 7.4.28 Timber Boring and Drilling Locations

Protective Coatings

Inspectors should probe the coating with the point of a knife to test paint adhesion to wood and 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. Any amount of paint removed should be noted. Adhesion is rated on a scale of 0 to 5. Refer to ASTM D-3359 for the rating criteria.

Advanced Inspection Methods

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

There are several advanced methods for timber inspection that may be performed in the field. They can be done by a bridge inspector or certified NDE technician and include the following.

- Stress wave velocity.
- Ultrasonic testing.
- Vibration.

Other methods that may be considered destructive testing methods include the following.

- Drilling and probing.
- Moisture meter.

All advanced methods are described in detail in Chapter 17.

Section 7.5 Stone Masonry

7.5.1 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 necessitate inspections. Granite, limestone, and sandstone are the most common types of stone that were used and are still found today in bridges (see Figure 7.5.1). 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 7.5.1 Stone Masonry Arch

7.5.2 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. Other stones may be harder than some grades of steel.
- Workability - measures the amount of effort necessary 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. Water absorption is directly related to the degree of porosity. A stone that is less porous can offer increased resistance to freeze/thaw action when compared to a stone with a higher 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 satisfactory. 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 may have the ability to effectively resist the elements for two hundred years. However, other stone types may deteriorate after about ten years.

Mortar

Mortar, when utilized, is used to seal the joints between the individual stones. 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.

7.5.3 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 that 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 is the least expensive type of stone masonry construction and is considered strong and durable for small spans if well-constructed.

Squared-Stone Masonry

Squared-stone masonry consists of stones that are squared and dressed roughly. They can be laid randomly or in courses.

Ashlar Masonry

Ashlar consists of stones that are precisely squared and finely dressed. Like squared-stone masonry, it could be laid randomly or in courses.

7.5.4 Protective Systems

The different types of protective systems used for concrete (refer to 7.1.7) may also be used for stone masonry. Protective systems for stone masonry are paints or water-repellent sealers. These systems are not very common for stone masonry structures.

7.5.5 Anticipated Modes of Stone Masonry and Mortar Deficiencies

The primary types of deterioration in stone masonry are:

- Missing stones or mortar.
- Weathering - hard surfaces degenerate into 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 7.5.2).
- Fire - masonry is not flammable but can be damaged by high temperatures.

Figure 7.5.2 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.
- Impact – Damage from debris or any material floating in the water that may strike the substructure.
- 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.

7.5.6 Inspection Methods for Stone Masonry and Mortar

There are three basic methods used to inspect stone masonry members and mortar joints. Depending on the type of inspection and deficiency, it may be necessary for the inspector to use only one individual method or all methods.

The three basic methods include:

- Visual inspection methods.
- Physical inspection methods.
- Advanced inspection methods.

Inspection techniques are generally the same as for concrete (refer to 7.1.9 for the examination of concrete).

Visual Inspection Methods

All stone masonry surfaces and mortar joints should receive a thorough visual assessment to identify obvious surface deficiencies during a routine inspection or in-depth inspection. The shape of a masonry arch should be examined for misalignment. Misalignment of the arch can indicate overstress or differential settlement. The visual inspection methods may be supplemented by physical or advanced inspection methods.

Physical Inspection Methods

Some stone masonry and mortar deficiencies may necessitate physical inspection methods in addition to visual inspection. Inspectors should physically examine stone masonry and mortar using a measuring tape, crack comparator card, inspection hammer, rotary percussion tool, or straight edge.

Suspect areas should be sounded using an inspection hammer. Hammer sounding is commonly used to detect areas of delamination and unsound stone. A delaminated area will most likely have a distinctive hollow “clacking” sound when tapped with a hammer, or rotary percussion tool. Striking sound stone masonry should result in a solid “pinging” type sound. The limits of delamination can be outlined with marking keel.

Inspectors should give special observation to documenting the length and width of cracks found during the physical inspection methods. For typical stone masonry members, a crack comparator card or measuring tape can be used to measure the width of cracks. This crack comparator card is described in the concrete inspection section. A ruler or measuring tape should be used to determine the crack length.

A string line or straight edge can be used to determine arch misalignment.

Advanced Inspection Methods

If the extent of the stone masonry deficiency cannot be determined by the visual and/or physical inspection methods described above, advanced inspection methods may be used.

There are several advanced methods for stone masonry inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Flat Jack testing.
- Impact echo testing.
- Infrared thermography.
- Rebound and penetration methods.
- Ultrasonic Pulse Velocity (UPV).
- Ultrasonic Pulse Echo (UPE).
- Ultrasonic Surface Waves (USW).
- GPR has been used to determine the size/extent/depth of the substructure or arch when plans are not available.

All advanced methods are described in detail in Chapter 17.

Section 7.6 Miscellaneous

7.6.1 Introduction

Fiber Reinforced Polymer (FRP) is a modern bridge material that was first used in the United States in the early 1990s, and although not common, FRP composite bridge applications include new bridge construction (primarily bridge deck members) as well as strengthening and rehabilitation of existing members.

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 7.6.1).

Figure 7.6.1 Concrete Beam Repaired Using FRP

Through extensive research and analysis, FRP laminate applications were found to increase the flexural strength of structural concrete members and exhibit very few problems. In some cases, girders repaired using FRP laminate or wrapping techniques had increased capacity and now have greater structural capacity 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 by encapsulating the concrete members. As with any repairs, surface preparation is critical as well as proper mixing of adhesives to the success of the application. Failures of the bond of the FRP has occurred when proper care has not been taken.

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 7.6.2) 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 seismic FRP composite wraps 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. One concern is the lack of ability to visually inspect the condition of the internal concrete and steel. In some cases, windows or gaps in the wrap were provided so inspectors could evaluate the internal material.

Figure 7.6.2 Seismic Retrofit of Concrete Columns Using FRP Composites

Repair and Retrofit of Steel Members Using FRP Composites

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 Figure 7.6.3 and Figure 7.6.4).

Initial findings suggest that CFRP strengthening systems may not reduce live load deflections (or increase member stiffness). However, 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 7.6.3 CFRP Post-tensioned Steel Girder

Figure 7.6.4 Externally Bonded CFRP Plates to Steel Girder Bottom Flange

Repair and Retrofit of Other Structural Members Using FRP Composites

CFRP strands have been used to provide transverse post-tensioning of timber decks, in limited 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. Post-tensioning bars or CFRP plates may be used to increase the live-load capacity of steel girders. Timber beams and decks may be prestressed or post-tensioned to increase overall structure performance. As with steel, the use of FRP composites is being researched to determine long term effects. Concerns regarding CFRP strands are their brittle failure mode.

FRP Decks and Slabs in New Construction

Decks and slabs are the primary use of FRP composites for new bridge construction. At the construction site, the individual panels (typically 8 to 10 ft wide and up to 30 ft in length) are bonded to each other 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 beam top flanges and then grouting the pockets. It is important to note that FRP decks are typically non-composite. However, non-composite action systems can benefit from a significant weight reduction, which lowers the dead load and provides for a greater live load capacity. Refer to Chapter 8 for more information on FRP decks and slabs.

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 7.6.5). Steel and timber members can also benefit from FRP composite reinforcement.

Figure 7.6.5 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.

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 being researched in various configurations as a possible alternative to traditional superstructure materials (see Figure 7.6.6 and Figure 7.6.7). 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 7.6.6 Steel I-Beam (back) and Pultruded FRP I-Beam (front)

Figure 7.6.7 Pultruded FRP Double Web Beam

7.6.2 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, 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 common types of matrix resin currently used for commercially available FRP:

- Orthophthalic polyester - most widely used resin for commercially available FRP composites. This general-purpose low performance resin is inexpensive.
- Isophthalic polyester - offers increased corrosion resistance and structural performance when compared to ortho-polyester and 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 generally 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 percent 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 to form a “roving” (see Figure 7.6.8). This form of fiber reinforcement may be used in the pultrusion process and should offer highly uniaxial mechanical properties if aligned in a single direction.

Figure 7.6.8 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 7.6.9). This form of fiber reinforcement is used in fiber reinforced concrete (FRC) and other applications where lower mechanical properties are sufficient.

Figure 7.6.9 Discontinuous Roving

- Woven roving - glass or carbon fiber roving is woven into a coarse fabric that is commonly used in hand lay-up processes (see Figure 7.6.10). 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 7.6.10 Woven Roving Fabric

- Mats - mats are produced by attaching continuous or discontinuous roving with a binder (see Figure 7.6.11). As with roving, continuous mats provide higher mechanical properties than discontinuous mats.

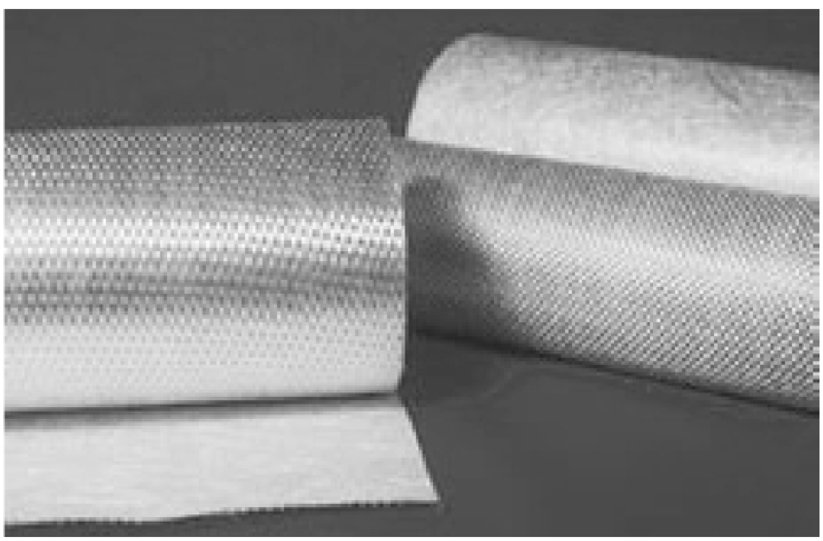


Figure 7.6.11 Discontinuous Roving Mat Fabric

- Non-crimp fabric - reinforcing fibers are stitched or combined by knitting to produce straight layers of sheet fabric in multiple directions (see Figure 7.6.12). 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 widely used for the fabrication of deck panels, despite being more expensive than the other forms of fiber reinforcement.

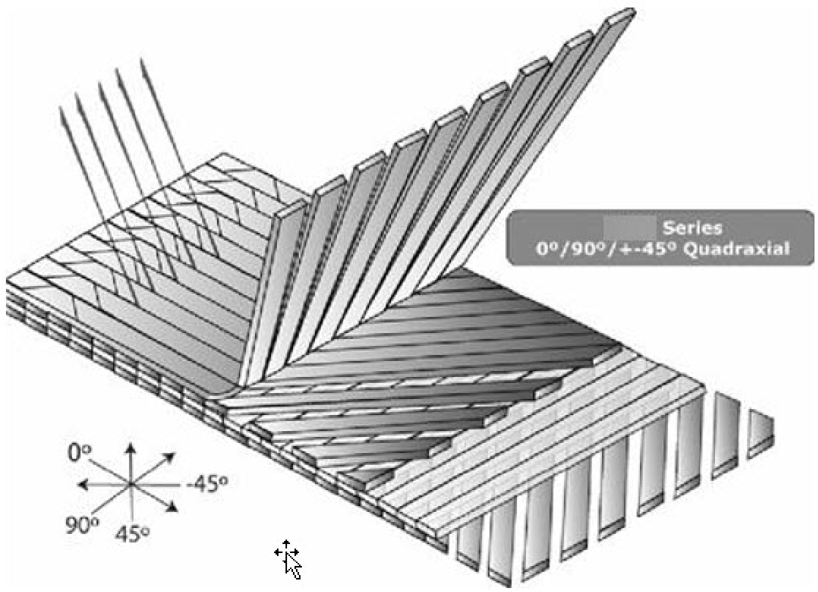


Figure 7.6.12 Non-Crimp Fabric

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 enabled 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.
- 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 typically controls 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 typically 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 percent 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 effects from harsh field conditions and repetitive loading. Detailed analyses and studies are continuing to be conducted regarding this topic.

7.6.3 Fiber Reinforced Polymer Construction Methods

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 provides 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

Each lamination is 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 should be performed in a controlled environment such as a fabrication shop.

Pultrusion

Pultrusion is desirable when FRP composite components necessitate uniformity and consistency. Typically used for structural shapes such as boxes and I-beams, this method involves drawing a resin-fiber mixture through heated dyes that cure the mixture immediately. Necessitating almost no physical labor, pultrusion is very efficient for creating standard shapes and is cost-efficient when producing large quantities. However, the main limitation to pultrusion is the ability to only produce long and narrow objects. FRP composite decks may be produced using pultruded members such as box shapes, but should be bound to each other using an adhesive or bonding agent to achieve the desired width. Similar to VARTM, pultrusion should be performed using large machines in a fabrication shop.

7.6.4 Anticipated Modes of Fiber Reinforced Polymer Deficiencies

In order to properly inspect FRP components, the inspector should 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 gel-coated surfaces due to trapped moisture in the laminate. 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.

Voids and Delamination

Voids are debonded areas within the laminates. These regions may be visible only after they have grown and resulted into a surface crack (see Figure 7.6.13). Delamination typically starts at the initial site of a void, which can be detected with signal penetration equipment or by a tap test.

Figure 7.6.13 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 7.6.14). This deficiency is generally not a structural problem unless present at connectivity points or bonding regions.

Figure 7.6.14 Wrinkling of FRP Fabric

Fiber Exposure

Fiber exposure is a structural deficiency that is typically a result from improper handling and erection methods (see Figure 7.6.15). Given the vulnerability of the fibers when exposed to moisture and contaminates, this deficiency could lead to significant damage if left untreated.

Figure 7.6.15 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.

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 7.6.16). Cracks typically develop throughout the entire thickness of the laminate.

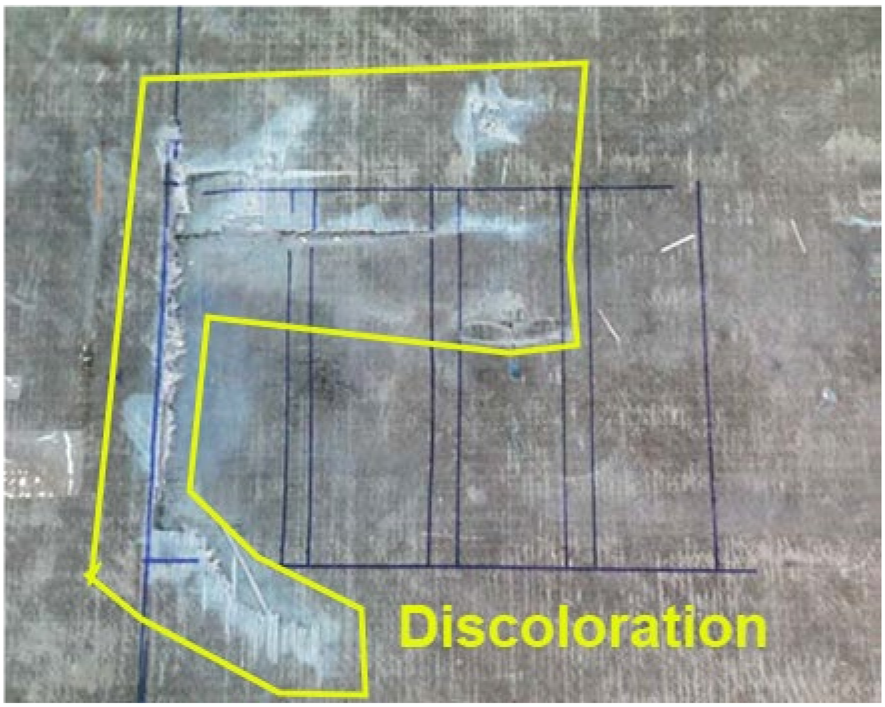


Figure 7.6.16 Cracks and Discoloration Around Punched Area

7.6.5 Inspection Methods for Fiber Reinforced Polymer

There are three basic methods used to inspect and evaluate FRP members. Depending on the type of inspection, it may be necessary for the inspector to use one or more of the methods. These methods include:

- Visual inspection methods.
- Physical inspection methods.
- Advanced inspection methods.

Visual Inspection Methods

All FRP surfaces should receive a thorough visual assessment to identify obvious surface deficiencies during a routine inspection or in-depth inspection. The visual inspection methods may be supplemented by physical or advanced inspection methods. Very often, visual means do not provide definitive results during examination of FRP.

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 Chapter 8) to assist in detecting cracks and other deficiencies including vertical movement.

Physical Inspection Methods

Some FRP deficiencies may necessitate physical inspection methods in addition to visual inspection. Inspectors should physically examine FRP members using a measuring tape, crack comparator card, inspection hammer, chain drag, feeler gage, taper gage, or rotary percussion tool.

Suspect or delaminated areas should be sounded using an inspection hammer, chain drag, or rotary percussion tool.

Special observation should be given to document the length and width of cracks found during the physical inspection methods. For typical FRP members, a crack comparator card can be used to measure the width of cracks. A ruler or measuring tape should be used to determine the crack length. A feeler or taper gage can be used to measure the spacing between layers that are supposed to be bonded to each other.

If the inspection is performed within a noisy environment, electronic units may be used to indicate suspect areas (see Figure 7.6.17). However, these units are typically not preferred over conventional methods due to the additional time necessary 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.

Traditional and electronic tap testing does not necessitate NDE certification and may be performed by a typical bridge inspector or engineer. The equipment can be operated with very little training, but understanding the results takes experience. FRP tends to sound hollow, even if the material is still in good condition.

Figure 7.6.17 Electronic Tap Testing Device

Advanced Inspection Methods

If the extent of the FRP deficiency cannot be determined by visual and/or physical examination methods described above, advanced inspection methods may be used. Examples of nondestructive evaluation methods are listed below.

- Acoustic emission testing.
- Ultrasonic testing.
- Laser-based ultrasound testing.
- Tap testing.
- Thermal testing.

All advanced methods are described in detail in Chapter 17.

This page intentionally left blank.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 8 TABLE OF CONTENTS

CHAPTER 8 LIST OF FIGURES

CHAPTER 8 LIST OF TABLES

Chapter 8 Inspection and Evaluation of Decks and Adjacent Areas

Section 8.1 Concrete Decks

8.1.1 Introduction

The most common bridge deck material is concrete. The physical properties of concrete permit it to be placed in various shapes and sizes, providing the bridge designer and the bridge builder with a variety of construction methods. This section presents various aspects of concrete bridge decks and related bridge inspection issues. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

The role of a concrete bridge deck is to transmit vehicular traffic to the superstructure. The deck also provides a smooth riding surface for motorists, and diverts runoff water. 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 ultra-high performance concrete, as well as different reinforcement options, to help concrete bridge decks more effectively 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.

Deck material and type are recorded in FHWA *SNBI* item B.SP.09. In cases where a combination of deck type and materials are present, the predominant deck material and type is coded based on deck area. Codes for this item are provided for aluminum, various types of reinforced concrete (cast-in-place, precast, pre-tensioned, etc.), various types of fiber reinforced polymer (FRP) (aramid fiber, carbon fiber, glass fiber, etc.), steel (open grid, filled or partially filled grid, plate, orthotropic, corrugated, other), and timber (glue laminated, nail laminated, solid sawn, stress laminated, other). "None" is coded for this item when the bridge or culvert is under fill since no deck is present. "None" is also coded for slab bridges, arches without spandrels, closed spandrel arches, pipes, and three-sided or four-sided rigid frames.

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 typically used to contain conventional reinforcing bars and wet concrete so that after curing, all parts of the deck will be in the correct position and shape. "Bar chairs" are generally 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 typically 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 8.1.1).

Figure 8.1.1 CIP Concrete Deck with Stay-in-Place Forms

Precast Conventionally Reinforced

Precast deck panels are conventionally reinforced concrete panels that are formed, cast, and cured somewhere other than at the bridge site. Proper deck elevations are generally accomplished using leveling bolts and a grouting system.

The precast deck panels can be connected 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 (see Figure 8.1.2). The deck panels are positioned over the shear connectors, and the block-out holes are filled with concrete or grout.

Figure 8.1.2 Block-Out Holes in Precast Panels

Precast Prestressed

Like conventionally reinforced panels, precast prestressed deck panels are also reinforced concrete decks that are formed, cast, and cured away from the bridge site. However, they are reinforced with prestressing steel in addition to some mild reinforcement. The prestressing can be tensioned prior to placing the deck concrete (pre-tensioned) or after the deck concrete is cured (post-tensioned). The pre-tensioning process utilizes strand tendons, whereas bars are typically used for post-tensioned applications. The prestressing is held in position until the deck has sufficiently cured, and the tendons are released, or the bars are tensioned and locked off. This process creates compressive forces in the deck, that reduces the amount of tension cracking in the cured concrete. Installation of the deck precast prestressed panels is similar to that of precast conventionally reinforced deck panels.

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 8.1.3). A cast-in-place (CIP) layer, which may be reinforced depending on thickness, is then placed over the panels that 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 thorough shear connectors and the superstructure (welded studs for steel superstructures and extended stirrups for concrete superstructures).

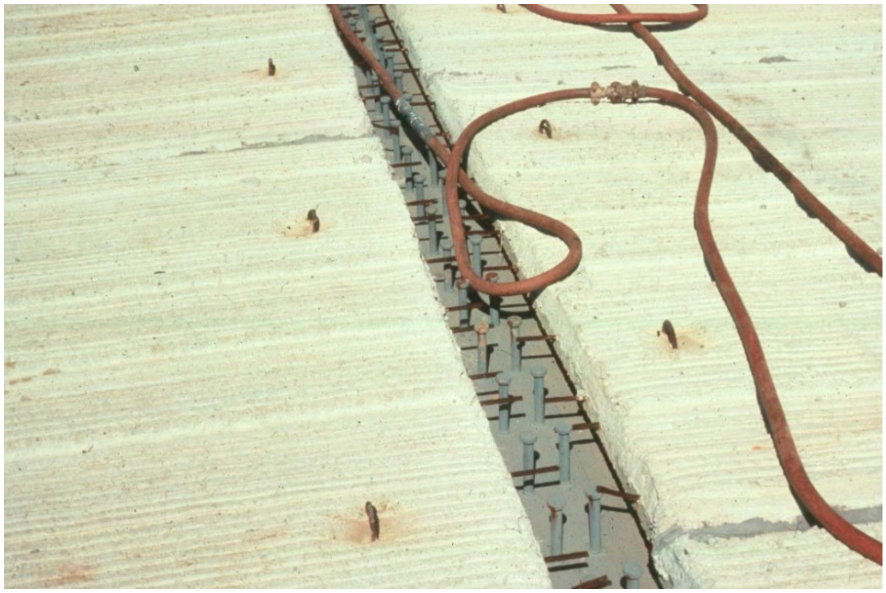


Figure 8.1.3 Precast Deck Panels with Lifting Lugs Evident and Top Beam Flange Exposed (Prior to CIP Topping)

Composite Action

A concrete deck is generally necessary when composite action is desired with the superstructure (refer to Chapter 6). Composite action can be defined as dissimilar materials combined so they behave as one structural unit. A composite bridge deck structure is one in that the deck acts structurally connected 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 Figure 8.1.4 and Figure 8.1.5). A precast deck can also develop composite action through grout pockets that engage shear connectors. 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.

Figure 8.1.4 Shear Connectors Welded to the Top Flange of Steel Girders

FHWA *SNBI* item B.SP.08 records the type of structural interaction between the bridge deck and superstructure. This item can be coded for composite action – both for shored and unshored construction. Unshored construction indicates that the deck acts composite with the superstructure, and that the superstructure can carry its own self-weight, plus that of the deck concrete prior to curing. Shored construction indicates that the deck is composite but that the superstructure requires shoring to carry its own self weight without the deck, the weight of the deck prior to curing, or both. B.SP.08 may also be coded to indicate that the deck and superstructure are integral or monolithic, meaning that the deck was cast or fabricated with the same material and at the same time as the superstructure. As a result, both the deck and superstructure can be expected to act as a unit. When the type of interaction is unknown, B.SP.08 is coded to be consistent with assumptions used in previous load rating calculations.

Figure 8.1.5 Prestressed Concrete Beams with Shear Connectors Protruding Composite Deck

Non-Composite Action

A non-composite concrete deck is not mechanically attached with shear studs to the superstructure and does not contribute to the capacity of the superstructure.

If the deck and superstructure do not interact structurally with each other, then FHWA *SNBI* item B.SP.08 is coded as non-composite.

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 round or square steel rods with a smooth finish and could 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 "rebar". 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 approximately

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 or glass fiber bars. Refer to Chapter 7 for detailed explanations on various reinforcement types.

Primary reinforcement carries the tensile stress in a concrete deck and is in both the top and bottom of the deck. Decks are currently designed with a thickness that shear reinforcement is not normally needed. Older, thinner decks utilized bent tensile reinforcement that also act as shear reinforcement in areas close to superstructure support. These bent bars are sometimes referred to as ‘crank’ bars.

Secondary reinforcement is used to control cracking that occurs from expansion/contraction of concrete with temperature change as well as shrinkage that occurs when the deck concrete is curing. This steel is sometimes referred to as temperature and shrinkage steel and is normally 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 to be able to identify the direction of the primary reinforcement to properly evaluate any cracks in the deck. Primary reinforcement is normally placed perpendicular to the deck's support points, spanning the shortest distance between points. For example, the support points on a multi-beam bridge or 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 8.1.6). In a girder-floorbeam system with floorbeams that are spaced closer than the girders, the primary reinforcement runs floorbeam to floorbeam. 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.

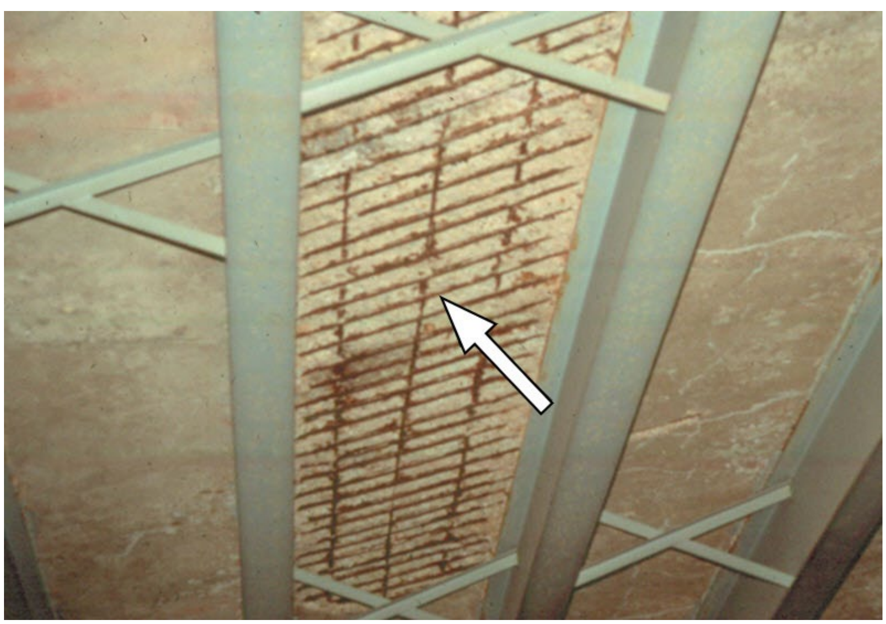


Figure 8.1.6 Spalling Showing Primary Deck Reinforcing Steel Perpendicular to Traffic

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.

8.1.2 Wearing Surfaces

Wearing surfaces are placed on top of the deck, and serve to protect the deck and provide a smooth riding surface. The wearing surface materials most commonly used on concrete decks are generally special concrete mixes or bituminous (asphalt) concrete. Wearing surfaces are incorporated in many new deck designs and are also a common repair procedure for decks.

The predominant wearing surface material on a bridge deck is reported in the FHWA *SNBI* item B.SP.10. Bridge decks may have several types of wearing surfaces, but the predominant type based on surface area is coded in this item. Patch repairs are not to be considered. B.SP.10 wearing surface codes exist for various types of concrete including, but not limited to, monolithic, latex modified, low slump, and fiber reinforced, as well as bituminous (asphalt). Codes are also provided for polymers, steel, timber, earth, other, and none (coded if no sacrificial wearing surface is present).

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 this time. These include:

- Low slump dense concrete (LSDC).
- Polymer/latex modified concrete (LMC).
- Internally sealed concrete.
- Lightweight concrete (LWC).
- Fiber reinforced concrete (FRC).
- Polyester Polymer Concrete (PPC).
- Self-Consolidating Concrete (SCC).
- Ultra-High-Performance Concrete (UHPC).
- Fiber Reinforced Concrete (FRC).

See Chapter 7 for detailed explanations of these types of concrete.

Bituminous

The most common overlay material for existing concrete decks is bituminous (commonly referred to as “asphalt”). Bituminous overlays generally range from 1 ½ inch up to 3 inches thick, depending on the design, owner’s preferences, and the load capacity of the superstructure. When bituminous is placed on reinforced 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.

Polymers

Polymer overlays on concrete decks help prevent the infusion of chlorides. The longevity of these overlays is dependent on the volume of traffic, the use of studded tires, thickness of overlay and condition of the deck when the overlay was applied. Typical polymer wearing surfaces may be constructed of epoxy or polyester-based material.

Refer to Chapter 7 for detailed descriptions of materials used in wearing surfaces.

8.1.3 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 members of the bridge structure. Therefore, protecting the bridge deck from contamination and deterioration is of high importance.

Deck protective systems and deck reinforcement protective systems are coded in FHWA *SNBI* items B.SP.11 and B.SP.12, respectively. In cases where a combination of protective systems is implemented, the predominant protective system on protected area is recorded. In cases where a combination of protective systems is used in the same area, the outermost protective system is recorded. Codes for B.SP.11 are provided for various types of protective admixtures, including but not limited to, internally sealed, low permeability, and polymer impregnated. Protective coatings such as paint, silane/siloxane, methacrylate, and other are also possible codes. Codes for protective membranes are also provided including built up, sheet, liquid applied, unknown, and other. B.SP.11 is coded as “none” if no known internal or external protective system is in place. B.SP.11 is not reported if Deck Material and Type (B.SP.09) is coded “none”.

Codes for B.SP.12 are provided for various types of deck reinforcing protective systems. Codes are provided for coatings (epoxy, galvanized, metalized, and other), reinforcing (stainless clad, stainless solid, high chromium, FRP, and other), sacrificial (cathodic passive, cathodic active, and other), as well as “other” and “none”. “None” is coded for this item if the deck reinforcement is unprotected, such as with black steel.

Sealants

Reinforced concrete deck sealants are typically 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.

In the past, boiled linseed oil sealant has been used to 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 but has to be reapplied periodically. Usage today is normally limited to low traffic volume bridge decks.

Methacrylate also referred to as High Molecular Weight Methacrylate (HMWM) or Methyl Methacrylate, is an impregnating sealer for concrete and masonry surfaces. HMWM penetrates into surfaces through its low viscosity allowing it to fill and rebond cracks.

Silane Sealer

Silane sealer is a water-based water repellent and impregnating sealer that is sometimes used on the deck surface prior to placing waterproofing membranes or bituminous wearing surfaces.

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 typically used to retard reflective cracking and provide waterproofing.

Reinforcement Bars

Reinforcement bars in concrete decks are treated to slow the corrosion process. They include:

- Epoxy coated reinforcement bars.
- Galvanized reinforcement bars.
- Solid stainless steel or stainless steel clad reinforcement bars.
- Fiberglass reinforced polymer (FRP) bars.
- MMFX reinforcement bars.
- Cathodic protection of reinforcement bars.
- Anodic protection of reinforcement bars.

Refer to Chapter 7 for detailed descriptions of Reinforcement Bars.

8.1.4 Overview of Common Deficiencies

Common concrete deck deficiencies are listed below. Refer to Chapter 7 for a detailed description of these deficiencies:

- Cracking.
- Scaling.
- Delamination.
- Spalling.
- Exposed rebar.
- 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.

8.1.5 Inspection Methods

Visual and Physical

It is very important to use whatever inspection methods are necessary to fully ascertain conditions, including the causes and importance of each deficiency, so that serious and critical findings can be properly conveyed in the inspection report and other communication channels. The inspection of concrete for surface deficiencies and downward camber is primarily a visual activity. Inspectors should determine and note deficiencies in primary vs. secondary reinforcement or members, and structural vs. non-structural problems.

Physical inspection methods may often be employed to supplement visual inspection methods. Physical methods for concrete are primarily used to detect delaminated areas by sounding and to measure defects such as cracks, spalls, scaling, and other deficiencies (see Figure 8.1.7).

Refer to Chapter 7 for detailed procedures of visual and physical inspection methods.

Figure 8.1.7 Sounding for Delaminated Areas of Concrete

Advanced Inspection

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.

There are several advanced methods for concrete inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Electrical Resistivity (ER).
- Galvanostatic Pulse Measurement (GPM).
- Ground Penetrating Radar (GPR).
- Half-Cell Potential (HCP).
- Impact Echo (IE).
- Infrared Thermography (IT).
- Linear Polarization (LPR).
- Magnetic Flux Leakage (MFL).
- Magnetometer (MM).
- Rebound and penetration methods.
- Ultrasonic Pulse Velocity (UPV).
- Ultrasonic Pulse Echo (UPE).
- Ultrasonic Surface Waves (USW).

Other methods that may be considered destructive testing methods include the following.

- Concrete cores.
- Borescopes (considered destructive if drilling a hole is necessary).
- Moisture content.
- Petrographic examination.
- Alkali-Silica Reaction (ASR) evaluation.
- Reinforcing steel sample.
- Rapid chloride permeability testing.

All advanced methods are described in detail in Chapter 17.

8.1.6 Inspection Areas and Deficiencies

All areas of both the top and bottom surfaces of concrete decks are to be inspected for deficiencies.

Prior to inspecting specific areas below, the concrete decks should be inspected for general global alignment. The curb line, top of railing, and the bottom of the deck should be viewed and examined for vertical or horizontal misalignment. Both sides of expansion joints should be examined to check if they line up properly. Vertical misalignment, horizontal misalignment or sagging may be an indication of specific details listed below. For instance, sagging may be evidence of prestress strand relaxation. Misalignment of expansion joints may be evidence of differential substructure settlement or deficiencies in the bearing devices. Also, inspectors should look at the condition of secondary members. Distorted secondary members may be evidence of primary member differential movement or overstress. Misalignment may be indicators of potential deficiencies.

Visual, physical, or advanced inspection methods may be necessary to evaluate the following inspection areas and deficiencies:

- Areas exposed to traffic - Surface texture and wheel ruts should be examined due to wear. The inspection team should check cross-slopes for uniformity. It should be verified that repairs are acting as intended.
- Areas exposed to drainage, curb lines, etc. – inspectors should investigate for ponding water, scaling, delamination, and spalls.
- Bearing and shear areas where the concrete deck is supported – the inspection team should check for cracks, spalls and crushing near supports.
- Shear key joints between precast deck panels - leaking joints, cracks, and other signs of independent panel action should be inspected.
- Anchorage zones of precast prestressed deck tendons - deteriorating grout pockets or loose lock-off devices (if visible) should be checked.
- Top of the deck over the supports - flexure cracks which typically would be perpendicular to the primary tension reinforcement should be examined.
- Bottom of the deck between the supports – the inspection team should check for flexure cracks which would be perpendicular to the primary tension reinforcement (see Figure 8.1.8).

- Bituminous overlays - if present, they should be inspected. Cracks, delamination, and spalls should be noted. Often water penetrates overlays and then penetrates 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 - deterioration and corrosion of the forms should be investigated (see Figure 8.1.9), often indicating advanced contamination of the concrete deck; these forms can retain moisture and chlorides which have penetrated full depth cracks in the deck. Signs of deterioration in the SIP forms may be the only visual sign of deterioration for the deck as the forms hide the bottom side of the deck and a wearing surface may be on the top side.
- Cathodic protection – inspectors should check that all visible electrical connections and wiring from the rectifier to the concrete structure are intact and operating. Care should be taken while working around electrical systems. If not trained properly in inspection of these devices, a specialized inspection team should perform the electrical circuitry assessment. The rectifiers should be checked 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, maintenance personnel should be notified. Some agencies that use cathodic protection have specialized inspection/maintenance crews for these types of bridge decks.
- Areas previously repaired - deterioration of any patches that were previous noted should be investigated. Inspectors should determine if the repairs are in place and functioning properly, including checking for recent post-repair leakage.
- Areas of closure pours - signs of any delamination or spalling around the area of a closure pour should be inspected.
- Adjacent to joints - signs of delamination or spalling in general area around the deck joint should be investigated.
- Fire damage - any damage caused by fire should be examined.

Figure 8.1.8 Underside View of Concrete Deck Longitudinal Crack and Efflorescence

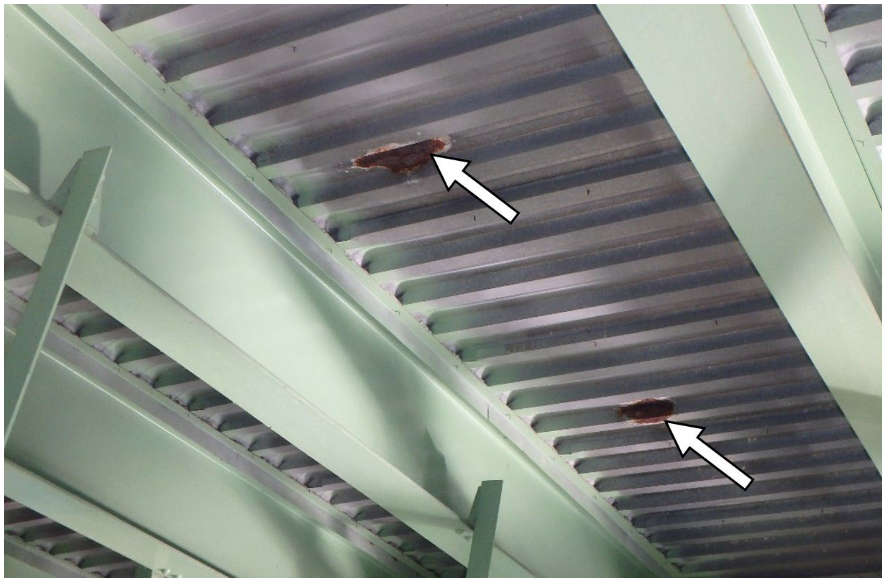


Figure 8.1.9 Stay-in-Place Form with Isolated Rusting

FHWA *SNBI* item B.C.01 for deck condition rating represents the condition of the deck as determined from the inspection of all deck surfaces (top, underside, and edges). Refer to FHWA *SNBI* Subsection 5.3 for more information on coding this item.

Section 8.2 Steel Decks

8.2.1 Introduction

Steel decks are present on a small number of older bridges and moveable bridges. Their popularity grew until concrete decks were introduced. Today, less than 3 percent of decks in the NBI are steel. Steel bridge decks have various advantages and limitations described below.

Steel bridge decks are mainly used when weight is a major factor, since 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. These decks are slippery when wet, they are noisy under traffic, and welds commonly experience fatigue cracks. Steel grid decks can be filled or partially filled with concrete to prevent water from passing through, but this negates the benefit of weight reduction.

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 perpendicular 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 8.2.1). FHWA *SNBI* item B.SP.09 would be coded as "Steel-orthotropic" for a bridge with an orthotropic deck.

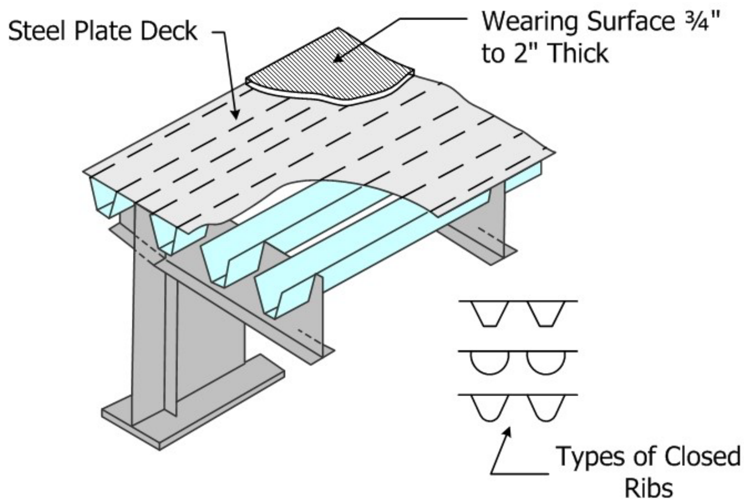


Figure 8.2.1 Orthotropic Bridge Deck

Buckle Plate Decks

Buckle plate decks are typically found on older bridges. They consist of steel plates attached to the floor system that support a layer of reinforced concrete (see Figure 8.2.2). 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. Reinforced concrete on these types of decks is typically cast in place on top of the partial depth structural steel panels. Since these steel panels are not just considered stay-in-place forms, the commentary for FHWA *SNBI* item B.SP.09 directs inspection teams to code this item as "Reinforced concrete – cast-in-place" or "Prestressed concrete – cast-in-place" as appropriate.

Figure 8.2.2 Underside View of Buckle Plate Deck

Corrugated Steel Decks

Corrugated steel flooring can be used because of its light weight and high strength. This deck consists of corrugated steel planks usually covered by a layer of gravel or bituminous (asphalt) (see Figure 8.2.3).

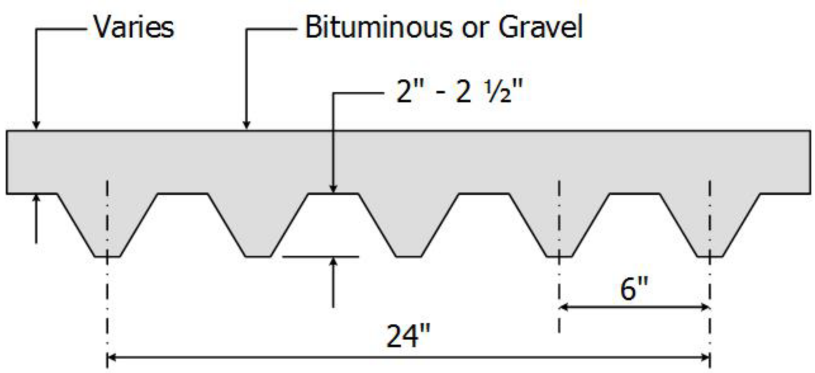


Figure 8.2.3 Corrugated Steel Deck

The gravel or bituminous 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 members. Corrugations are larger 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 the steel superstructure. In the case of timber superstructures, the corrugated flooring is attached by lag bolts. The corrugations are filled with bituminous or gravel and there are no reinforcement bars utilized in this deck type. A significant limitation of this deck type is a short life span due in part to corrugations holding water from leakage through the gravel or asphalt, causing corrosion and section loss of the steel corrugated pan. FHWA *SNBI* item B.SP.09 for this type of bridge deck is coded as “Steel – corrugated” with B.SP.10 for wearing surfaced coded based on material placed on top of the corrugated steel deck.

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.

Grid decks are often found on moveable bridges or rehabilitated bridges. Their lower weight reduces the dead load, 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.
- Other proprietary decks.

FHWA *SNBI* item B.SP.09 for these types of bridge decks may be coded as "Steel – open grid", "Steel – filled or partially filled grid", or "Steel – other" as appropriate.

Welded Grid Decks

Welded grid decks have their pieces connected by welds. These pieces consist of bearing bars, cross bars, and supplementary bars (see Figure 8.2.4).

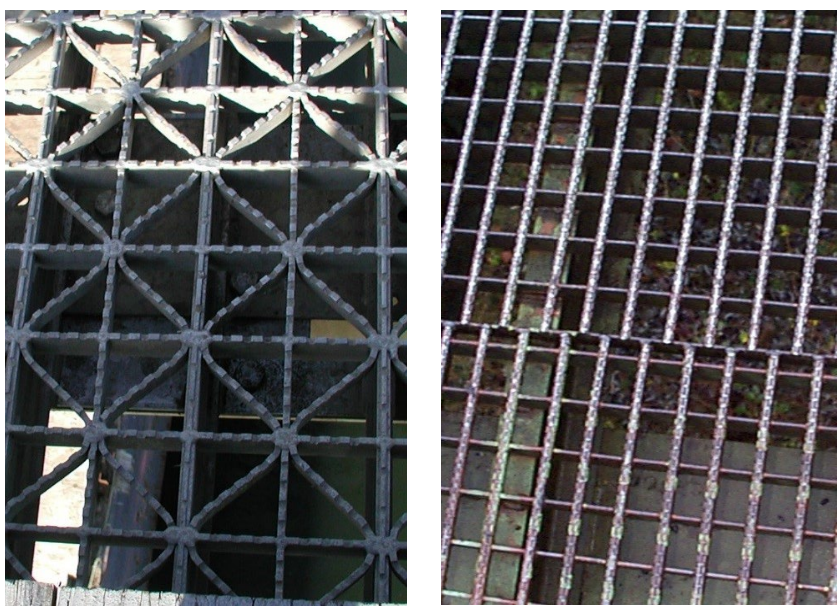


Figure 8.2.4 Various Patterns of Welded Steel Grid Decks

The bearing bars supports 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 8.2.7).

The distribution bars are grating bars that are laid perpendicular to the bearing bars. They may be shop-or field-welded to the grating system. Cross bars, also referred to as secondary bars or distribution bars (see Figure 8.2.7).

The supplementary bars are grating bars parallel to the bearing bars. They can also be 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 welds, rivets, or bolts. They are similar to the bearing bars in welded grates (see Figure 8.2.5).

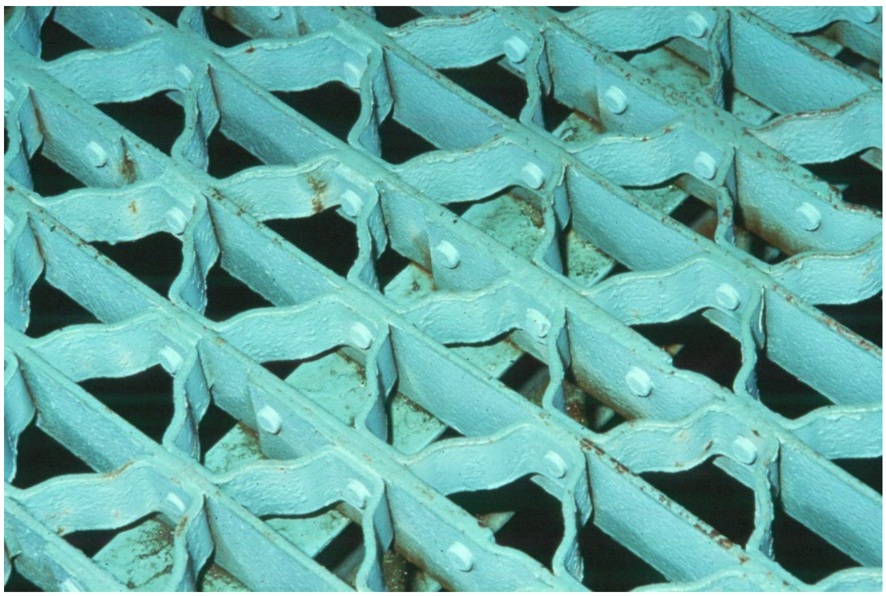


Figure 8.2.5 Riveted Grid Deck

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 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 8.2.6).

Figure 8.2.6 Steel Grid Deck with Slotted Holes to Eliminate Welding and Riveting

Concrete-Filled Decks

Concrete-filled grid decks offer protection for the floor system and superstructure against water, dirt, debris, and deicing chemicals that usually pass directly through open grid decks. They can be partially or fully filled. Concrete is not typically considered when determining the total capacity of the concrete-filled deck.

Partially filled decks are grid decks where 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.

Fully filled decks are grid decks that have been completely filled with concrete (see Figure 8.2.7). These decks provide the maximum protection of the underlying bridge members (see Figure 8.2.8). Form pans are generally welded at the bottom of the grid to hold the concrete.

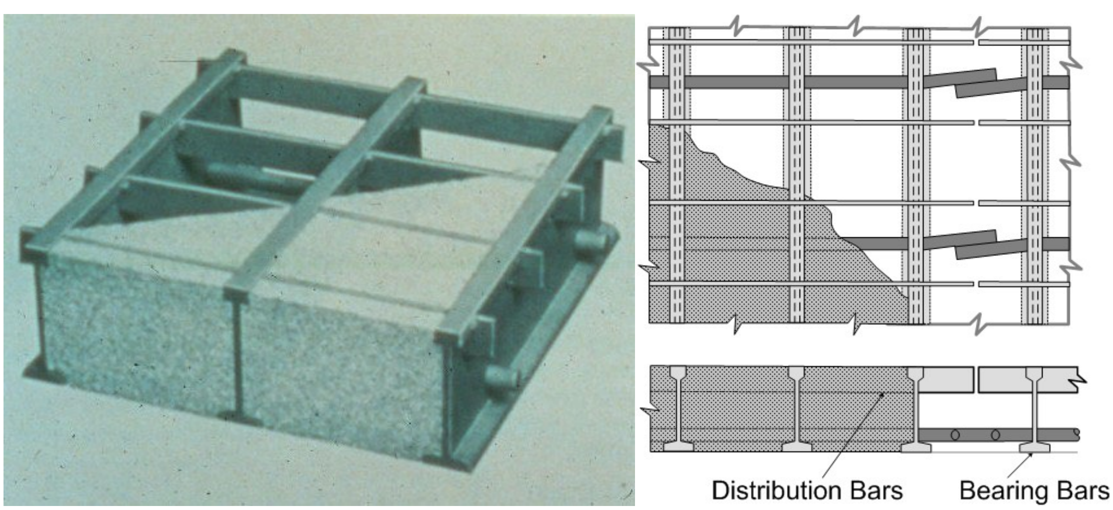


Figure 8.2.7 Concrete-Filled Grid Deck

Figure 8.2.8 Filled and Un-filled Steel Grid Deck

Other Proprietary Decks

Other proprietary decks (such as Exodermic®) consist of reinforced concrete that is composite with the steel grid (see Figure 8.2.9). Composite action is achieved by studs that extend into the reinforced concrete deck and are welded to the grid deck and superstructure. Galvanized sheeting is used as a bottom form to keep the concrete from falling through the grid holes. These types of decks generally weigh significantly less than precast reinforced concrete decks. In this case, FHWA *SNBI* item B.SP.09 may qualify as a "Steel – other" code if it does not qualify for the other steel deck codes.

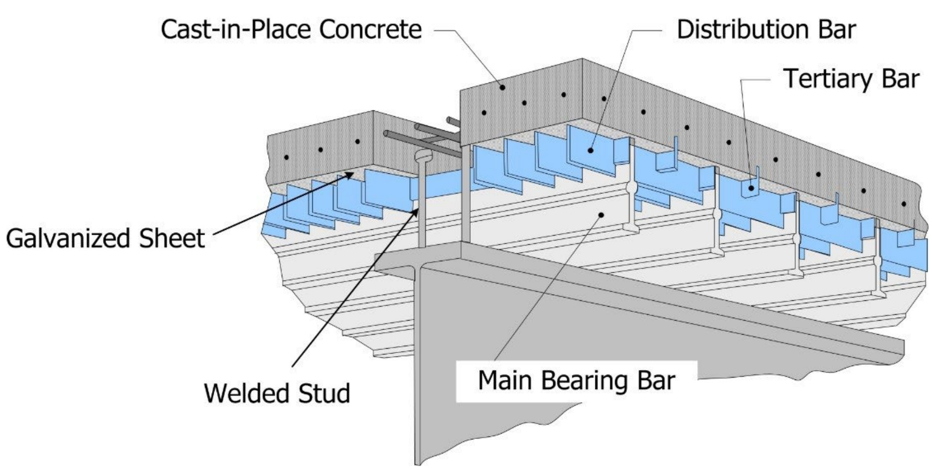


Figure 8.2.9 Schematic of an Exodermic® Deck

8.2.2 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.

Serrated Steel

Open grid decks may have serrated edges on the grating (see Figure 8.2.4). These serrations allow the standing water to pass more easily through the deck and reduces the chance of hydroplaning and improves skid resistance. Studs may be welded to steel decks for additional skid resistance.

Concrete

Concrete above the top of the grid 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. 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. This bituminous helps to increase vehicle traction, in addition to providing protection of the steel plate. Bituminous overlays generally range from 1½ inches up to 3 inches thick. Corrugated steel plank decks may also have bituminous wearing surfaces.

Epoxy bituminous polymer concrete is also 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 types of decks, drains may be found at midspan to minimize water accumulation in the corrugations.

8.2.3 Protective Systems

Protective systems may be added to protect the exposed surface of the steel deck. Their intent is slow the growth rate of corrosion and section loss. Common protective systems include:

- Paints.
- Galvanizing.
- Metalizing.
- Weathering steel patina.
- Epoxy coating.

Refer to Chapter 7 for detailed descriptions of protective systems for steel.

8.2.4 Overview of Common Deficiencies

Some of the common steel deck deficiencies are listed below. Refer to Chapter 7 to review steel deficiencies in detail.

- Bent, damaged, or missing members.
- Corrosion.
- Fatigue cracks.
- Other stress-related cracks.

8.2.5 Inspection Methods

Visual and Physical

It is very important to use whatever inspection methods are necessary to fully ascertain conditions, including the causes and importance of each deficiency so that serious and critical findings can be properly conveyed in the inspection report and other communication channels. The inspection of steel for surface deficiencies and downward camber is primarily a visual activity. Defects in primary vs. secondary members, and structural vs. non-structural problems should be determined and noted.

Physical inspection methods may often be employed to supplement visual inspection methods. Physical methods for steel decks with concrete are primarily used to detect delaminated areas by sounding and to measure defects such as cracks, spalls, scaling, and other defects. Connections at welds or rivets can be tapped with a hammer to determine integrity and tightness of the connections.

Refer to Chapter 7 for detailed procedures of visual and physical inspection methods.

Advanced Inspection

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

There are several advanced methods for steel inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Acoustic Emission (AE).
- Crack Propagation Gage (CPG).
- Dye Penetrant Testing (PT).
- Eddy Current Testing (ET).
- Magnetic Particle Testing (MT).
- Magnetic Flux Leakage (MFL).
- Ultrasonic Testing (UT).
- Phased Array Ultrasonic Testing (PAUT).

Other methods that may be considered destructive testing methods include the following.

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

All advanced methods are described in detail in Chapter 17.

8.2.6 Inspection Areas and Deficiencies

Prior to inspecting specific areas below, the steel decks should be inspected for general global alignment. The curb line, top of railing, and the bottom of the deck should be viewed and examined for vertical or horizontal misalignment. Both sides of expansion joints should be examined to check if they line up properly. Vertical misalignment, horizontal misalignment or sagging may be an indication of specific details listed below. For instance, upward camber may be evidence corrosion between connecting grid deck members. Misalignment of expansion joints may be evidence of differential substructure settlement or deficiencies in the bearing devices. Also, inspectors should look at the condition of secondary members. Distorted secondary members may be evidence of primary member differential movement or overstress. Misalignment may be indicators of potential deficiencies.

Visual, physical, or advanced inspection methods may be needed to evaluate the following inspection areas and deficiencies:

- Sagging, deflection under live load, misalignment – the inspection team should look for general or global indications of movement, overloading, or loss of capacity to carry normal loads, such as permanent sagging near midspan, deflection under live load, misalignment at bearing areas, etc.
- Bearing and shear areas - primary bearing bars should be checked for buckling, cracked welds, broken fasteners, or missing bars that connect the steel deck to the supporting superstructure.

- Areas exposed to traffic - the top surface should be examined for wheel ruts or wear. The deteriorated deck should be verified that it will not damage tires.
- Tension areas - on steel grid decks, the positive and negative moment regions of the primary bearing bars should be checked. Inspectors should look for deficiencies such as distorted or missing bars, fatigue cracks or other stress related problems (see Figure 8.2.10).
- Areas exposed to drainage - areas where drainage can lead to corrosion should be checked. Inspectors should look at areas along the curb lines that collect dirt and debris. Drainage is problematic for open grid decks.
- Corrugated deck – inspectors should check between the support points for section loss due to corrosion. Excessive vertical movement of the deck under live load may indicate weld failure.
- Orthotropic decks - orthotropic steel plate decks should be checked for debonding of the overlay, rust-through or cracks in the steel plate, and for the development of fatigue cracks in the web portions or connecting welds. The connection between the orthotropic plate deck and supporting members should be examined.
- Connections - broken connections should be inspected; listen for rattles as traffic passes over the deck.
- Filled grid decks - the inspection team should look for grid expansion at joints and bridge ends, often caused by corrosion. The condition of the concrete in the grids should be checked.
- Areas previously repaired – inspectors should document the location and condition of any repair plates and their connections to the deck.

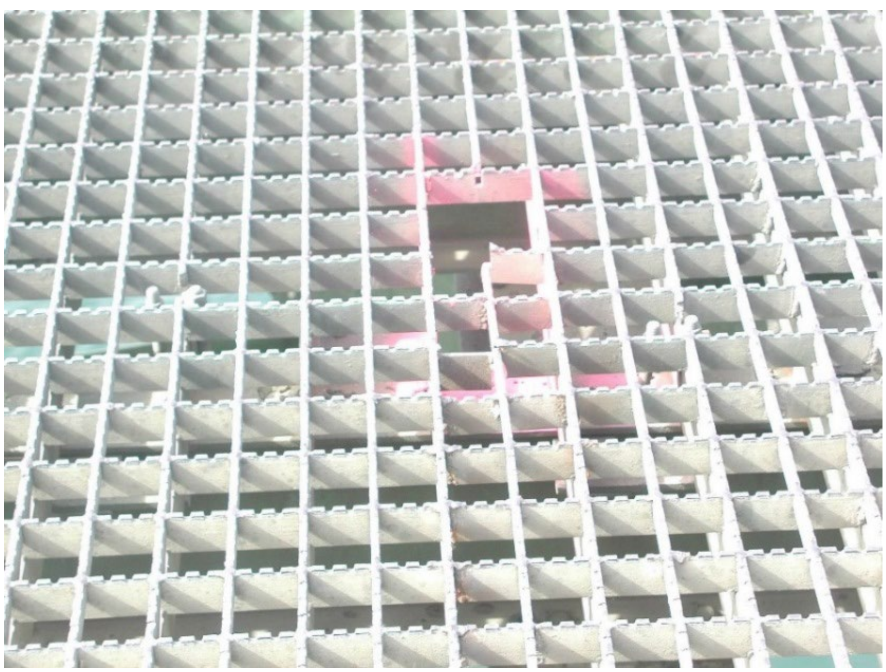


Figure 8.2.10 Broken Members of an Open Steel Grid Deck

FHWA *SNBI* item B.C.01 for deck condition rating represents the condition of the deck as determined from the inspection of all deck surfaces (top, underside, and edges). Refer to FHWA *SNBI* Subsection 5.3 for more information on coding this item.

Section 8.3 Timber Decks

8.3.1 Introduction

Timber can be desirable for use as a bridge decking material because it is resistant to deicing agents, that 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. Timber decks are most common for local, low traffic applications, and for settings where a timber aesthetic is desired.

Timber decks are normally referred to as decking, planks, or timber flooring, and the term is generally limited to the roadway portion that 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.

FHWA *SNBI* item B.SP.09 for deck material and type may be coded as “Timber – glue laminated”, “Timber – nail laminated”, “Timber – solid sawn”, “Timber – stress laminated”, or “Timber – other”.

Plank Decks

Plank decks consist of timber boards laid transversely across the bridge (see Figure 8.3.1). The planks are individually attached to the superstructure using spikes or bolt clamps, depending on the superstructure material. Occasionally, plank decks have 2-inch depth timbers nailed longitudinally on top of the planks for the width of the deck to distribute load and retain a bituminous wearing surface.

Figure 8.3.1 Plank Deck

Nailed-laminated Decks

Nail-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 8.3.2). When supported on timber beams, each lamination is toenailed to the beam. On steel beams, clamp bolts are typically used as needed to provide adequate attachment but not to provide composite action. In both cases, laminates span across the beams and are perpendicular to the supporting superstructure.

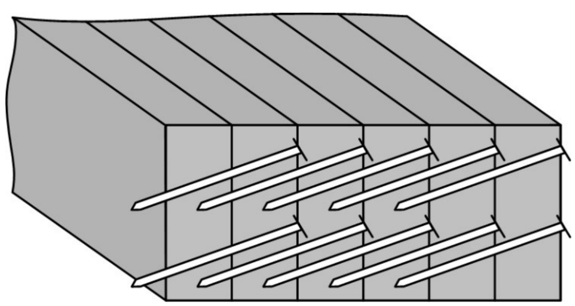


Figure 8.3.2 Nail-laminated Deck

Glued-laminated Deck Panels

Glued-laminated (Glulam) deck panels are an engineered wood product in which pieces of sawn lumber are attached with waterproof glue adhesives. Glulam deck panels come in sizes usually 4 ft 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 8.3.3). In other cases, the glulam panels are laid longitudinally without the use of supporting beams and supported by abutments or piers. This installation would not technically be a deck but rather a slab.

Figure 8.3.3 Glued-laminated Deck Panels Attached to Superstructure

Bolting the deck to the superstructure or floor system provides a greater resistance to uplift than nailing. 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. 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. The nailing method is generally not preferred due to the possibility of the nails being pried loose by the vehicle traffic vibrations and deflections. In each case the bolts or nails tend to loosen over time.

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.

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 without the use of separate supporting beams. The timber planks vary in length and size. The laminations are pulled tight by prestressing (post-tensioning) high strength steel bars, spaced approximately 24 inches on center. The bars are passed through predrilled holes in the timber planks and are tensioned with a hydraulic jacking system. Steel channel bulkheads or anchorage plates are then used to anchor the prestressing bars (see Figure 8.3.4). This prestressing operation creates friction connections at the interface between planks, thereby enabling the laminated planks to span longer distances.

Stressed-laminated decks are present 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 8.3.4 Stressed-laminated Deck

Structural Composite Lumber Decks

Structural composite lumber (SCL) decks include laminated veneer lumber (LVL) and parallel strand lumber (PSL). LVL is fabricated by combining thin sheets of rotary-peeled wood veneer with a waterproof adhesive. PSL is fabricated by taking narrow strips of veneer, gluing, and compressing them with the wood grain parallel. SCL bridge decks 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 connected by stress lamination by 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 8.3.5).

Figure 8.3.5 Structural Composite Lumber Deck Using Box Sections

Refer to Chapter 7 for various T-beam and box shape configurations used for structural composite lumber decks.

8.3.2 Wearing Surfaces

The wearing surface of a timber deck is constructed of timber, bituminous, or concrete.

Concrete

While concrete may be used as a wearing surface on timber decks, it is not frequently used for this purpose. However, concrete overlays composite with timber decks have been constructed but are rare. They generally involve a timber deck with steel shear studs doweled into the timber deck with a concrete overlay completing the composite action.

Bituminous

Bituminous wearing surfaces generally utilize a coarse aggregate mix. The aggregate is mixed with a binder substance that joins the aggregate and bonds the surfacing to the deck. Bituminous, commonly referred to as asphalt, can be hot mix asphalt or a chip and seal method. Bituminous wearing surfaces are sometimes used with a membrane to waterproof the deck. However, it is not commonly used on the plank deck type because deflection of the planks causes the asphalt to break apart.

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 8.3.6). This provides some load distribution, and can provide a visual path for travel.

Figure 8.3.6 Timber Wearing Surface on a Timber Plank Deck

8.3.3 Protective Systems

Protective systems are necessary to resist decay in timber bridge decks. Typical protective systems include:

- Water repellents.
- Preservatives.
- Fire retardants.
- Paints.

In order for the protective system to serve its purpose, the surface of the timber has to be properly prepared. Refer to Chapter 7 for detailed information on protective systems.

8.3.4 Overview of Common Deficiencies

The following is a list of common deficiencies that may be encountered when inspecting timber bridge decks. Refer to Chapter 7 for a detailed description of these common deficiencies:

- Inherent defects: checks, splits, shakes, knots.
- Fungi.
- Insects.
- Marine borers.
- Chemical attack.
- Delaminated areas.
- Loose connections.
- Surface depressions.
- Fire.

- Impact or collision.
- Wear, abrasion, and mechanical wear.
- Overstress.
- Weathering or warping.
- Protective coating failure.

8.3.5 Inspection Methods

Bridge inspectors have the difficult task of accurately assessing the condition of an existing timber deck, since most decay occurs on the inside of a timber member. Timber inspection necessitates some knowledge and understanding of wood pathology, wood technology, and timber engineering.

Visual and Physical

It is very important to use whatever inspection methods are necessary to fully ascertain conditions, including the causes and importance of each deficiency, so that serious and critical findings can be properly conveyed in the inspection report and other communication channels. The inspection of timber for surface deficiencies and downward camber is primarily a visual activity. Inspectors should determine and note defects in primary vs. secondary members, and structural vs. non-structural problems.

Physical inspection methods may often be employed to supplement visual inspection methods. Physical methods for timber are primarily used detect decayed delaminated areas by sounding and to measure deficiencies such as inherent defects, insects, and other deficiencies. A pick test can be used to determine the soundness of the timber surface (see Figure 8.3.7) defects. Connections at tie bolts and transverse stressed rods can be tapped with a hammer to determine integrity and tightness of the connections.

Figure 8.3.7 Inspector Probing Timber with a Pick

Refer to Chapter 7 for detailed procedures of visual and physical inspection methods.

Advanced Inspection

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

There are several advanced methods for concrete inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Stress wave velocity.
- Ultrasonic testing.
- Vibration.

Other methods that may be considered destructive testing methods include the following.

- Drilling and probing.
- Moisture meter.

All advanced methods are described in detail in Chapter 17.

8.3.6 Inspection Areas and Deficiencies

Prior to inspecting specific areas below, the timber deck should be inspected for general global alignment. The curb line, top of railing, and the bottom of the deck should be viewed and examined for vertical or horizontal misalignment. Both sides of expansion joints should be examined to check if they line up properly. Vertical misalignment, horizontal misalignment or sagging may be an indication of specific details listed below. Misalignment of expansion joints may be evidence of differential substructure settlement or deficiencies in the bearing devices. Also, inspectors should look at the condition of secondary members. Distorted secondary members may be evidence of primary member differential movement or overstress. Misalignment may be indicators of potential deficiencies.

Visual, physical, or advanced inspection methods may be necessary to evaluate the following inspection areas and deficiencies:

- Areas exposed to traffic – the inspection team should examine for wear, weathering, and impact damage (see Figure 8.3.8).
- Bearing and shear areas where the timber deck contacts the supporting superstructure – the inspection team should look for crushing, decay, and fastener deficiencies (see Figure 8.3.9).
- Tension areas between the support points – the inspection team should investigate for flexure damage, such as splitting, sagging, and cracks.
- Areas exposed to drainage – the inspection team should check for decay, particularly in areas exposed to drainage (see Figure 8.3.10).
- Outside edges of deck – decay should be inspected.

- Connections – inspectors should note any looseness that may have developed from inadequate nailing or bolting, or where the spikes have worked loose. Observation under passing traffic can 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. Loose, corroded, or damaged nails should be inspected.
- Post Tensioning anchorages and bar tendons – tightness, corrosion, crushing, and decay should be checked(see Figure 8.3.11).
- Fire damage – the inspection team should check for any section loss or member damage caused by fire.

Figure 8.3.8 Wear and Weathering on a Timber Deck

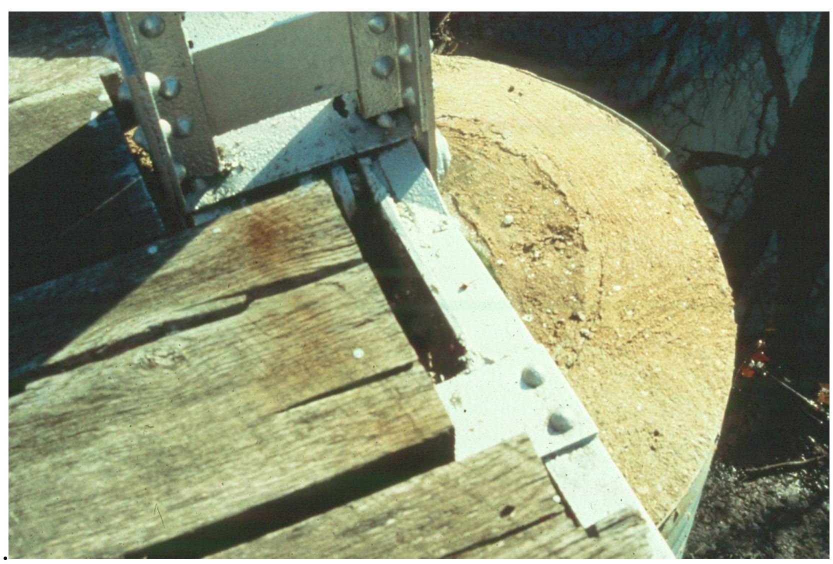


Figure 8.3.9 Bearing Area on a Timber Deck

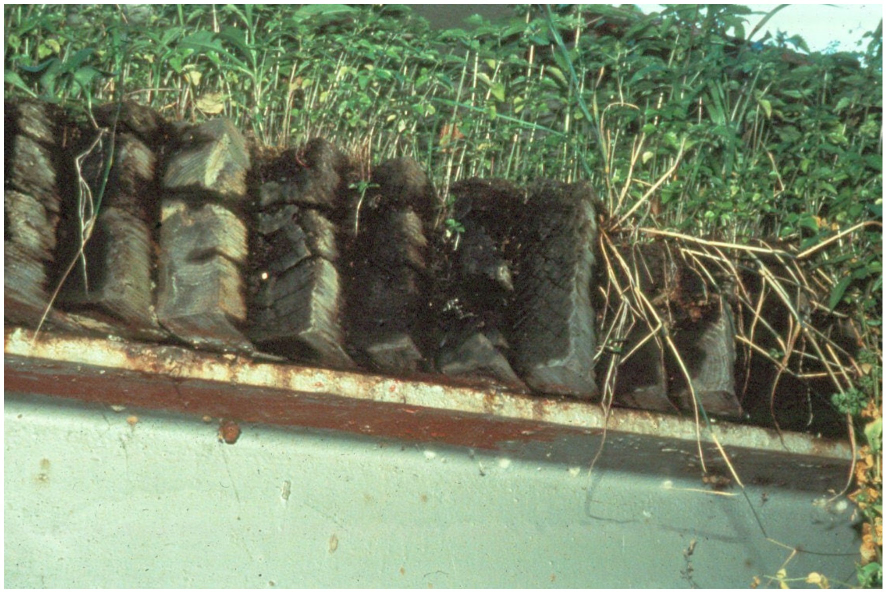


Figure 8.3.10 Timber Deck Exposed to Drainage, Resulting in Decay and Plant Growth

Figure 8.3.11 Failed Post-Tensioning Tendons

FHWA *SNBI* item B.C.01 for deck condition rating represents the condition of the deck as determined from the inspection of all deck surfaces (top, underside, and edges). Refer to FHWA *SNBI* Subsection 5.3 for more information on coding this item.

Section 8.4 FRP Decks

8.4.1 Introduction

Fiber Reinforced Polymer (FRP) has been explored both in the repair and retrofit of existing structures as well as new bridge construction (see Figure 8.4.1). FRP composite decks are typically made of pultruded sections (e.g., honeycomb shaped, trapezoidal, or double-web I-beams). FRP was first used in the United States in the early 1990s.

Figure 8.4.1 Fiber Reinforced polymer (FRP) Deck

There are three types of FRP composite decks:

- Honeycomb sandwich.
- Solid core sandwich.
- Hollow core sandwich.

FHWA *SNBI* item B.SP.09 for deck material and type may be coded as "FRP composite – aramid fiber", "FRP composite – carbon fiber", "FRP composite – glass fiber", or "FRP composite – other" as appropriate.

Honeycomb Sandwich

Honeycomb sandwich construction will provide considerable flexibility in the depth of the deck. However, the hand lay-up process will need a large amount of observation to quality control when bonding the top and bottom face sheets to the core (see Figure 8.4.2).

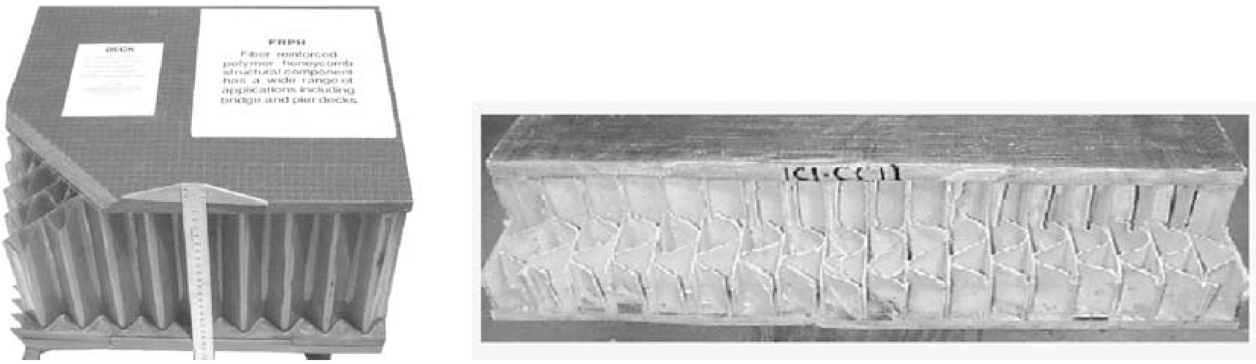


Figure 8.4.2 Honeycomb Sandwich Configuration

Solid Core Sandwich

Solid core sandwich decks contain foam or other fillers at the core. This type of deck is manufactured by using a process called Vacuum-Assisted Resin-Transfer Molding (VARTM) (see Figure 8.4.3).

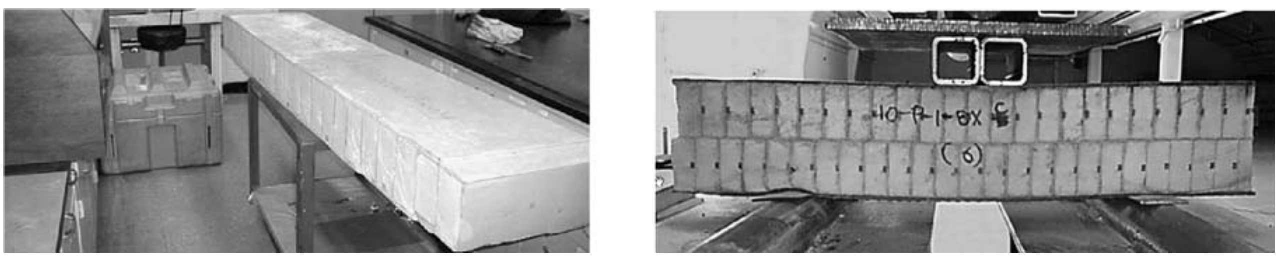


Figure 8.4.3 Solid Core Sandwich Configuration

Hollow Core Sandwich

Hollow core sandwich decks consist of deck sections that contain pultruded shapes that are fabricated in union (see Figure 8.4.4).

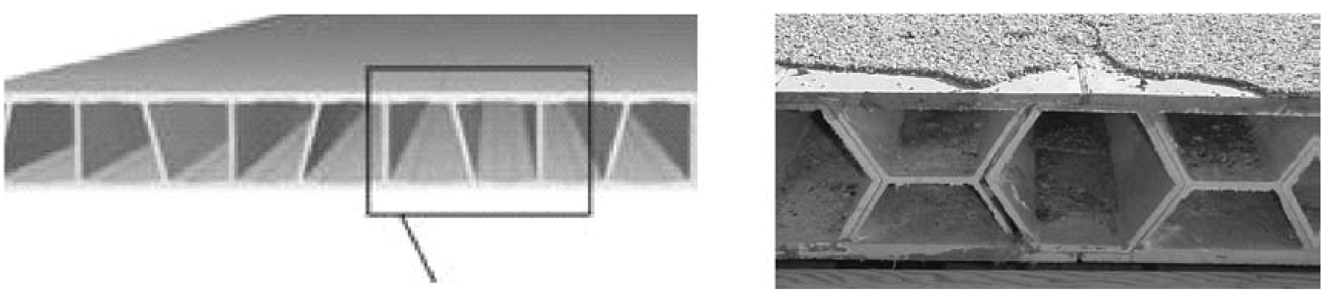


Figure 8.4.4 Hollow Core Sandwich Configuration

8.4.2 Wearing Surfaces

The low skid resistance of FRP materials make it necessary to overlay FRP decks. Bituminous or thin polymer-concrete overlays are typically used for the wearing surface of FRP decks.

Bituminous

The most common overlay material for FRP decks is bituminous 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 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.

8.4.3 Overview of Common Deficiencies

Common FRP deck deficiencies are listed below. Refer to Chapter 7 for a detailed description of these deficiencies:

- Blistering.
- Voids and Delaminated areas.
- Discoloration.
- Wrinkling.
- Fiber exposure.
- Scratches.

8.4.4 Inspection Methods

Visual and Physical

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 8.4.5).

Figure 8.4.5 Use of Truck for Visual Inspection of FRP Deck

Tap testing is the most common method for physical 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 8.4.6). However, the traditional tap test is preferred over the electronic method due to less time needed to perform the traditional test and the ineffectiveness of an electronic tap test for certain deck sections, such as sections with varying thicknesses.

Figure 8.4.6 Electronic Tap Testing Device

Advanced Inspection

If the extent of the FRP deficiency cannot be determined by visual and/or physical examination methods described above, advanced inspection methods may be used. Several advanced methods available for FRP inspection may be performed in the field by the bridge inspector or a certified NDE technician and some methods are performed off site in a lab. These advanced inspection methods are listed below.

- Acoustic emission testing.
- Ultrasonic testing.
- Laser-based ultrasound testing.
- Tap testing.
- Thermal testing.

All advanced methods are described in detail in Chapter 17.

8.4.5 Inspection Areas and Deficiencies

Prior to inspecting specific areas below, the FRP decks should be inspected for general global alignment. The curb line, top of railing, and the bottom of the deck should be viewed and examined for vertical or horizontal misalignment. Both sides of expansion joints should be examined to check if they line up properly. Vertical misalignment, horizontal misalignment or sagging may be an indication of specific details listed below. Misalignment of expansion joints may be evidence of differential substructure settlement or deficiencies in the bearing devices. Also, inspectors should look at the condition of secondary members. Distorted secondary members may be evidence of primary member differential movement or overstress. Misalignment may be indicators of potential deficiencies. Both the top and bottom surfaces of FRP decks should be inspected for any blistering, delaminated areas, discoloration, wrinkling, fiber exposure, scratches, or cracking.

Refer to Chapter 7 for a detailed description of FRP deficiencies.

Visual, physical, or advanced inspection methods may be needed to evaluate the following inspection areas and deficiencies:

- Deck panel splice joints - reflective cracking or oozing of joint material should be checked which may indicate movement or improper fitment between panels (see Figure 8.4.7).
- Deck panel butt joints - where joints are left exposed on the deck underside, the gap between panels should be measured.
- Vicinity of joints - signs of delamination or spalling in general area around the joint should be investigated. Tap tests should be performed to detect possible delamination.
- Areas exposed to traffic – the inspection team should examine for surface texture and wheel ruts due to wear.
- Areas exposed to drainage - ponding water and delamination should be investigated.
- Top of deck - at the expansion joints, the inspection team should check for signs of buckling, misalignment, differential vertical or horizontal movement.
- Underside of deck - near support beams or abutments, the inspection team should look for discoloration, signs of flow, cracks, or other signs of distress (see Figure 8.4.8).
- Haunch areas - separation between deck and haunch or supporting superstructure component should be inspected and measured. The inspection team should also note any cracking of haunch grout material.
- Deck support areas - tap tests should be performed near supports to check for delamination.
- Connections - all clip-type connections should be checked (see Figure 8.4.9) for tightness, soundness, scratches, abrasion, signs of movement, or any cracks in FRP from bearing against bolt for clip connection.

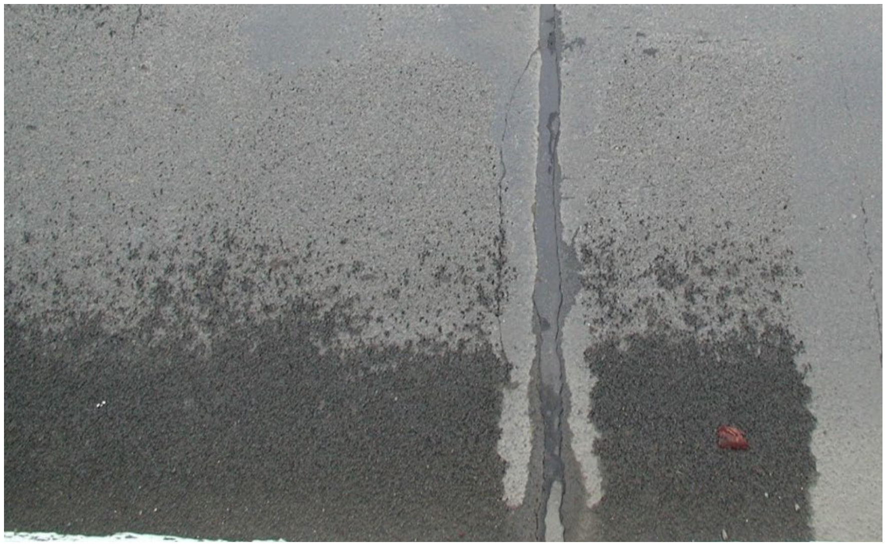


Figure 8.4.7 FRP Panel Splice

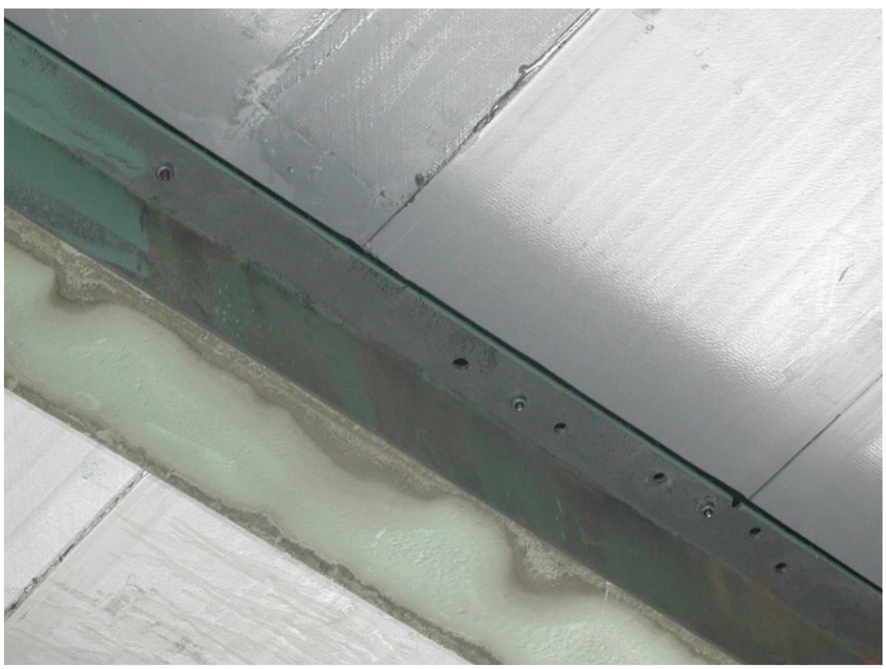


Figure 8.4.8 FRP Deck Underside Near Superstructure Beam

Figure 8.4.9 Clip-type Connection Between FRP Deck and Steel Superstructure

FHWA *SNBI* item B.C.01 for deck condition rating represents the condition of the deck as determined from the inspection of all deck surfaces (top, underside, and edges). Refer to FHWA *SNBI* Subsection 5.3 for more information on coding this item.

Section 8.5 Deck Joints, Drainage Systems, Lighting, and Signs

8.5.1 Function of Deck Joints, Drainage Systems, Lighting, and Signs

Deck Joints

The primary function of deck joints is to accommodate the expansion, contraction and rotation of the deck and superstructure. Depending on the design, deck joints are usually at each abutment, above piers in multiple span non-continuous bridges, or at the ends of a drop-in span. For recent bridge construction, jointless decks or integral abutments have been used to minimize or eliminate the number of deck joints, but trade-offs include excessive movement at the abutments or approaches. In most bridges, the deck joints accommodate this movement and prevent runoff from reaching bridge members below the surface of the deck. In addition, the deck joint provides a smooth transition from the approach roadway to the bridge deck.

Drainage Systems

The function of a drainage system is to remove water and all hazards associated with it from the structure. Removal of water also protects the superstructure, bearings, and substructure. The drainage system should be low-maintenance and placed 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 the bridge. Navigational lighting is used for the safe control of waterway traffic. 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. Guide or directional signs may be present to help guide motorists to their desired destinations.

8.5.2 Deck Joints, Drainage Systems, Lighting, and Signs

Deck Joints

Deck joints are primarily used to facilitate expansion, contraction, and rotation of the deck and superstructure. The seven categories of deck joints are:

- Strip seal expansion joint.
- Pourable joint seal.
- Compression joint seal/cellular seal.
- Assembly joint with seal.
- Other joint.
- 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 8.5.1 and Figure 8.5.2).

Figure 8.5.1 Strip Seal

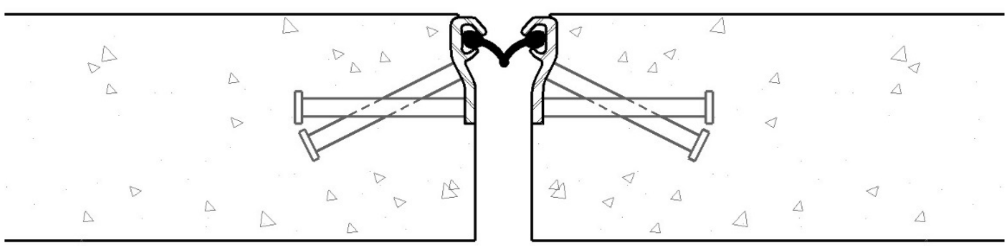


Figure 8.5.2 Cross Section of a Strip Seal

Pourable Joint Seal

A pourable joint seal is made up of three materials: backing material, preformed joint filler, and poured sealant (see Figure 8.5.3 and Figure 8.5.4). 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. The pourable joint seal can only accommodate a movement of about ¼ inch.

Neoprene foam can be used as an alternative to the preformed expansion filler, allowing a movement of greater than ¼ inch. Both types are typically used mostly for short span bridges.

Figure 8.5.3 Pourable Joint Seal

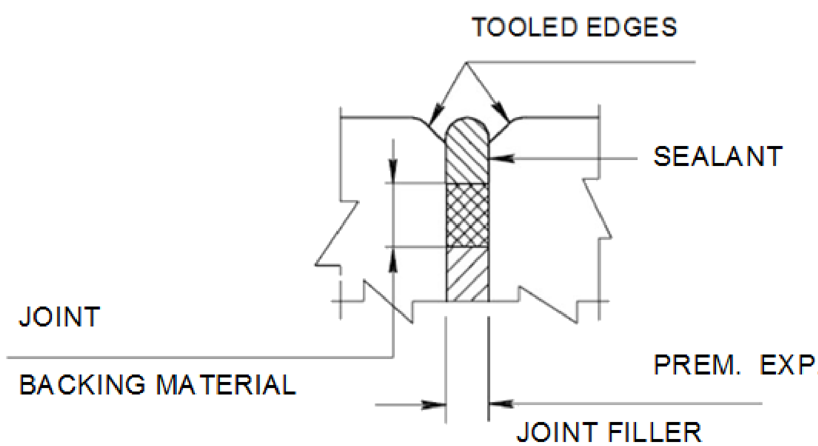


Figure 8.5.4 Cross Section of a Pourable Joint Seal

Compression Joint Seal and Cellular Seal

A compression joint seal consists of neoprene formed in a rectangular shape with a honeycomb cross section (see Figure 8.5.5 and Figure 8.5.6). 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 abutment 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 2.5 inches.

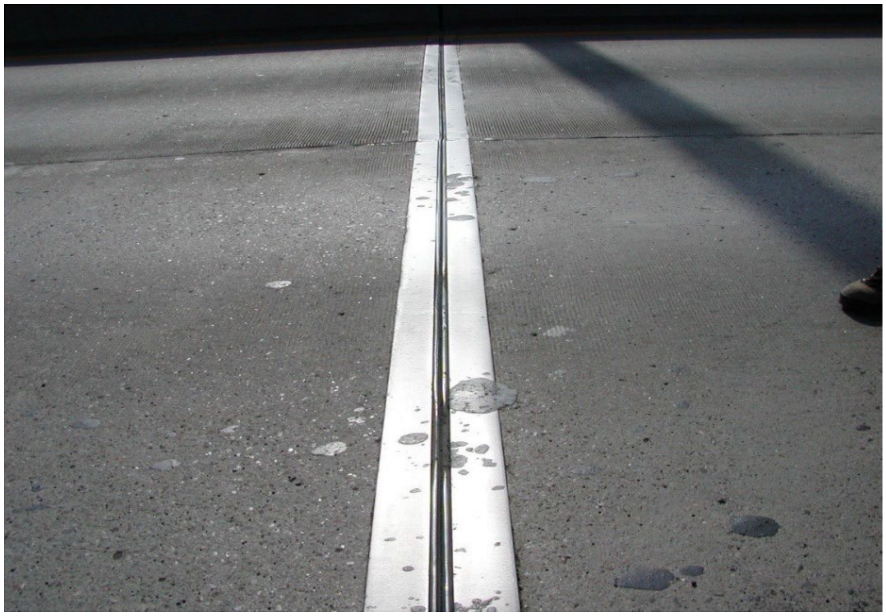


Figure 8.5.5 Compression Joint Seal with Steel Angle Armoring

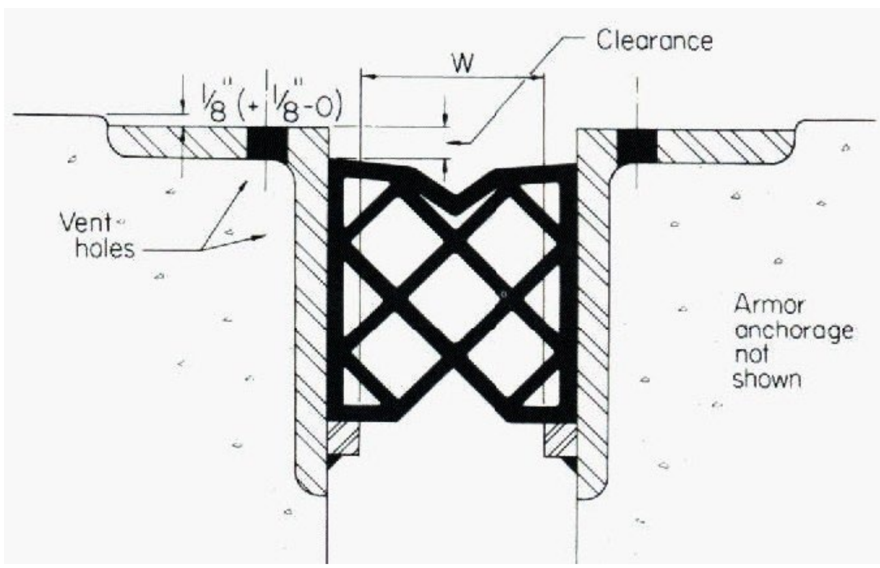


Figure 8.5.6 Cross Section of a Compression Joint Seal with Steel Angle Armoring

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 (see Figure 8.5.7). This foam allows for expansion and contraction both parallel and perpendicular to the joint. Unlike the compression joint seal, the cellular seal allows the joint to move in different directions without losing its contact with the deck or armor anchorage. 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. A large cellular seal can accommodate a maximum movement of approximately 2.5 inches.

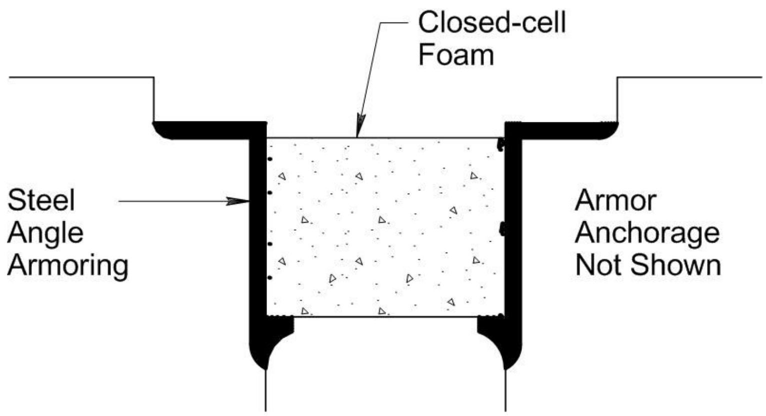


Figure 8.5.7 Cross Section of a Cellular Seal

Assembly Joint with Seal: Modular Seal

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 8.5.8 and Figure 8.5.9). 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.

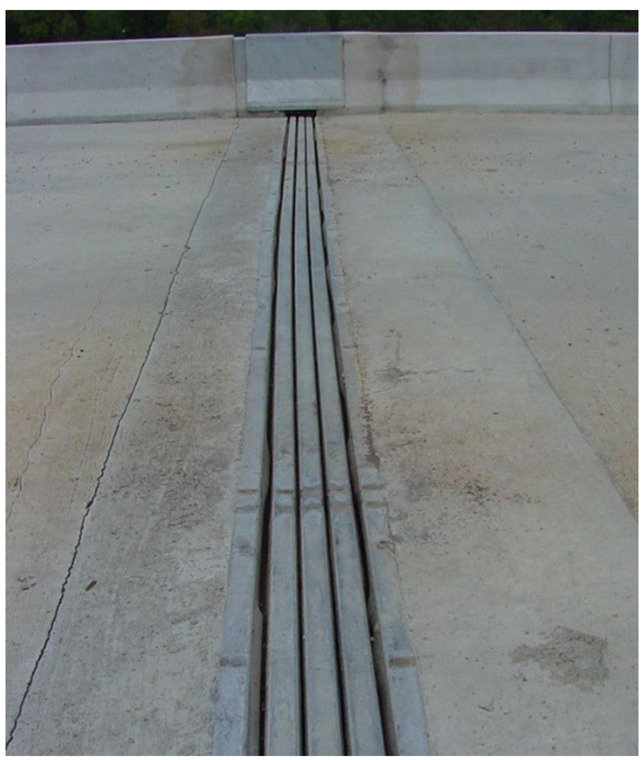


Figure 8.5.8 Modular Seal

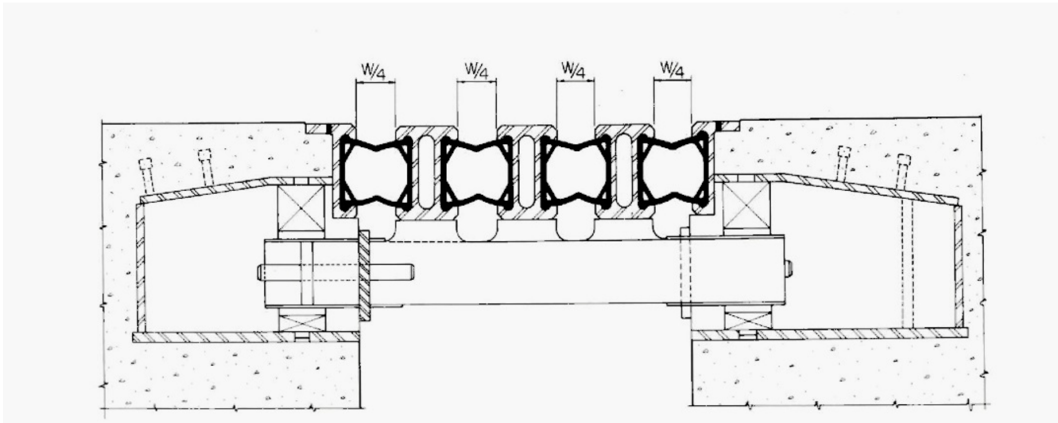


Figure 8.5.9 Schematic Cross Section of a Modular Seal

Assembly Joint with Seal: 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 8.5.10). Plank seals are no longer commonly used.

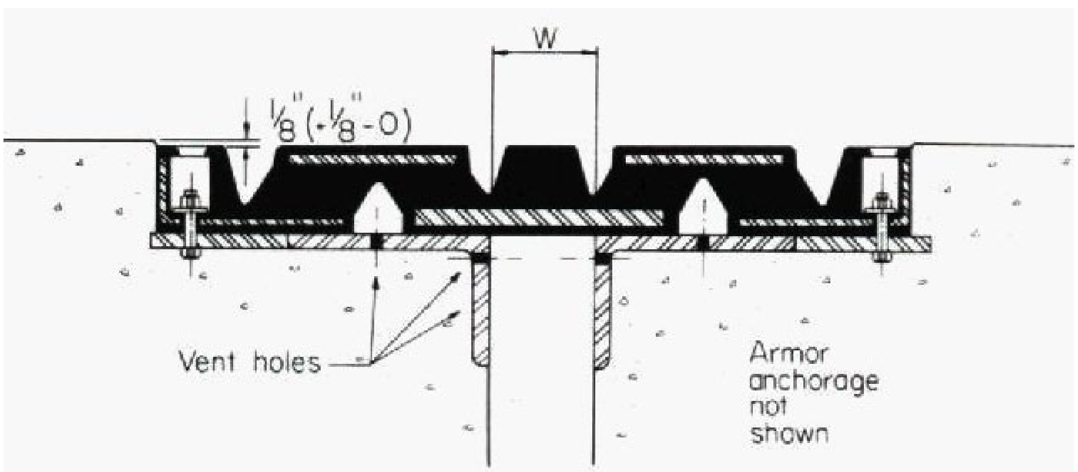


Figure 8.5.10 Plank Seal

Assembly Joint with Seal: Sheet 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 8.5.11).

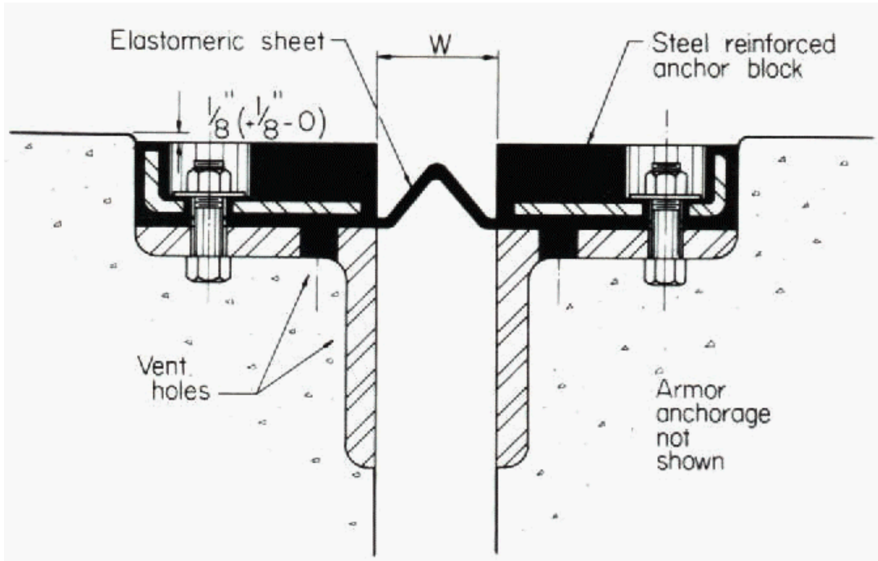


Figure 8.5.11 Sheet Seal

Asphaltic Expansion Joint

An asphaltic expansion joint is typically used on short bridges. This joint is also known as an Asphalt Plug Joint. Occasionally, these joints are overlaid with asphalt. The joint can accommodate an expansion movement up to 1 inch. Often these joints are typically used to retrofit another type of leaking small movement joint. 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 asphalt 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. (see Figure 8.5.12).

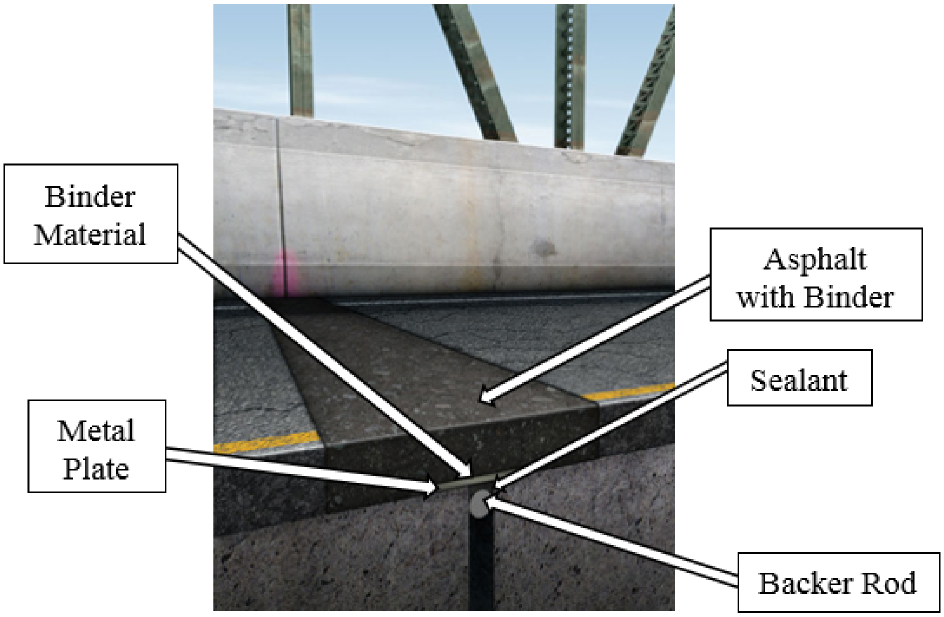


Figure 8.5.12 Asphaltic Expansion Joint

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 Figure 8.5.13 and Figure 8.5.14) in areas that do not use deicing material.

Figure 8.5.13 Open Expansion Joint

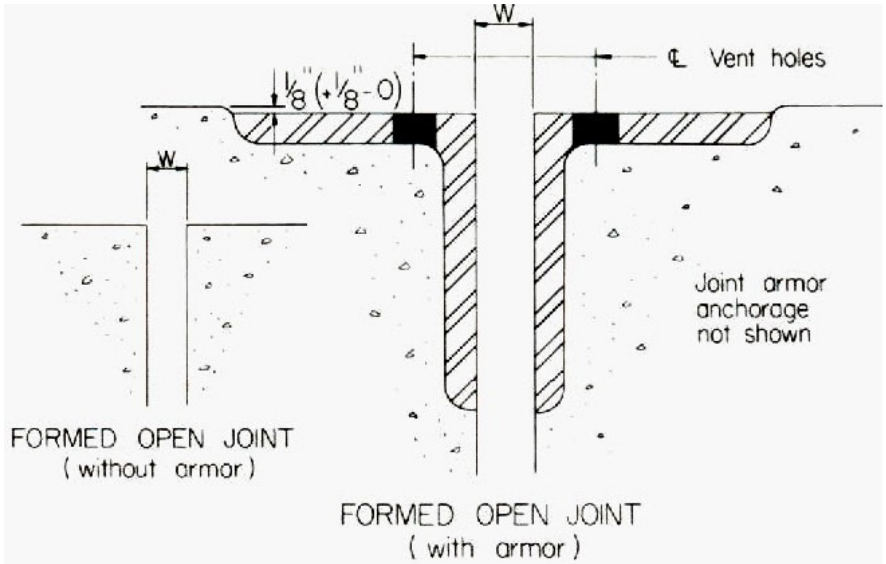


Figure 8.5.14 Cross Section of an Open Expansion Joint

Assembly Joint without Seal

Assembly joints without a seal can include finger plate joints and sliding plate joints.

Assembly Joint without Seal: Finger Plate Joint

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 necessary. 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 necessary. 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 primarily used to accommodate movement from 4 to over 24 inches (see Figure 8.5.15 through Figure 8.5.17).

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.

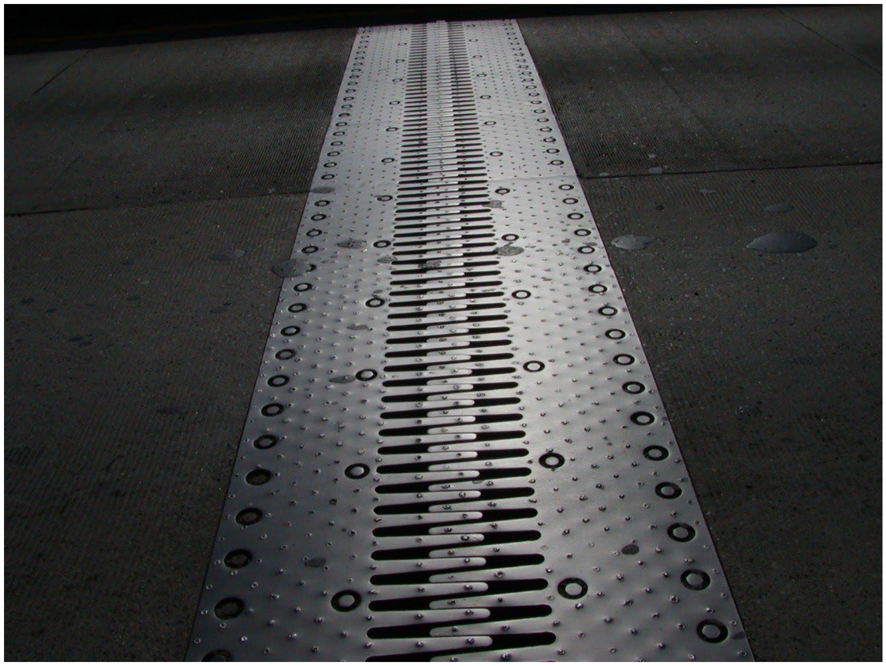


Figure 8.5.15 Finger Plate Joint

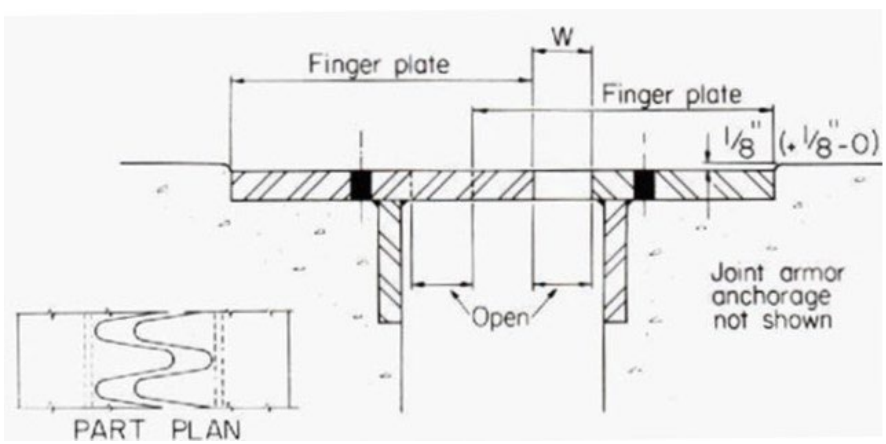


Figure 8.5.16 Cross Section of a Cantilever Finger Plate Joint

Figure 8.5.17 Supported Finger Plate Joint

Assembly Joint without Seal: 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 8.5.18 and Figure 8.5.19). The sliding plate joint can accommodate a maximum movement of approximately 4 inches.

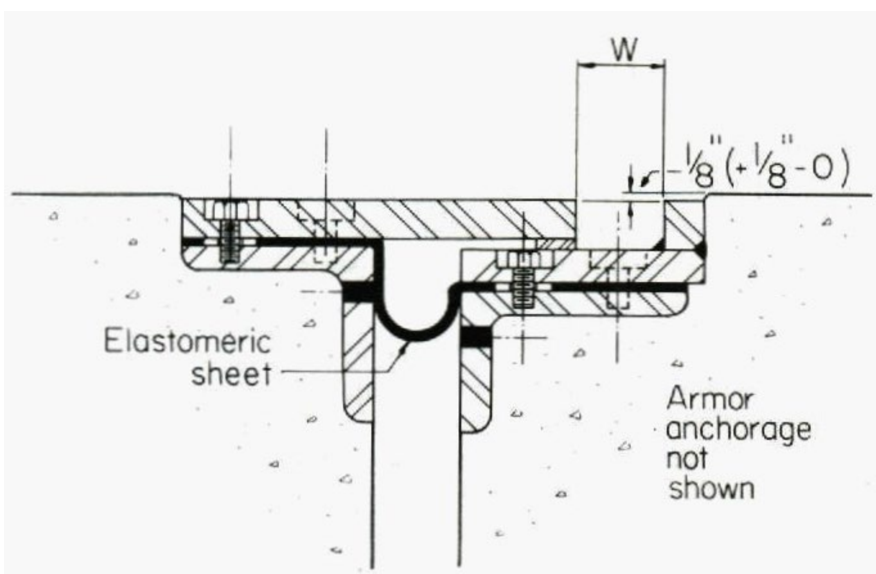


Figure 8.5.18 Cross Section of a Sliding Plate Joint

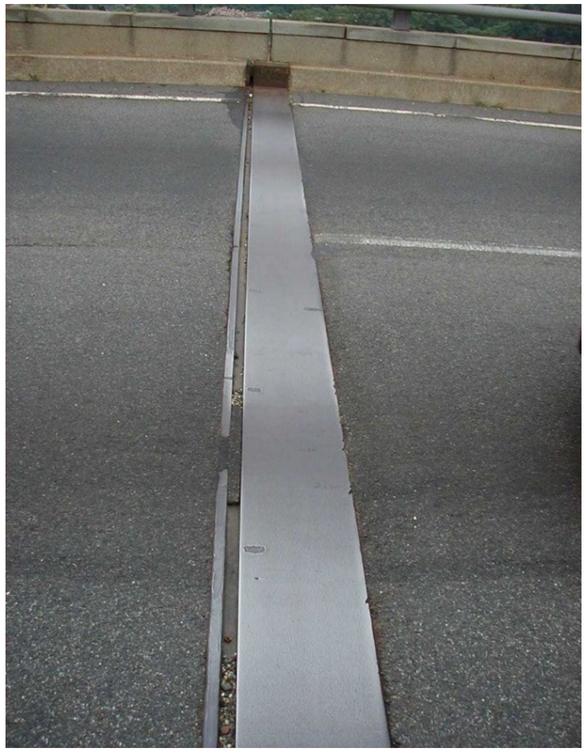


Figure 8.5.19 Sliding Plate Joint

FHWA *SNBI* item B.C.08 for bridge joint condition rating represents the condition of the deck as determined from the inspection of all deck surfaces (top, underside, and edges). Refer to FHWA *SNBI* Subsection 5.3 for more information on coding this item.

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 parts:

- 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 discharge point or storm sewer.
- Cleanout plugs - removable plug in the piping system that allows access for cleaning.
- Drainage troughs – may be found under open joints to divert runoff away from underlying superstructure, bearings, and substructure members.

Deck Drainage

Inlet systems incorporate scuppers or deck drains (see Figure 8.5.20). 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 be open holes or embedded pipes that are made of plastic or metal and functions similarly to scuppers (see Figure 8.5.20).

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 8.5.20 Bridge Deck Scupper (left) and Deck Drain (right)

The outlet system may incorporate outlet pipes or downspouts. If present, the outlet pipe leads water away from the inlet system (see Figure 8.5.21). 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 situated 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 8.5.22).

Figure 8.5.21 Outlet Pipes

Figure 8.5.22 Downspout Pipe

Joint Drainage

Joint drainage systems use a separate gutter or trough (see Figure 8.5.23) to collect water that passes through an unsealed joint such as a finger plate joint or sliding plate joint. Once the water is collected here, the water is then transported away from the bridge members.

Debris from the deck runoff may cause the trough to clog frequently and may need frequent cleaning to enable them to function as designed. These systems may be constructed from copper, steel, or elastomeric sheeting.

Figure 8.5.23 Drainage Trough with Missing Bolts

Substructure Drainage

Substructure drainage consists of weep holes and underdrains. Weep holes are small drainage holes found in abutment stems and retaining walls which allow water to drain from behind the abutment (see Figure 8.5.24). This type of drainage reduces the hydrostatic pressure behind the substructure.

Underdrains are perforated pipes that are routed along the back face of the abutment or retaining wall and are channeled to a nearby waterway or storm water drainage systems.

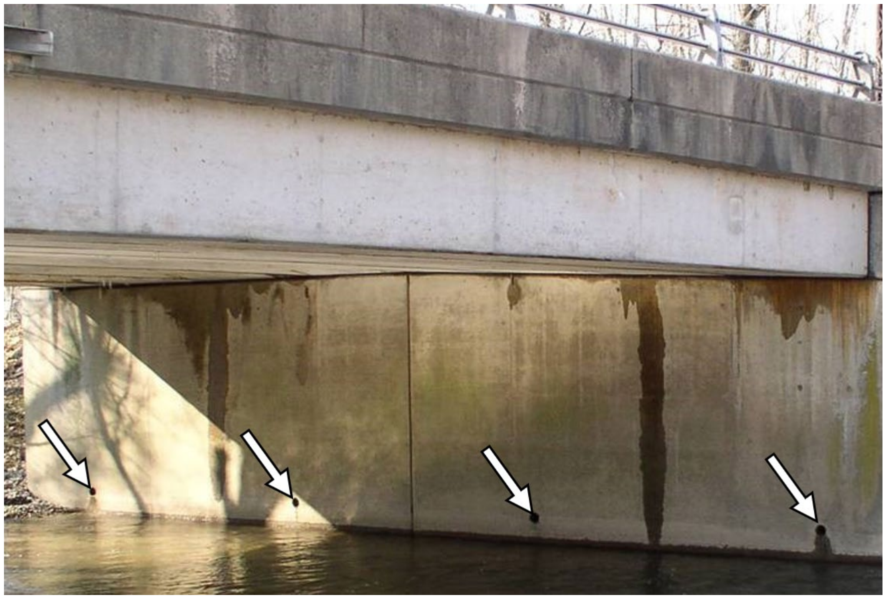


Figure 8.5.24 Weep Holes

Lighting

The four basic types of lighting that 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 luminaire attached to a bracket arm. The bracket arm is 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 primarily 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 typically 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 placed 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 necessary.

Navigation Lighting

Navigation lights are typically used for the safe control of waterway traffic. The United States Coast Guard determines the need for the type, number, and placement of navigation lights on bridges. The lights are 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 signs indicate the minimum vertical clearance for the structure. This clearance is measured at the most restrictive location within the traveling lanes. 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 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 that 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 signs are important since they indicate any speed restriction that may exist on the bridge. 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. These signs may be directly attached to the outside of the bridge railing or superstructure on over pass type bridges.

8.5.3 Inspection Methods

Visual and Physical

It is very important to use whatever inspection methods are necessary to fully ascertain conditions, including the causes and importance of each deficiency, so that serious and critical findings can be properly conveyed in the inspection report and other communication channels. The inspection of deck joints, drainage systems, lighting, and signs is primarily a visual activity.

Physical inspection methods may often be employed to supplement visual inspection methods. Physical methods are dependent upon the material of the deck joint, drainage system, lighting, or sign.

Refer to Chapter 7 for detailed procedures of visual and physical inspection methods of typical bridge materials.

Advanced Methods

Several advanced methods available for various materials used for deck joints, drainage systems, lighting and signs may be performed in the field by the bridge inspector or a certified NDE technician and some methods are performed off site in a lab. These testing methods are dependent upon the material used and the suspected deficiency. Refer to Chapter 17 for advanced inspection methods.

8.5.4 Inspection Areas and Deficiencies

Deck Joints

The deck joints should be inspected for:

- Dirt and debris accumulation.
- Proper alignment (horizontal/vertical).
- Damage to seals and armored plates.
- Indiscriminate overlays.
- Joint supports.
- Joint anchorage devices.
- Deck areas adjacent to deck joints.

Dirt and Debris Accumulation

Dirt and debris lodged in the deck 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 are continually driven into a deck joint, the joint material can eventually fail (see Figure 8.5.25 and Figure 8.5.26).

Figure 8.5.25 Deck Joint with Dirt and Debris Accumulation

Figure 8.5.26 Soil and Debris in a Compression Seal Joint

Proper Alignment

Any vertical or horizontal displacement between the two sides of the joint should be documented by the inspector. On straight bridges, the joint opening is typically designed to be parallel across the deck.

In a finger plate joint, the individual fingers will mesh properly, and they should be in the same plane as the deck surface. Any vertical or horizontal misalignment should be documented (see Figure 8.5.27).

Figure 8.5.27 Improper Vertical Alignment at a Finger Plate Joint

The current temperature and the deck joint opening should be recorded to determine if the opening is consistent with the temperature. 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 should 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 snowplows, traffic, and debris can cause the joint seals to be torn, pulled out of the anchorage, or removed altogether (see Figure 8.5.28). It can also cause damage to armored plates. The inspection team should look for evidence of leakage through sealed joints. Refer to FHWA *SNBI* Appendix C Table 53 for a descriptive list of bridge joint defects and descriptions of severity. Defects in this table include leakage, seal adhesion, seal cracking, seal damage, debris impaction, adjacent deck or header, and metal deterioration or damage.

Figure 8.5.28 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 8.5.29).

Figure 8.5.29 Asphalt Wearing Surface over an Expansion Joint

Joint Supports

Joint supports are necessary when large deck joints are utilized. These supports connect the deck joint devices to the superstructure. Water can leak through deck joints. The joint supports should be inspected carefully for proper function and for corrosion and section loss (see Figure 8.5.30).

Figure 8.5.30 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.

Deck Areas Adjacent to Deck Joints

Many deck joints are connected to the deck utilizing some type of armoring or anchorage (see Figure 8.5.2 and Figure 8.5.9). Inspectors should examine deck areas adjacent to deck joints for material deterioration such as section loss, spalls, delaminated areas, and vehicular/snowplow damage. Deterioration of the deck in these areas may be an indication of problems with the anchorage.

Drainage Systems

Some bridge owners have developed Agency Defined Elements to evaluate drainage systems.

The following drainage system elements should be inspected for:

- Profile Grade.
- Inlets.
- Outlet pipes.
- Downspout pipes.
- Cleanout plugs.
- Drainage troughs.

Profile Grade

The deck profile grade should not prevent runoff from entering the deck drains and inlets. Inspectors should check to determine adequate profile is provided so that water runs off the bridge deck at a sufficient rate. Ponding is typically an indication of insufficient profile.

Inlets

Careful examination of the drainage elements should be performed at each bridge inspection since runoff conditions can change. Inspectors should 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 8.5.31). Standing water on the deck is considered a safety hazard.

Figure 8.5.31 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. It should be checked to see if outlet pipes are clogged or broken or not connected to the inlets.

Downspout Pipes and Cleanout Plugs

Downspout pipes are a series of pipes (see Figure 8.5.32). The inspection team should examine downspout pipes for split or disconnected pipes that may allow runoff to accelerate deterioration of the structure. The connections between the downspout pipes and substructure should be checked. If a pipe is embedded inside of a substructure unit such as a concrete pier wall, the area should be checked for cracking, delamination, or another 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 8.5.32). If there is evidence of clogged outlet pipes, the inspection team should make recommendations to remove the cleanout plugs and clear the debris. In colder climates, pipes are often damaged from water freezing and expanding.

Figure 8.5.32 Damaged Outlet Pipe with Cleanout Plug

Drainage Troughs

The inspection team should carefully examine drainage troughs under unsealed joints, if present. A buildup of debris can accelerate the deterioration of the trough or its supports and allow water to drain onto structural members (see Figure 8.5.33). If possible, inspectors should 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. Any evidence that indicates the trough is overflowing should be recorded.

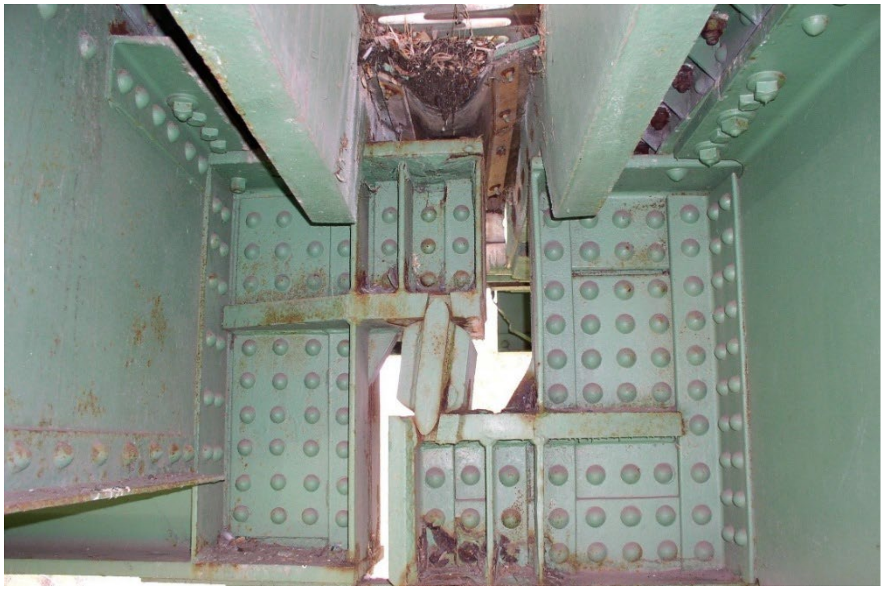


Figure 8.5.33 Drainage Trough with Debris Accumulation

Lighting

All lights are to be clearly visible. Inspectors should verify that all lights are functioning and that they are not obstructed from view. Light supports should be checked for fatigue cracking, corrosion, and collision damage. Inspectors should verify that appropriate lighting is provided. Caution should be exercised against electrical shock. Inspectors should contact the maintenance department to de-energize the lighting.

Signs

All signs should be clearly legible. Inspectors should verify that signs have not been defaced and are not obstructed from view. Sign supports should be inspected for fatigue cracking, corrosion, and collision damage. Inspectors should verify that appropriate signing is provided.

Adhesive Anchors

Adhesive anchors have several applications used in bridge construction. Two prominent applications include fence supports and light or sign support attachments. (see Figure 8.5.34).

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 indications 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 inspection team should look for anchor embedment problems, or anchor pullout, that results from adhesive failure. Fence or light pole anchors will often be subjected to moment and not axial tension. Axial tension anchorages are not very common unless the attachment is "hung" from the bridge.

Figure 8.5.34 Light Pole Attached to a Bridge

The inspection team should give particular observation to any anchor pullout that may exist. This could be caused by excessive creep or failure of the adhesive. Inconsistent spacing between the anchor plate and concrete surface should be checked (see Figure 8.5.35). This could occur from axial tension load or tension due to moment.

Figure 8.5.35 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 8.5.36). It is important to not only document the anchor location and orientation, but to determine, as close as possible, how the anchor functions. Any gaps between the mounting hardware and the concrete surface where the anchor is embedded should be noted.

Figure 8.5.36 Sign Mount with Loose Adhesive Anchorage

Section 8.6 Roadside Hardware

8.6.1 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), roadside hardware such as highway or bridge barrier systems have been provided to screen motorists from the hazards present (see Figure 8.6.1). Such barriers sometimes constitute fixed object hazards themselves, though hopefully of less severity than the hazard they screen.

Figure 8.6.1 Roadside Hardware

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 transition 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 transition guardrail systems should do all of this within tolerable deceleration limits for seat-belted occupants.

Basic Roadside Hardware

Roadside hardware systems at bridge sites are composed of two basic parts:

- Bridge railings.
- Transitions.

Other traffic safety features include the approach guardrail and end treatments, or approach guardrail ends. Although not inventoried or inspected as part of the bridge inspection, they also serve to redirect errant vehicles. Roadside hardware is designed to satisfy agency standards that specify heights, materials, strengths, and geometric features. Reference the AASHTO *Roadside Design Guide* and AASHTO *Manual for Assessing Safety Hardware (MASH)* for additional guidance.

Bridge Railings

The function of the bridge railing is to contain and smoothly redirect errant vehicles on the bridge. Many bridge rails could conceivably do this, but the safety of the driver and redirection of the vehicle should be considered.

Transitions

A transition occurs between the approach guardrail system and bridge railing (see Figure 8.6.2 and Figure 8.6.3). 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 unsafe vehicular decelerations. When a portion of the test vehicle, such as a wheel, engages a vertical member 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 also cause large and unsafe vehicular decelerations.

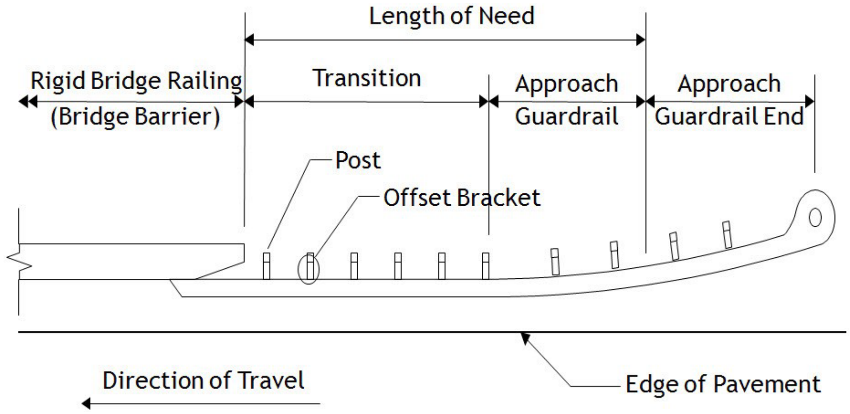


Figure 8.6.2 Roadside Hardware

Figure 8.6.3 Bridge Railing Transition

8.6.2 Design Criteria

The various parts of roadside hardware are designed to meet a specific function. Based on items from an inspection checklist, the inspection team can decide whether or not each one functions as intended. Bridge railings and transition guardrail systems should pass the minimum standard criteria established by AASHTO, FHWA, and NCHRP minimum standards for structures on the NHS.

Until the mid-1980s, 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 included specific dimensions. Full-scale crash testing was not necessary, although a design that "passed" such testing was also considered allowable 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 should be successfully crash tested and approved to be considered allowable for use on Federal-aid projects. Crash testing criteria has been updated several times since 1986. See the next three sections for History of Crash Testing, Crash Test Criteria, and Current FHWA Policy.

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 unsafe designs and prompted development of several new guardrail systems and details that were then identified as tolerable 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 necessary 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 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 necessary documentation on vehicle rebound for crash cushion tests.
- Changes in test documentation and performance evaluation.

In December 2015, the AASHTO/FHWA Joint Implementation Agreement for AASHTO MASH was successfully balloted by AASHTO's Standing Committee on Highways and approved by FHWA. The Agreement helps encourage the application of the newest and safest generation of roadside hardware. It defined actions needed for full implementation of AASHTO's MASH over the course of several years. Status of the AASHTO MASH Implementation can be found here: https://safety.fhwa.dot.gov/roadway_dept/countermeasures/reduce_crash_severity/aashto_mash_implementation.cfm.

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 necessary 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 tolerable based on Report 230 testing could also be considered allowable 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 roadside hardware. 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 necessary 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

On May 26th, 2017, FHWA issued a letter to the highway safety hardware and roadside design community documenting the requirements to attain a Federal-aid eligibility letter from FHWA for roadside safety hardware systems. The letter can be found at: https://highways.dot.gov/sites/fhwa.dot.gov/files/2022-06/openletter052617.pdf.

A FHWA memo issued on April 9, 2018 stated that "An eligibility letter is not a requirement for roadside safety hardware to be determined eligible for Federal funding. Roadside safety hardware is eligible for Federal funding if it has been determined to be crashworthy by the user agency (i.e., State DOT)." The memo can be found at: https://safety.fhwa.dot.gov/roadway_dept/countermeasures/reduce_crash_severity/docs/memo040918.pdf.

Bridge railings to be installed on National Highway System (NHS) projects with a letting date after December 31, 2019, should meet the acceptance criteria contained in AASHTO MASH (see Table 8.6.1). The minimum allowable 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 45 mph or less, a TL-2 bridge railing is considered tolerable.

Table 8.6.1 2017 AASHTO LRFD Bridge Design Specification, Bridge Railing Test Levels and Crash Test Criteria

	Vehicle Characteristics	Small Automobiles		Pickup Truck	Single-Unit Van Truck	Van-Type Tractor-Trailer		Tractor-Tanker Trailer
NCHRP Report 350	*W* (kips)	1.55	1.8	4.5	18.0	50.0	80.0	80.0
	B (ft.)	5.5	5.5	6.5	7.5	8.0	8.0	8.0
	G (in.)	22	22	27	49	64	73	81
	Crash angle, θ	20°	20°	25°	15°	15°	15°	15°
	Test Level	Test Speeds (mph)						
	TL-1	30	30	30	N/A	N/A	N/A	N/A
	TL-2	45	45	45	N/A	N/A	N/A	N/A
	TL-3	60	60	60	N/A	N/A	N/A	N/A
	TL-4	60	60	60	50	N/A	N/A	N/A
	TL-5	60	60	60	N/A	N/A	50	N/A
	TL-6	60	60	60	N/A	N/A	N/A	50
AASHTO MASH	*W* (kips)	2.42	3.3	5.0	22.0	N/A	79.3	79.3
	B (ft.)	5.5	5.5	6.5	7.5	N/A	8.0	8.0
	G (in.)	N/A	N/A	28	63	N/A	73	81
	Crash angle, θ	25°	N/A	25°	15°	N/A	15°	15°
	Test Level	Test Speeds (mph)						
	TL-1	30	N/A	30	N/A	N/A	N/A	N/A
	TL-2	45	N/A	45	N/A	N/A	N/A	N/A
	TL-3	60	N/A	60	N/A	N/A	N/A	N/A
	TL-4	60	N/A	60	55	N/A	N/A	N/A
	TL-5	60	N/A	60	N/A	N/A	50	N/A
	TL-6	60	N/A	60	N/A	N/A	N/A	50

Crash-test level for bridge railings and transitions is reported in FHWA *SNBI* items B.RH.01 and B.RH.02, respectively. The inspection team is not expected to enter or alter these codes.

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: https://safety.fhwa.dot.gov/roadway_dept/countermeasures/reduce_crash_severity/.

For non-NHS projects, the setting of criteria for establishing acceptability for bridge railings has been relegated by the FHWA to the individual States and Tribal governments. 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 roadside hardware acceptability.

Railing Evaluation Results/Resources

All of the bridge and longitudinal roadside barrier systems, transitions, and approach guardrail ends that 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 can be found at: https://safety.fhwa.dot.gov/roadway_dept/countermeasures/reduce_crash_severity/.

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. 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 that meet current NCHRP Report 350 crash test performance criteria are found at: https://safety.fhwa.dot.gov/roadway_dept/countermeasures/reduce_crash_severity/listing.cfm?code=long&filter=MASH.

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 Design.

This three-day course discusses the use of the *AASHTO Roadside Design Guide* including applying the clear zone concept, identifying the need for a traffic barrier, recognizing unsafe roadside design features and making appropriate changes, identifying the need for a traffic barrier, and applying other highway core competencies.

FHWA-NHI-133116 Maintenance of Traffic for Technicians.

The Maintenance of Traffic for Technicians Web-based training presents information about the placement of, field maintenance necessary for, and inspection of traffic control devices. In addition, drafting work zone traffic control plans and flagging are discussed. This training focuses on the design of a traffic control plan, and how and why one needs to operate and implement traffic control in the work zone.

FHWA-NHI-133117 Maintenance of Traffic for Supervisors.

The Maintenance of Traffic for Supervisors Web-based training presents information about the placement of, field maintenance necessary for, and inspection of traffic control devices. In addition, drafting work zone traffic control plans and flagging are discussed. This training focuses on the design of a traffic control plan, and how and why one needs to operate and implement traffic control in the work zone.

More detail on the courses listed above can be found at: https://www.nhi.fhwa.dot.gov/home.aspx

8.6.3 Roadside Hardware Inspection

The inspection of bridge roadside hardware involves evaluation of the condition of the bridge railing, and the transition guardrail system and whether these two systems will likely function acceptably in union to safely contain and redirect errant vehicles that may collide with them.

For structures that are over roadways, the adequacy and condition of roadside hardware for both the upper and lower roadways are to be evaluated during the inspection.

FHWA *SNBI* items B.C.05 and B.C.06 report the condition ratings for bridge railings and bridge railing transitions, respectively. The condition assessment for these items includes the portions of the railings, posts, blocking, and curbs that are part of the bridge railing system.

8.6.4 Inspection Methods

Visual and Physical

It is very important to use whatever inspection methods are necessary to fully ascertain conditions, including the causes and importance of each deficiency, so that serious and critical findings can be properly conveyed in the inspection report and other communication channels. The inspection of roadside hardware is primarily a visual activity.

Physical inspection methods may often be employed to supplement visual inspection methods. Physical methods are dependent upon the material of the roadside hardware.

Refer to Chapter 7 for detailed procedures of visual and physical inspection methods of typical bridge materials.

Advanced Methods

Several advanced methods available for various materials used for roadside hardware may be performed in the field by the bridge inspector or a certified NDE technician and some methods are performed off site in a lab. These testing methods are dependent upon the material used and the suspected deficiency. Refer to Chapter 17 for advanced inspection methods.

8.6.5 Inspection Areas and Deficiencies

Criteria considered during the inspection of the roadside hardware features are the height, material, strength, geometric features, and the likelihood of tolerable crash test performance. Deficiencies should be recorded due to the condition separately in the inspection notes and condition ratings.

Many State agencies have developed their own criteria for bridge railings. An inspector should be familiar with their State's agency or Tribal government's roadside hardware standards.

Approach guardrail and end treatments are not part of the NBI, but damage or major deterioration should be documented in the inspection report and communicated.

Bridge Railing

Comparison of existing bridge railing systems with approved crash-tested designs will establish their acceptability and crash worthiness. The condition assessment included the portions of the railings, posts, blocking, and curbs that part of the bridge railing system (*SNBI* Subsection 7.1, Item B.C.05 Commentary).

Metal bridge railings should be firmly attached to the deck or superstructure and should be functional. Inspectors should check especially for corrosion and collision damage, which might render these railings ineffective (see Figure 8.6.4). Loose or missing connections should also be checked.

Figure 8.6.4 Steel Post Bridge Railing with Impact Damage

Concrete bridge railing is generally cast-in-place and engages reinforcing bars to develop structural anchorage in the deck or slab. It should be verified that the concrete is sound and that reinforcing bars are not exposed. Inspectors should check for impact damage or rotation and note areas of damage or movement.

Precast parapets should be checked for evidence of anchorage failure. A physical examination should be performed by sounding exposed anchor bolts with a hammer. Separations between the base of the precast units and deck, or evidence of active water leakage between parapet and deck should be checked. Some States are removing all precast parapets because water is seeping in along the curb line and corroding reinforcement. This reinforcement cannot be visually inspected.

Post and beam railing systems should be inspected for collision damage and deterioration of the various members. Post bases should be checked for loss of anchorage. The exposed side of the railing should be smooth and continuous.

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.

Transition

Like the bridge railing, the condition assessment included the portions of the railings, posts, blocking, and curbs that part of the bridge railing transitions (*SNBI* Subsection 7.1, Item B.C.06 Commentary). Inspectors should check the approach guardrail transition to the bridge railing for adequate structural anchorage to the bridge railing system. Inspectors should check for sufficiently reduced post spacing to assure stiffening of the guardrail at the approach to the rigid bridge rail end. Smooth transition details should be checked 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 8.6.5).

Figure 8.6.5 Proper Nesting of Guardrail at Transition

Inspectors should also check the railing, post, and offset bracket condition. Any deficiency of the transition that could weaken the system should be noted. Note any area of settlement or frost heave. Posts embedded in the ground should not be able to be moved by hand. Inspectors should check the slope beyond the posts for settlement or erosion which may reduce embedment of the posts (see Figure 8.6.6).

Figure 8.6.6 Exposed Transition Post due to Roadway Runoff Erosion

Inspectors should document any significant collision damage and report posts that are displaced horizontally. Inspectors should check for cracks, rust, or breakage of any section. The connection between rails and posts should be secure and tight. Any loose or missing bolts should be noted. Wood posts should be checked for rot or insect damage, especially at the ground line. Timber should not be used for the rails in transitions on the National Highway System (NHS).

Inspection for Non-NHS Bridges

The standards for inspection of roadside hardware presented in this section are applicable to bridges on the National Highway System (NHS). For bridges that are not on the NHS, it is up to each governing agency to set their own standards.

There are still various criteria that should be met as a minimum for these installations. The bridge railing should be crashworthy. The transition should be adequately connected to the bridge railing. Post spacing from the approach guardrail to the transition should be reduced to limit deflection. It is recommended to have nested rail at the transition, but it is not necessary.

8.6.6 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. 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 are assessed under *SNBI* Item ID B.C.05, bridge railings condition rating. Inspect for collision damage and attachment to any additional roadside hardware. Check for deterioration and spalling on concrete median barriers, and examine for corrosion and loose connectors on steel railings and posts.

Section 8.7 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of the various bridge features discussed in this chapter. The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

This page intentionally left blank.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 9 TABLE OF CONTENTS

CHAPTER 9 LIST OF FIGURES

Chapter 9 Inspection and Evaluation of Concrete Superstructures

Section 9.1 Characteristics

9.1.1 Introduction

There are a wide variety of concrete bridge superstructures that may be selected based on owner span length and site requirements. This section outlines the characteristics of the different structure types and details the typical configuration of each type. It also includes a discussion of typical primary and secondary members. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

The structure types included in this section are reported per *Specifications for the National Bridge Inventory (SNBI)* Item B.SP.06, Span Type. The type of concrete used for the principal material is reported using Item B.SP.04, Span Material.

9.1.2 Cast-In-Place Slab

Design Characteristics

The cast-in-place slab bridge is the simplest type of reinforced concrete bridge and was a common choice for construction in the early 1900s (see Figure 9.1.1 and Figure 9.1.2). The terms “deck” and “slab” are often confused. 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 described 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.

Figure 9.1.1 Typical Simple Span Cast-in-Place Slab Bridge

Figure 9.1.2 Typical Multi-Span Cast-in-Place Slab Bridge

Even though slabs are not decks, many of the design characteristics, wearing surfaces, protective systems, inspection methods, inspection areas, and evaluation are similar. Refer to Chapter 8 for further details on decks.

This type of bridge generally consists of one or more simply supported spans and spans are typically less than 30 ft long. Continuous multi-span slab bridges are also in service; but are not as common as simply supported slabs.

Primary Members

The only primary superstructure 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 in the bottom of the slab. The reinforcement is placed longitudinally from support to support, parallel to the direction of traffic (see Figure 9.1.3). For continuous spans, additional primary reinforcement is placed 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 used to prevent shear cracking, is typically obtained by bending the tension bars at a 45-degree angle close to the slab supports (see Figure 9.1.3). The shear reinforcement is perpendicular to diagonal tension/shear forces and therefore resists those forces.

Secondary reinforcement, known as temperature and shrinkage steel, is arranged transversely throughout the top and bottom of the slab (see Figure 9.1.3). In simple span slabs, secondary reinforcement is also arranged 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).

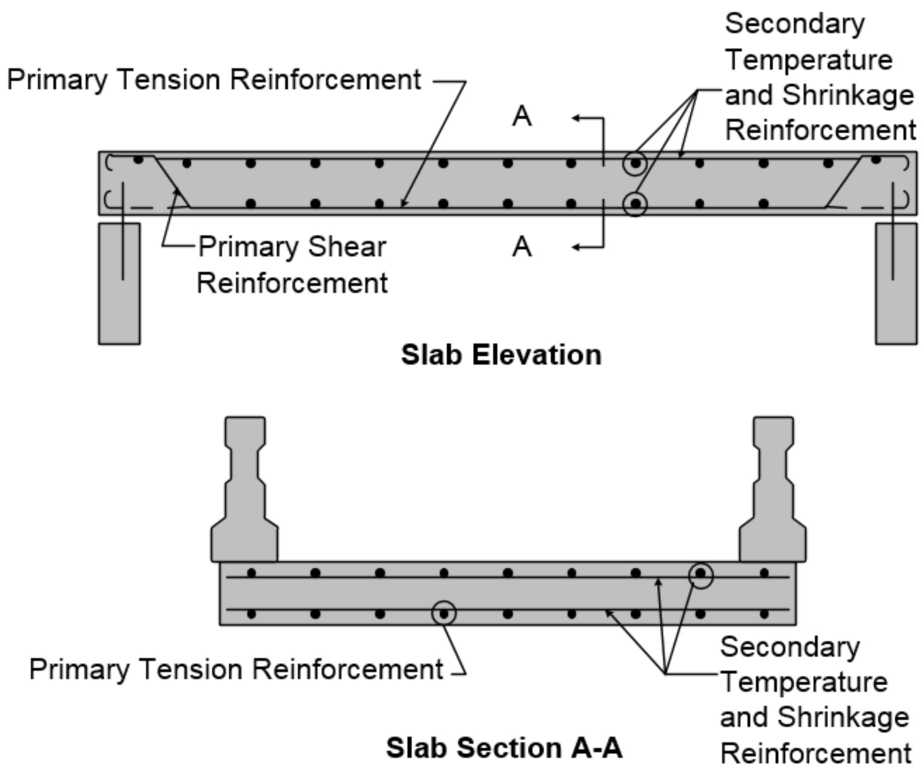


Figure 9.1.3 Steel Reinforcement in a Simply Supported Concrete Slab

9.1.3 Cast-In-Place Tee Beam

Design Characteristics

The concrete tee beam with conventional reinforcement, a predominant bridge type during the 1930s and 1940s, is generally a cast-in-place monolithic deck or top flange and stem system formed in the shape of the letter "T" (see Figure 9.1.4). The cast-in-place tee beam is a common type of tee beam. Some highway agencies use precast tee beam shapes that typically use prestressed reinforcement.

Figure 9.1.4 Simple Span Tee Beam Bridge

A typical cross section or a tee beam superstructure is shown in Figure 9.1.5. The deck, or top flange, portion of the beam is constructed to act integrally with the stem to achieve composite behavior and provides greater stiffness for increased span lengths. Span lengths for tee beam bridges are typically between 30 and 50 ft.

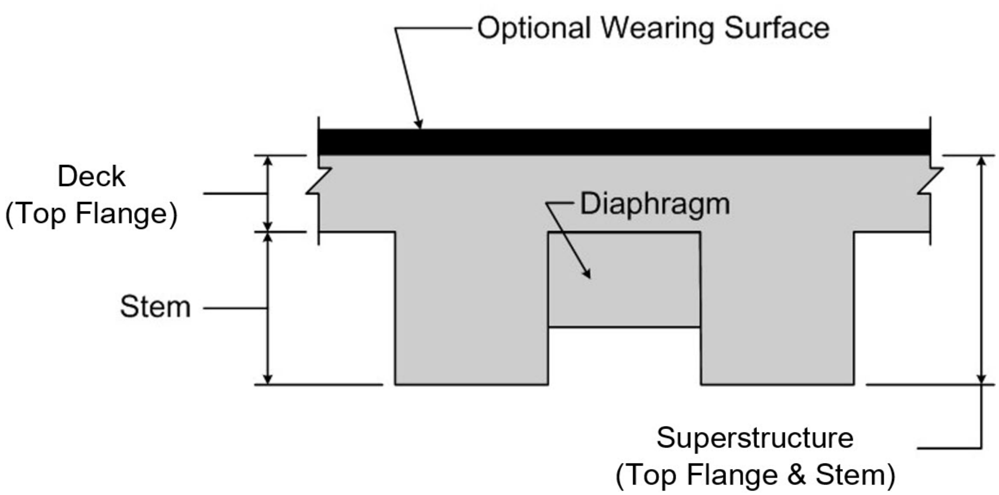


Figure 9.1.5 Cross Section - Tee Beam

Spacing of the tee beams is generally 3 to 8 ft, 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. A 3 inch or 4 inch fillet at the deck-stem intersection identifies this older form of construction (see Figure 9.1.6).

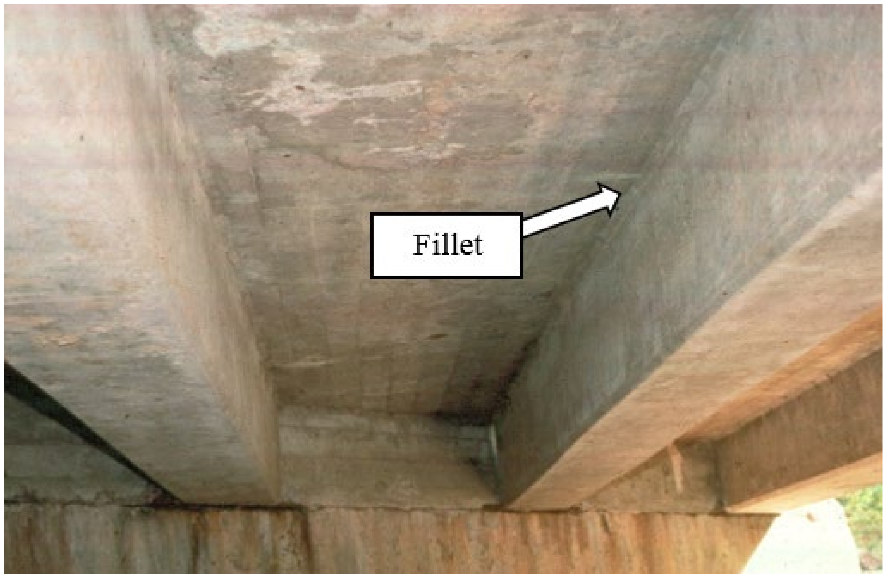


Figure 9.1.6 Typical Tee Beam Layout

It is important to not mistake a concrete encased steel I-beam bridge for a tee beam bridge. A review of plan or sketches in the bridge file should eliminate this problem. A spall on the bottom of the stem can also indicate if there are steel reinforcement bars of a tee beam present or the bottom flange of a steel I-beam (see Figure 9.1.8). Inspection of steel beams is presented in Chapter 10. The concrete encasement acts as a protective system for the steel beam.

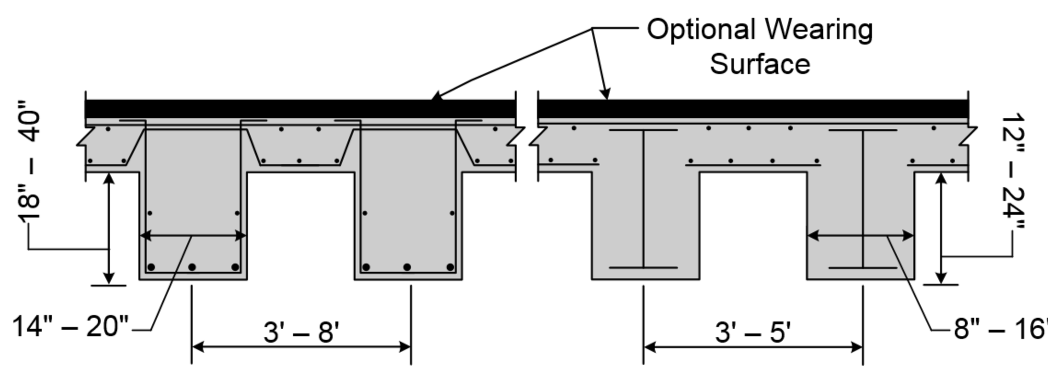


Figure 9.1.7 Comparison Between Tee Beam and Concrete Encased Steel I-beam

Figure 9.1.8 Concrete Encased Steel I-beam

Primary Members

The primary members of a tee beam bridge are the tee beam stem (web) and deck (flange) (see Figure 9.1.9). The primary members carry primary transient load and are supported by the substructure and the secondary members, or diaphragms, situated between the primary members (see Figure 9.1.9).

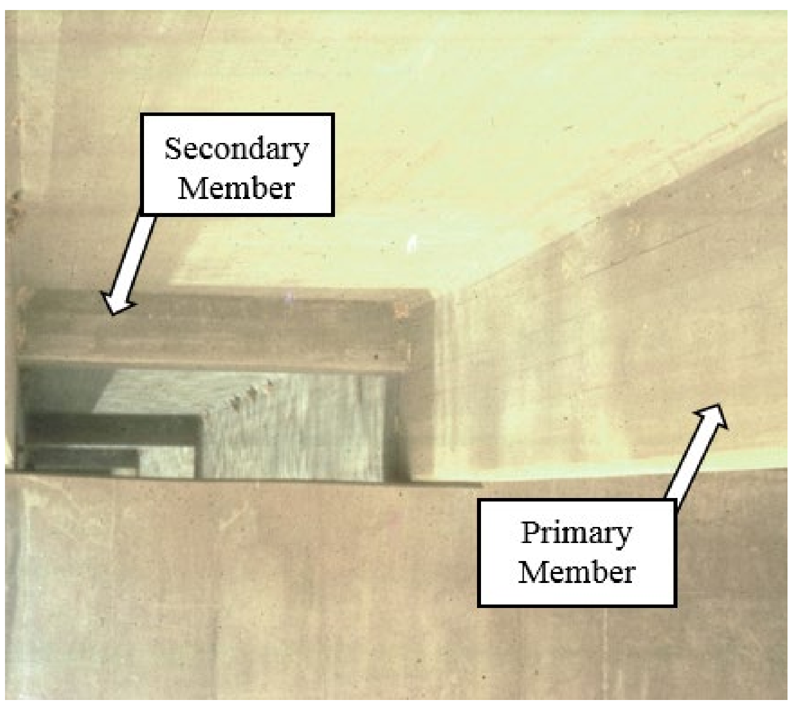


Figure 9.1.9 Tee Beam Primary and Secondary Members

Steel Reinforcement

The stem of a concrete tee beam has both primary and secondary reinforcing steel (rebar). The primary rebar consists of main tension reinforcement arranged longitudinally and shear reinforcement in the form of stirrups. The main tension reinforcement is in the bottom of the beam stem to resist tensile forces caused by positive moment (see Figure 9.1.10). The sides of the stem contain primary vertical shear reinforcement, called stirrups, established throughout the length of the stem at various spacings. Stirrups are generally U-shaped bars that run transversely across the bottom and sides of the stem (see Figure 9.1.10). The need for stirrups generally is greatest near the beam supports where shear stresses are the highest. Stirrup spacing is typically closer in the stem near the substructure supports. The secondary (temperature and shrinkage) reinforcing steel for the stem is oriented longitudinally in the sides (see Figure 9.1.10).

The primary and secondary reinforcing steel for the top flange portion of the beam is the same as a standard concrete deck (see Figure 9.1.10). Deck/top flange tension and shear reinforcement is transverse while temperature and deck/top flange shrinkage reinforcement is longitudinal. If the concrete tee beams are continuous, there will typically be additional longitudinal reinforcement close to the top surface of the deck/top flange over the piers to resist tensile forces caused by negative moment in the superstructure span.

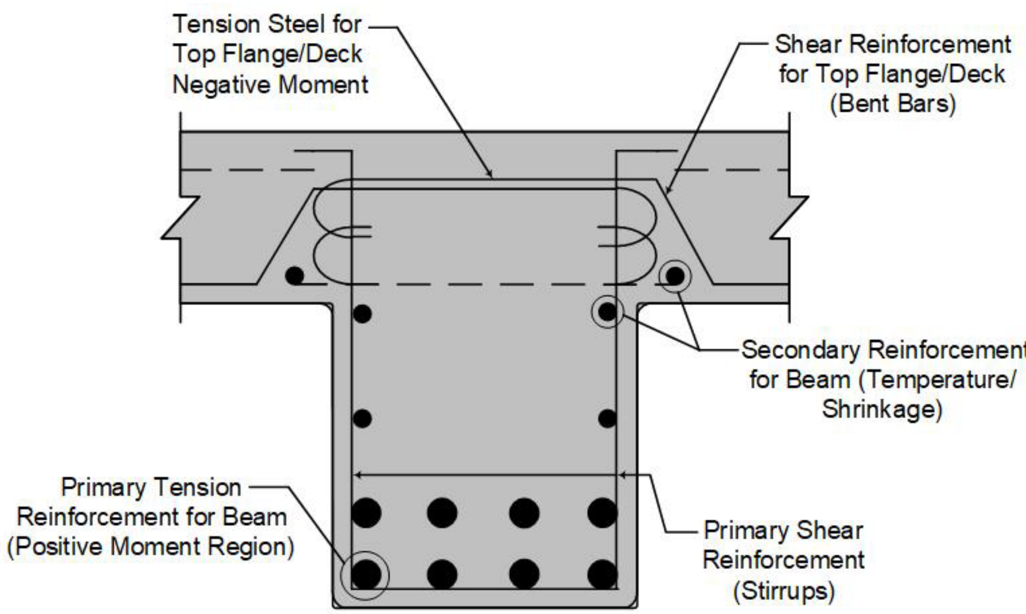


Figure 9.1.10 Cross Section - Steel Reinforcement in a Simple Span Concrete Tee Beam

9.1.4 Two-Girder System

Design Characteristics

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 (see Figure 9.1.12), or as through girders, where the deck is cast between the girders. Through girders are very large in appearance and serve as the bridge's parapets, or bridge rail, as well as the primary superstructure members.

Concrete through girders are generally used for spans ranging from 30 to 60 ft at locations with limited under-clearance. They are, however, not economical for wide roadways and are usually limited to approximately 24 ft wide. Girders are usually 18 to 30 inches wide and 4 to 6 ft deep. See Figure 9.1.13 for a typical cross section of a concrete through girder bridge.

Figure 9.1.11 Concrete Through Girder Bridge

For 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 9.1.12).

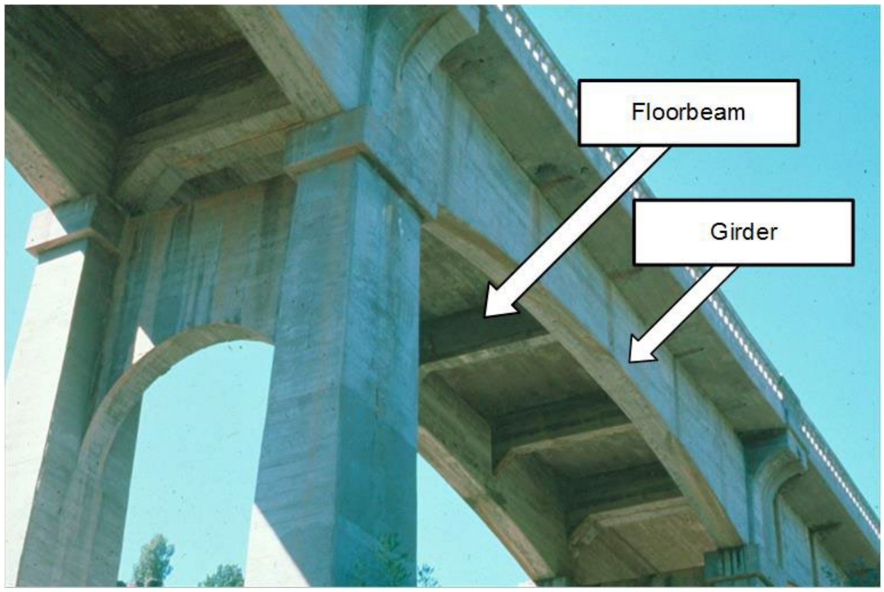


Figure 9.1.12 Concrete Deck Girder Bridge

Care should be taken not to describe concrete two-girder bridges as composite. They are not considered composite because the concrete girders and deck consist of the same material, even though they are rigidly connected with steel reinforcement.

Primary Members

The primary superstructure members of a two-girder bridge are the girders, floorbeams (if present) and stringers (if present).

Sometimes there can be confusion about whether a transverse member is a floorbeam or a diaphragm. If available, design drawings should be consulted to look at the reinforcement of these members. Typically, diaphragms are minimally reinforced while floorbeams will have more steel reinforcement. In the absence of drawings, inspectors should compare the spacing of the deck girders and the transverse members. Decks are typically 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 (see Figure 9.1.12). If stringers are present, the transverse members should be considered floorbeams and therefore primary members.

Steel Reinforcement

The primary reinforcing steel for the girders consists of main longitudinal tension reinforcement and shear reinforcement in the form of stirrups or inclined rebars (see Figure 9.1.13). Main tension reinforcement is in the bottom of the girder for positive moment resistance. The top of the girder can also contain main tension reinforcement when there is a negative moment due to continuous spans. The beam also contains shear reinforcement, called stirrups that are placed 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 9.1.13). Shear stresses are the highest near the supports, so there is typically an increased number of bars there. 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 9.1.13).

The deck and deck reinforcement between the girders are not typically used in the girder load carrying analysis.

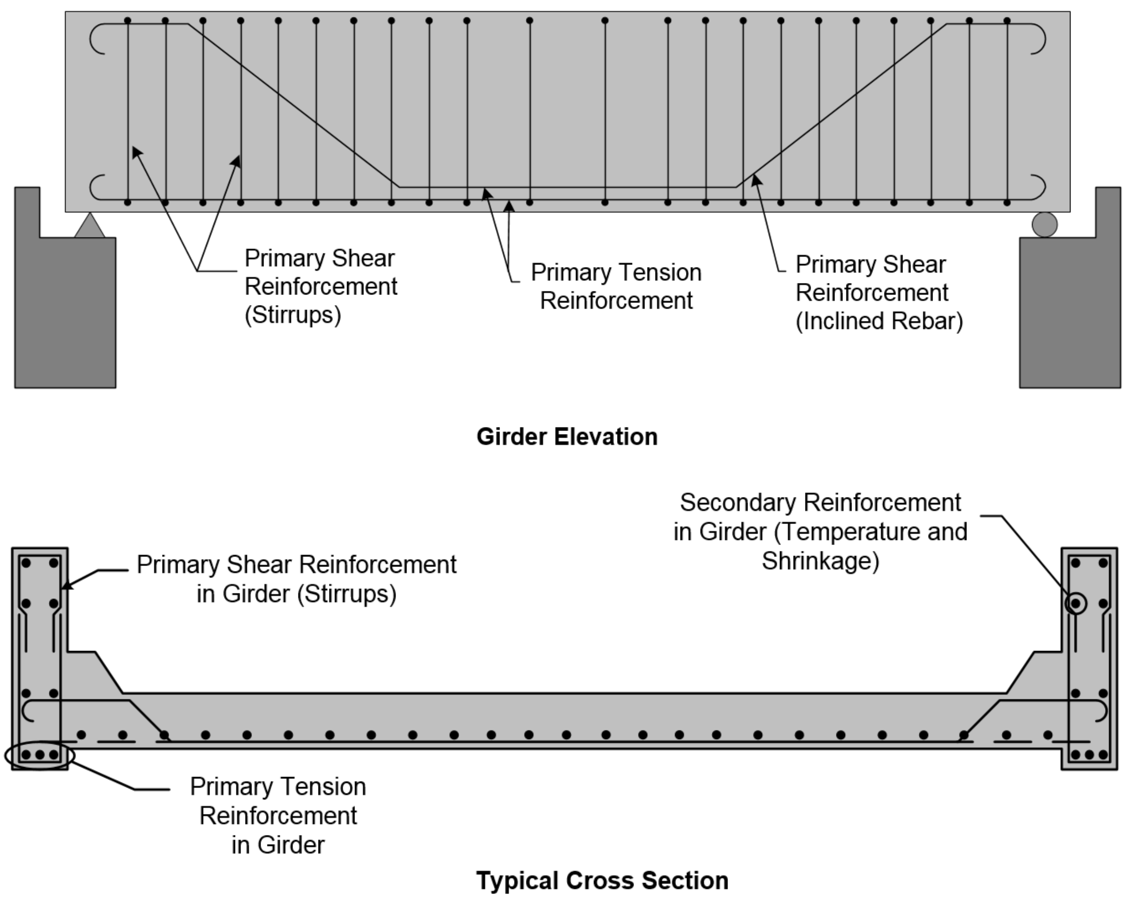


Figure 9.1.13 Steel Reinforcement in a Simple Span Concrete Through Girder

9.1.5 Channel Beams

Design Characteristics

In appearance, the channel beam bridge resembles the tee beam bridge because the stems of the adjacent channel beams extend down to form a stem. The channel beam can be precast or cast-in-place.

The cast-in-place channel beam structures with inverted U-shape beam forms are sometimes referred as “pan bridges” (see Figure 9.1.14).

Figure 9.1.14 U-shaped Channel Beam Bridge

Precast channel beams are placed adjacent to each other and are usually found on spans up to 50 ft. Channel beams consist of a top flange (deck) cast monolithically with two stems, or legs, spaced 3 to 4 ft apart (see Figure 9.1.15). Precast channel beam stems may be conventionally reinforced or may be prestressed. Stem tie bolts and grouted shear keys (see Figure 9.1.16) are used to achieve monolithic action between precast channel beams.

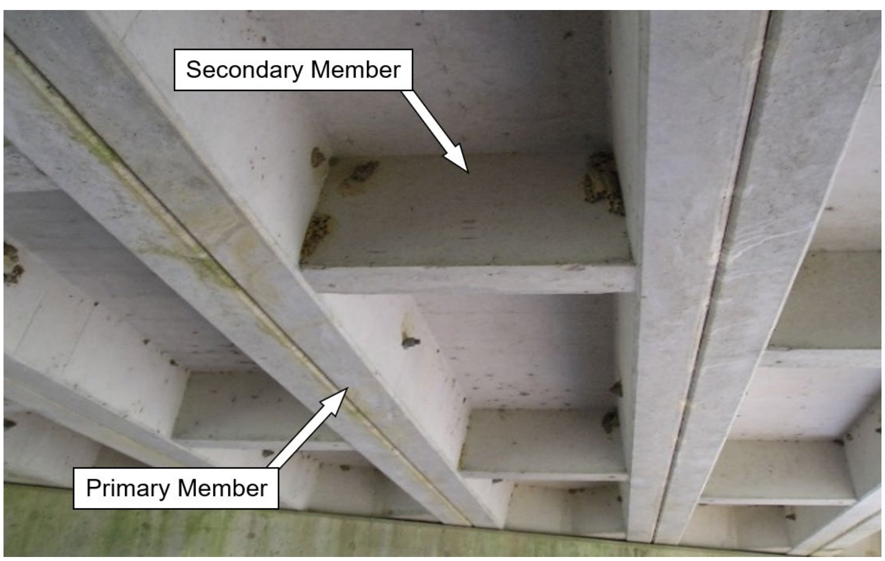


Figure 9.1.15 Channel Beam Bridge

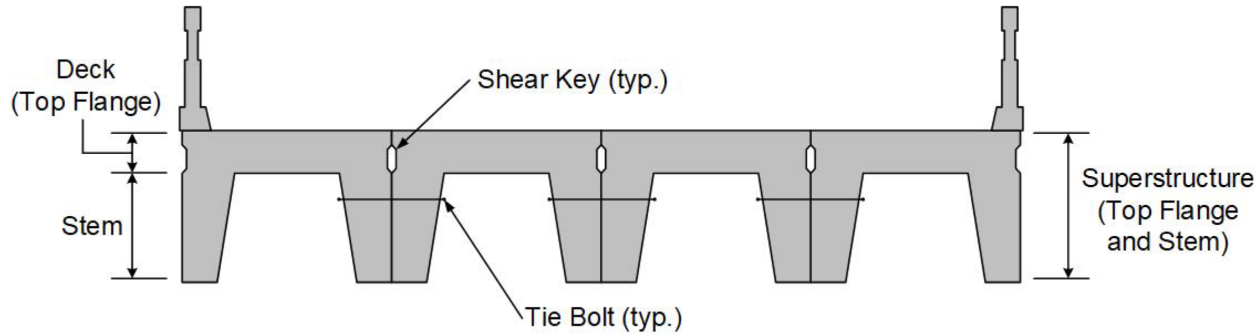


Figure 9.1.16 Cross Section - Precast Channel Beams with Tie Bolts and Shear Keys

Primary Members

The primary superstructure members of channel beam bridges are the channel beams themselves. Both the top flange and the stem portion of the channel beam resist live load.

Steel Reinforcement

The primary reinforcing steel consists of stem tension reinforcement and shear reinforcement, or stirrups. The tension reinforcement is in the bottom of the channel stem and oriented longitudinally (see Figure 9.1.17). The tension steel reinforcement in channel beam stems consist of mild reinforcing bars or prestressing strands. The sides of the stems are reinforced with stirrups. The stirrups are arranged vertically in the sides of the channel stems at various spacings throughout the length and are closer near the beam supports (see Figure 9.1.17). The need for stirrups is greatest near the beam supports where the shear stresses are typically the highest.

The primary reinforcing steel for the top flange (deck) portion of the beam is in the bottom of the deck and is placed transversely, or perpendicular to the channel stems (see Figure 9.1.17). The primary reinforcing steel for the top flange (deck) portion of the beam is similar to a standard concrete deck.

The secondary (temperature and shrinkage) reinforcing steel is oriented longitudinally in the sides of the channel stems and longitudinally in the top flange (deck). The secondary reinforcing steel for the top flange (deck) portion of the beam is similar to a standard concrete deck being supported by longitudinal members (see Figure 9.1.17).

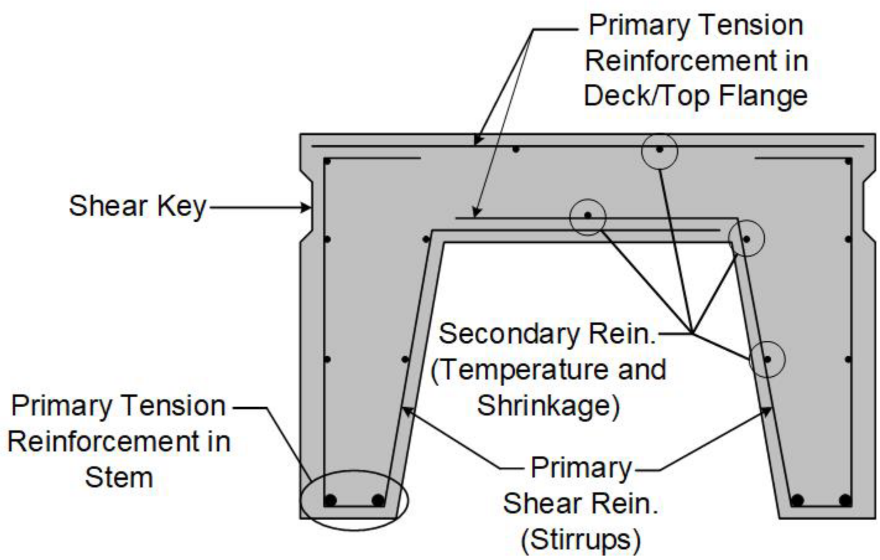


Figure 9.1.17 Steel Reinforcement in a Precast Channel Beam

9.1.6 Concrete Arches

Design Characteristics

A true arch has an elliptical shape and functions in a state of axial compression. It can be thought of as a long, curved column. This makes the true arch a desired form for the use of concrete due to concrete's high compressive strength. However, 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 moment, shear, and axial compression.

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 that are transferred from the deck through spandrel bents or columns (see Figure 9.1.18). The area between the deck and where the arch is supported is considered the superstructure for this bridge type. This type of arch is generally used for 200 ft and longer spans.

Figure 9.1.18 Open Spandrel Arch Bridge

Closed Spandrel Arch

Closed spandrel arches are deck arches since the roadway is above the arch. The spandrel area (the area between the arch and the roadway) is occupied by fill material retained by vertical spandrel walls. The arch member is called a ring or barrel that supports the spandrel walls (see Figure 9.1.19). Portions of the bridge between the spring line and the non-integral deck are considered to be the superstructure.

Closed spandrel arches receive traffic loads through the fill material. This type of arch can be efficient in short span applications.

Figure 9.1.19 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.

Through Arch

A concrete through arch is constructed having the crown of the arch above the deck. Hangers or cables suspend the deck from the arch (see Figure 9.1.20). Concrete through arches are very rare. Superstructure members are constructed above the bearings. Refer to Chapter 13 for Bridge Bearings.

Figure 9.1.20 Concrete Through Arch Bridge

Precast Arch

Precast concrete arches can be integral or segmental. The integral arches typically have an elliptical barrel with vertical integral sides. Segmental arches are oval or elliptical and can have several hinges along the arch. 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 perpendicular to the arch.

Large segmental precast arches that are post-tensioned may span great distances. For this type of design, the deck and supporting members bear on the top or crown of the arch (see Figure 9.1.21). 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.

High quality control can be obtained for precast arches. Sections are precast in a casting yard which allows manufacturers to properly monitor the reinforcement placement, concrete placement, and the curing process. Reinforcement clearances and placement are typically better controlled in a casting yard.

Figure 9.1.21 Precast Post-tensioned Concrete Arch without Spandrel Columns

This type of arch is less common and therefore the coding of *SNBI* Span Type, Item B.SP.06 is not straightforward. FHWA preference would be to code a precast arch as an open spandrel arch (A02) for the Span Type.

Primary 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 members of the superstructure and the following members supported by the arch are also considered primary superstructure members (see Figure 9.1.22):

- 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, when present.
- Floor system - a slab or tee beam arrangement supported by the spandrel bent caps.

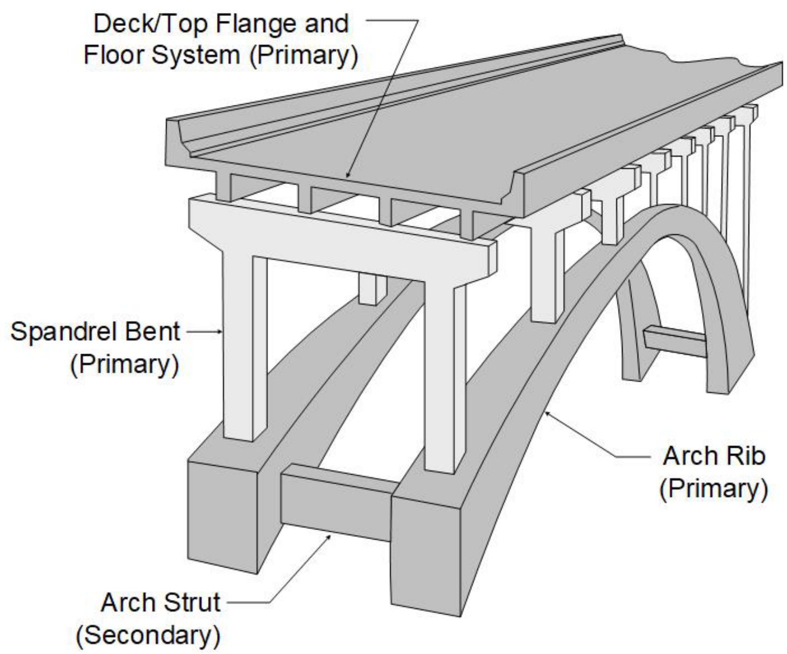


Figure 9.1.22 Open Spandrel Concrete Arch

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 members. The arch itself is the primary load-carrying member of the superstructure. Closed spandrel arches do not generally have a deck or slab.

Steel Reinforcement

Steel reinforcement for the arches, arch rings, spandrel columns and spandrel walls are similar to concrete columns and retaining walls (refer to Chapter 14). Steel reinforcement for spandrel bent caps, spandrel beams and floor systems are similar to concrete beams detailed earlier in this chapter.

9.1.7 Concrete Rigid Frames

Design Characteristics

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 cast-in-place or precast units.

The rigid frame bridge can be single span (see Figure 9.1.23) or multi-span (see Figure 9.1.24). Single span frame bridges generally utilize haunched slabs to span up to 50 ft.

Figure 9.1.23 Single-span Rectangular Concrete Rigid Frame Bridge

Multi-span frame bridges are typically used for spans over 50 ft 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 9.1.24). Due to frame action between the horizontal members and the vertical or inclined members, multi-span frames typically are considered continuous.

Figure 9.1.24 Three Span Concrete K-frame Bridge

Rigid frame structures are utilized both at grade and under fill, such as in concrete frame culverts. Refer to Chapter 4 for more information on culvert characteristics.

Primary Members

For single span frames, the primary members are considered to be the slab portion and the legs of the frame (see Figure 9.1.25). The slab portion is considered the superstructure while the legs are considered the substructure abutments.

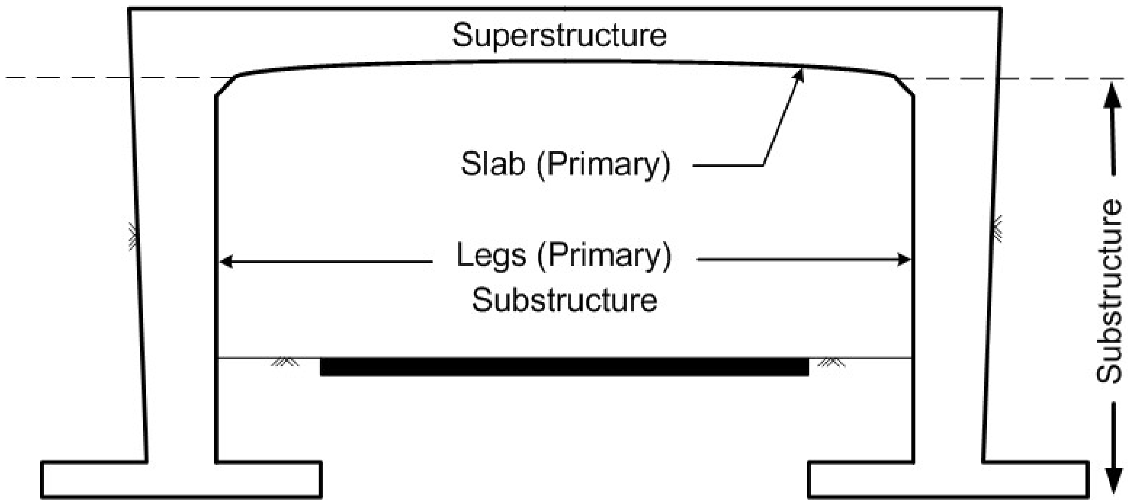


Figure 9.1.25 Elevation of a Single Span Frame

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 that is supported by the frame legs and abutments) (see Figure 9.1.26). The frame beams or slabs and frame legs are considered the superstructure while the abutments are considered the substructure.

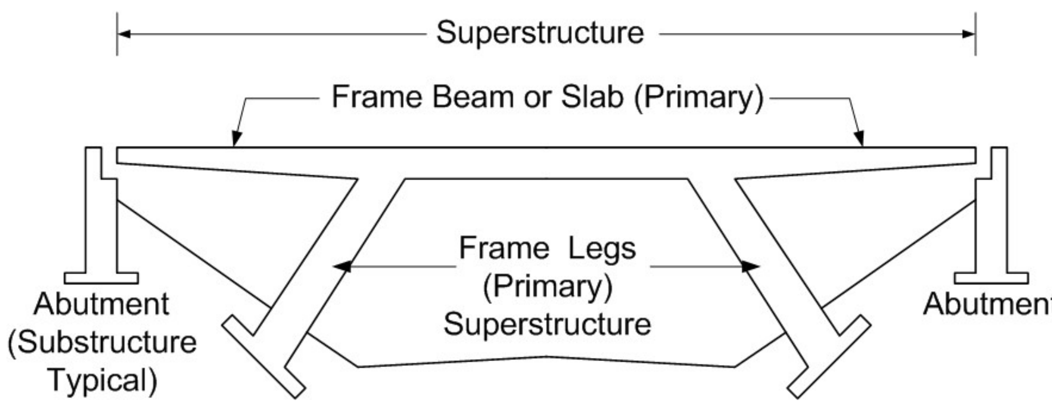


Figure 9.1.26 Elevation of a K-frame

Steel Reinforcement

Rigid frame structures develop positive and negative moment throughout due to the interaction of the frame legs and frame beams (see Figure 9.1.27). In slab or beam frames, the primary reinforcement is used to resist tension and possibly shear.

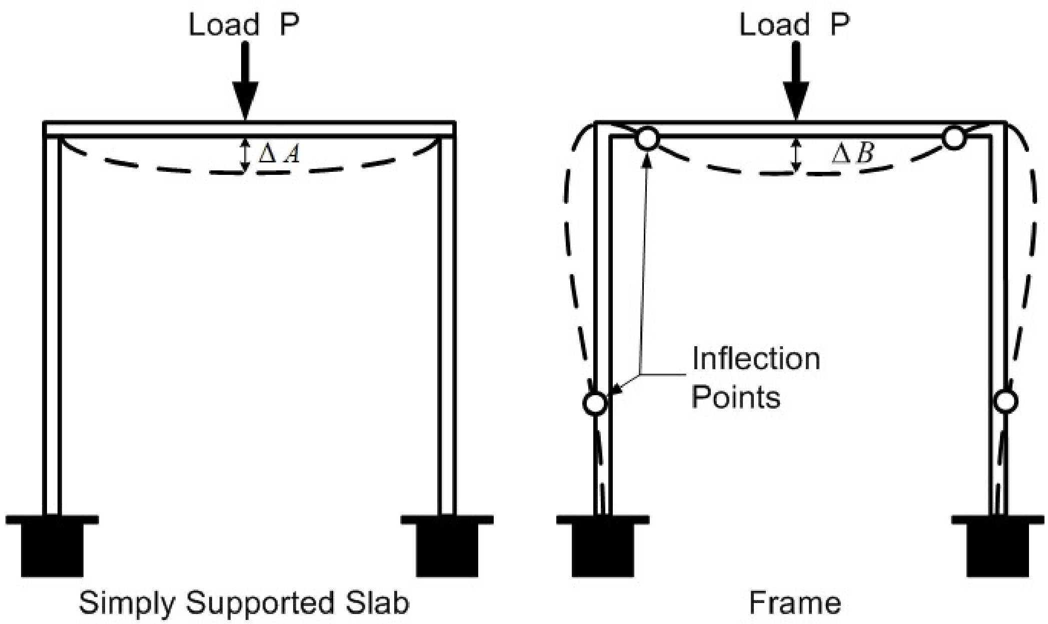


Figure 9.1.27 Deflected Simply Supported Slab versus Deflected Frame Shape

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 9.1.28).

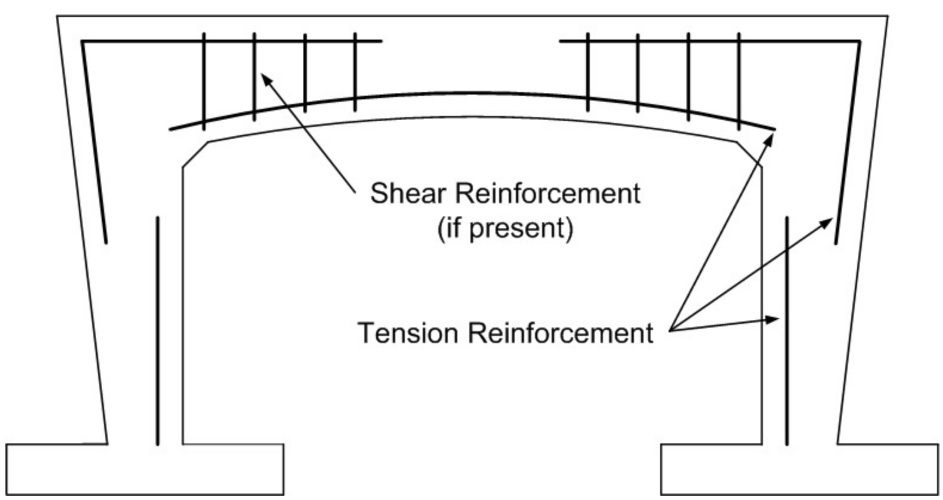


Figure 9.1.28 Primary Reinforcement in a Single Span Frame

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. If shear reinforcement is used to prevent shear cracking, stirrups should be provided (see Figure 9.1.29).

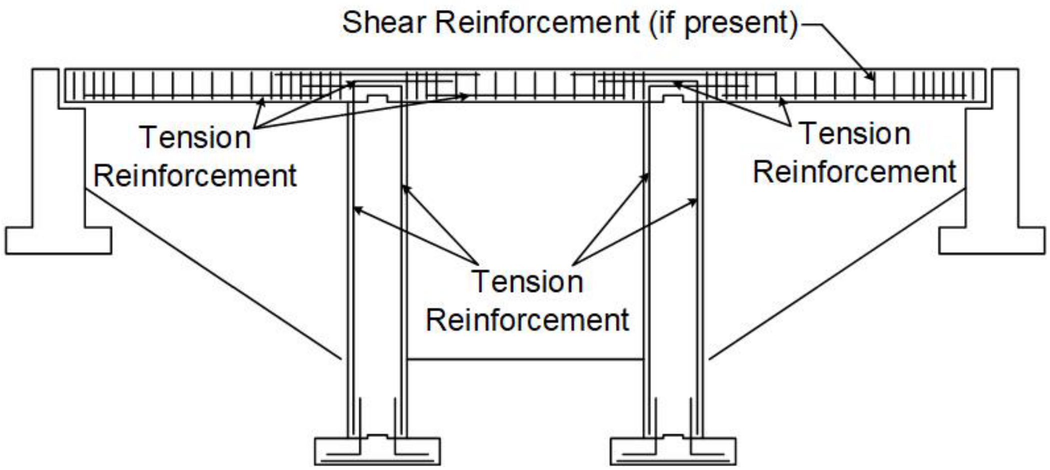


Figure 9.1.29 Primary Reinforcement in a Multi-span Slab or Beam Frame

The primary reinforcement in the frame beam portion is longitudinal tension and shear stirrup steel, similar to continuous beam reinforcement.

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 9.1.30).

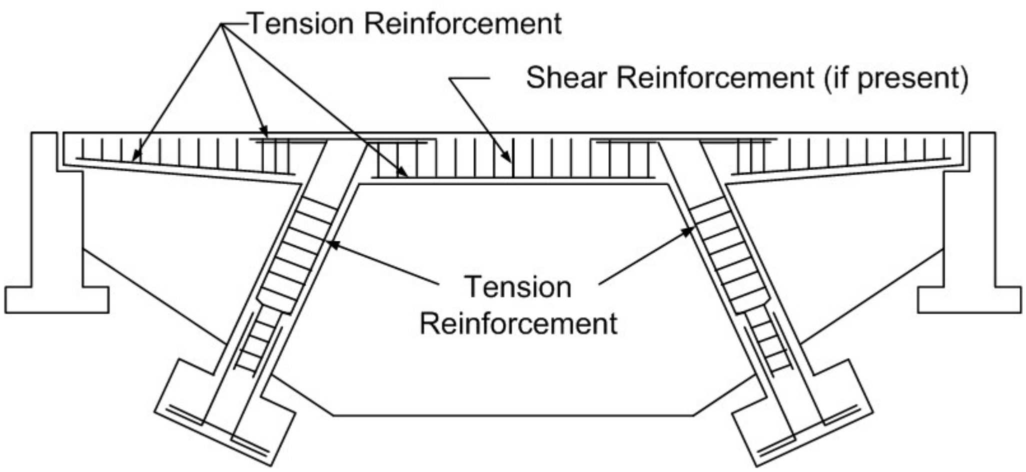


Figure 9.1.30 Primary Reinforcement in a K-frame

Sometimes, the frame legs resist compressive loads. Refer to Chapter 14 for a discussion of compression steel and column ties.

Secondary reinforcement for temperature and shrinkage is distributed similar to that of a slab or tee beam or box beams.

9.1.8 Precast and Prestressed Solid or Voided Slab Beams

Design Characteristics

Precast and prestressed slabs have become more common since the 1950s. This type of design acts as a deck and superstructure combined (see Figure 9.1.31). Individual members are placed side by side and connected so they act as one slab. When vertical clearances are lacking, this type of design is often constructed, due to the slab's relatively shallow depth. Wearing surfaces are generally applied to the top of precast and prestressed slab beams and are concrete or bituminous overlays. Refer to Chapter 8 for more information on wearing surfaces.

Figure 9.1.31 Typical Prestressed Slab Beam Bridge

Although precast and prestressed slab beams are different from concrete decks, inspection procedures are similar. Refer to Chapter 8 for additional information about concrete decks.

The precast voided slab beam 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 provide economy of material and reduce dead load (see Figure 9.1.32).

Precast slab beam bridges with very short spans may not contain voids. Precast slab beams also contain drain holes, which are strategically placed in the bottom of the member to allow accumulated moisture to escape.

Precast slab beams units are typically practical for spans up to 60 ft. The slab beams 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 concrete slab beams typically are precast with 4,000 to 8,000 psi concrete and reinforced with up to 270 ksi pre-or post-tensioned steel tendons. Conventional reinforced concrete slab beams are typically precast with 3,000 to 4,000 psi concrete and reinforcement of 40 or 60 ksi mild steel reinforcing bars.

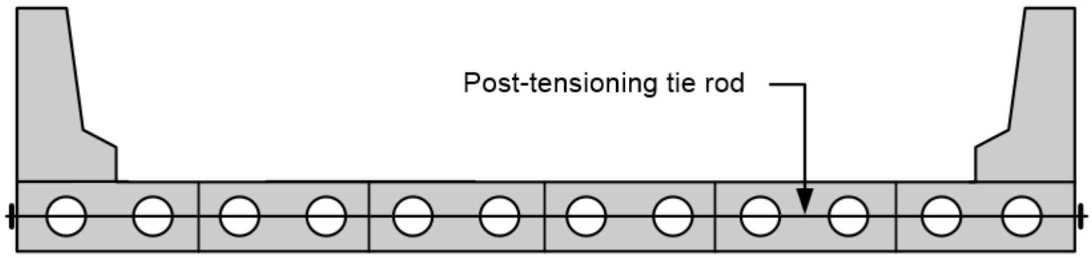


Figure 9.1.32 Cross Section - Precast Voided Slab Beam Bridge

Monolithic Behavior

Adjacent slab beams may be post-tensioned together with tie rods (see Figure 9.1.32) or post-tensioning strands having a tensile capacity of 150 ksi or 270 ksi respectively and are grouted at the shear keys. Together these enable the slab beams to act monolithically.

Identifying Voided Slab Beams

Physical dimensions alone typically are not enough to distinguish a slab beam from a box beam (refer to Section 9.1.9 and 9.1.12 for more information on box beams). Design or construction plans should 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 beam has two or three circular voids through it and may be as deep as 26 inches. It is also possible to find precast solid slab beams.

Primary Members

The primary members of a precast and prestressed voided slab beam bridge are the individual slab beams. The slab beams represent the deck and superstructure components, and the top surface is commonly protected by an asphalt or concrete overlay.

Steel Reinforcement

The primary reinforcement consists of longitudinal tension steel and shear reinforcement or stirrups.

Prestressing strands placed near the bottom of the slab beams make up the main tension steel. Depending on the age of the structure, the strands are fabricated in sizes of 1/4, 3/8, 7/16, or 1/2 inch in diameter. Prestressing strands are typically spaced 2 inches on center (see Figure 9.1.33) and have a tensile strength of 240 ksi or 270 ksi. Tension steel for conventionally reinforced precast slab beams 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 placed throughout the slab beam at various spacings as described previously. (see Figure 9.1.33).

Secondary reinforcement is provided to control temperature and shrinkage cracking. This reinforcement is placed longitudinally in the beam, typically at the top of the slab units, and holds the stirrups in place during fabrication. (see Figure 9.1.33) Transverse post-tensioning strands or bars through the diaphragms helps maintain monolithic action between the adjacent slab beams.

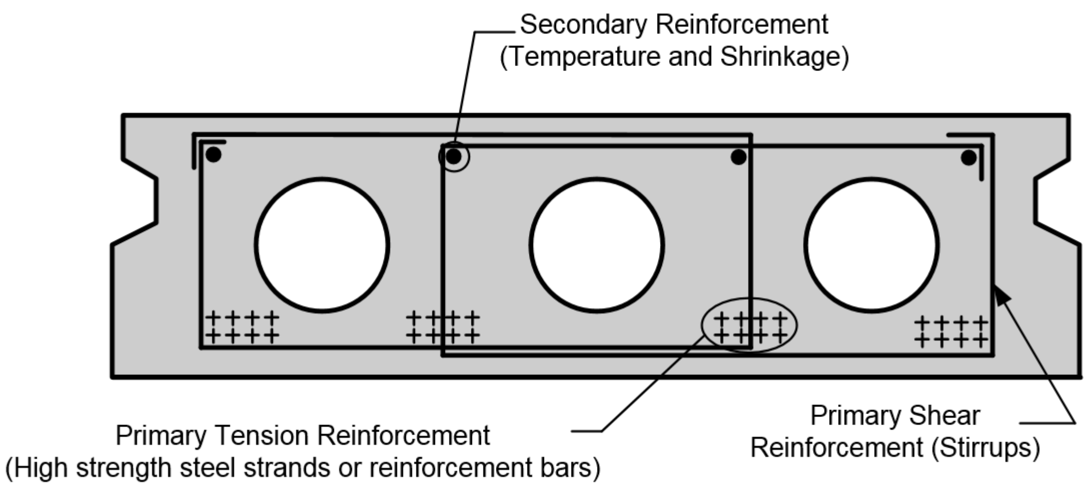


Figure 9.1.33 Steel Reinforcement in a Precast/Prestressed Slab Beam

9.1.9 Precast Prestressed Box Beams

Design Characteristics

Prestressed box beams have been used since the early 1950s (see Figure 9.1.34). These precast prestressed members may provide advantages from a construction and an economical standpoint by increasing strength while decreasing the dead load and providing a shallower beam that can increase vertical clearance for a roadway below.

Figure 9.1.34 Typical Box Beam Bridge

Prestressed box beams are constructed with a rectangular cross section with a single rectangular void inside. Many prestressed box beams constructed in the 1950s 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 but could also be present in the corners of the top flange (see Figure 9.1.35). Relevant plans and shop drawings should be reviewed for individual beam strand layout.

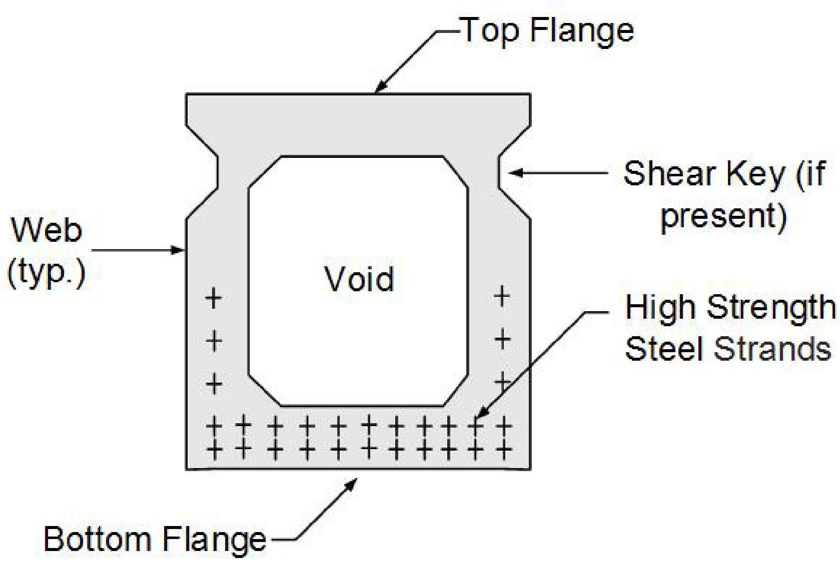


Figure 9.1.35 Cross Section - Prestressed Box Beam

The typical span length for prestressed concrete box beams ranges from 20 to 90 ft depending on the beam size and their spacing. Web wall thickness is typically 5 inches but can range from 3 to 6 inches. Prestressed box beams are typically 36 or 48 inches wide. The depth of a box beam generally ranges from 27 to 42 inches. Prestressed box beams are typically designed with a maximum depth of 42 inches, but may have depths up to 60 inches. Shallow depths make box beams viable solutions for field conditions where shallow vertical clearances exist.

Prestressed box beams can be simple or continuous spans. Similar to prestressed concrete I-beams and bulb tee beams, prestressed box beams can be considered composite or non-composite (refer to Section 9.1.11 for more information on prestressed concrete I-beams and bulb tee beams). Stirrup extension can be similar to prestressed concrete I-beams or bulb tee beams (see Figure 9.1.39). The voided box beam reduces dead load while still providing flanges to resist the design moments and webs to resist the design shears.

Precast members are cast and cured in a quality-controlled casting yard. Because box beams are precast, the construction erection process takes less time. When construction is properly planned, using precast members allows the structure to be erected with less traffic disruption than typical cast-in-place concrete construction.

There are two applications of prestressed box beams: spread and adjacent (see Figure 9.1.36).

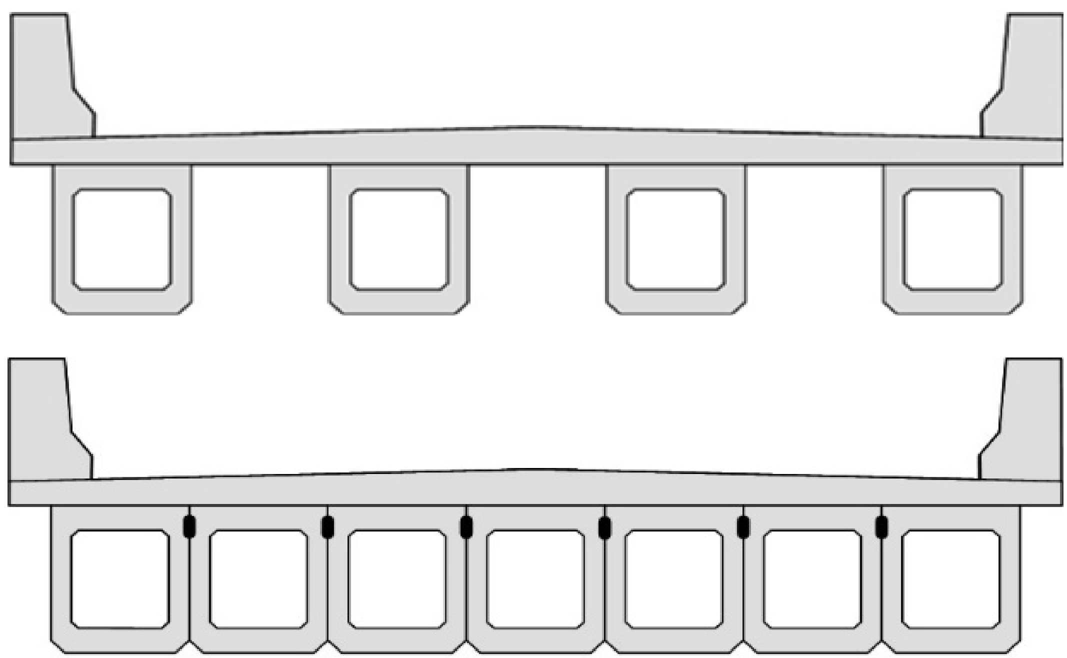


Figure 9.1.36 Cross Section - Prestressed Spread and Adjacent Box Beams

On a spread box beam bridge, the box beams are usually spaced from 2 to 6 ft apart and typically use a cast-in-place concrete deck for composite action. This application is practical for span lengths from 25 to 85 ft. Stay-in-place forms (refer to Chapter 8) or removable formwork is used between the box beams to provide support for the concrete deck before curing.

On an adjacent box beam bridge, the 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. The practical span lengths range from 20 to 130 ft.

In modern longer span applications, the deck is typically cast-in-place concrete and composite action with the box beam is achieved after the deck concrete hardens. Nonstructural asphalt overlays are sometimes applied and do not provide composite action. Sometimes a waterproofing membrane is applied before the overlay placement.

Like precast slab beams, adjacent box beams can be post-tensioned transversely. Transverse post-tensioning combined with grouted shear keys provides for monolithic action and, in theory, allow the beams to act as one unit (see Figure 9.1.37). A common issue in adjacent box beams is that the post-tensioning corrodes or disengages, resulting in the loss of the monolithic behavior.

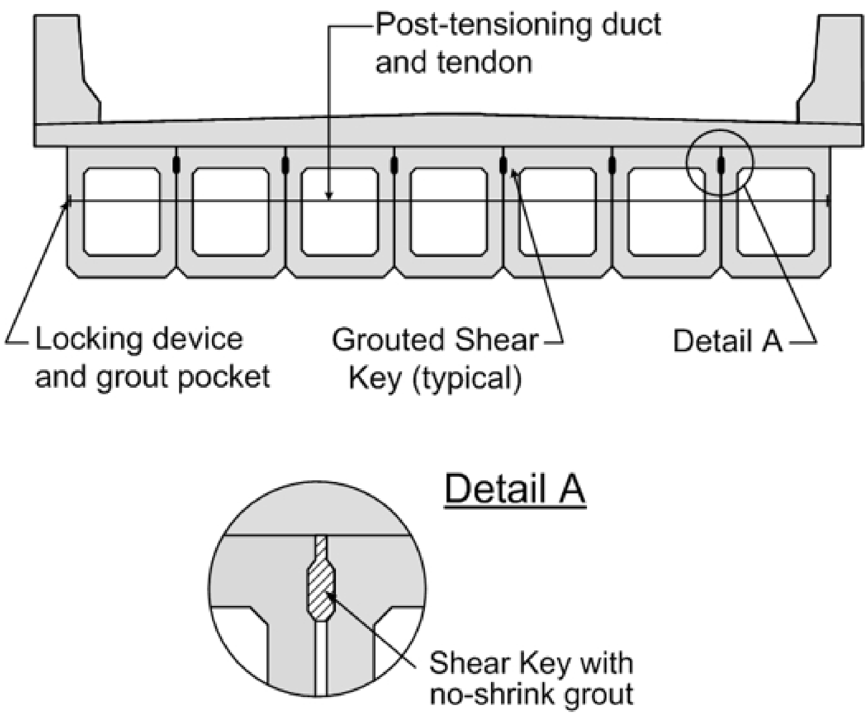


Figure 9.1.37 Transverse Post-Tensioning and Shear Keys for Adjacent Box Beams

Most modern box beams have drain holes that are installed in the bottom flange during fabrication to allow any moisture in the void to escape, although they still become clogged over time.

Primary Members

The primary superstructure members of box beam bridges are the prestressed concrete box beams. An internal diaphragm is considered part of the prestressed box beams and not a secondary member (see Figure 9.1.38).

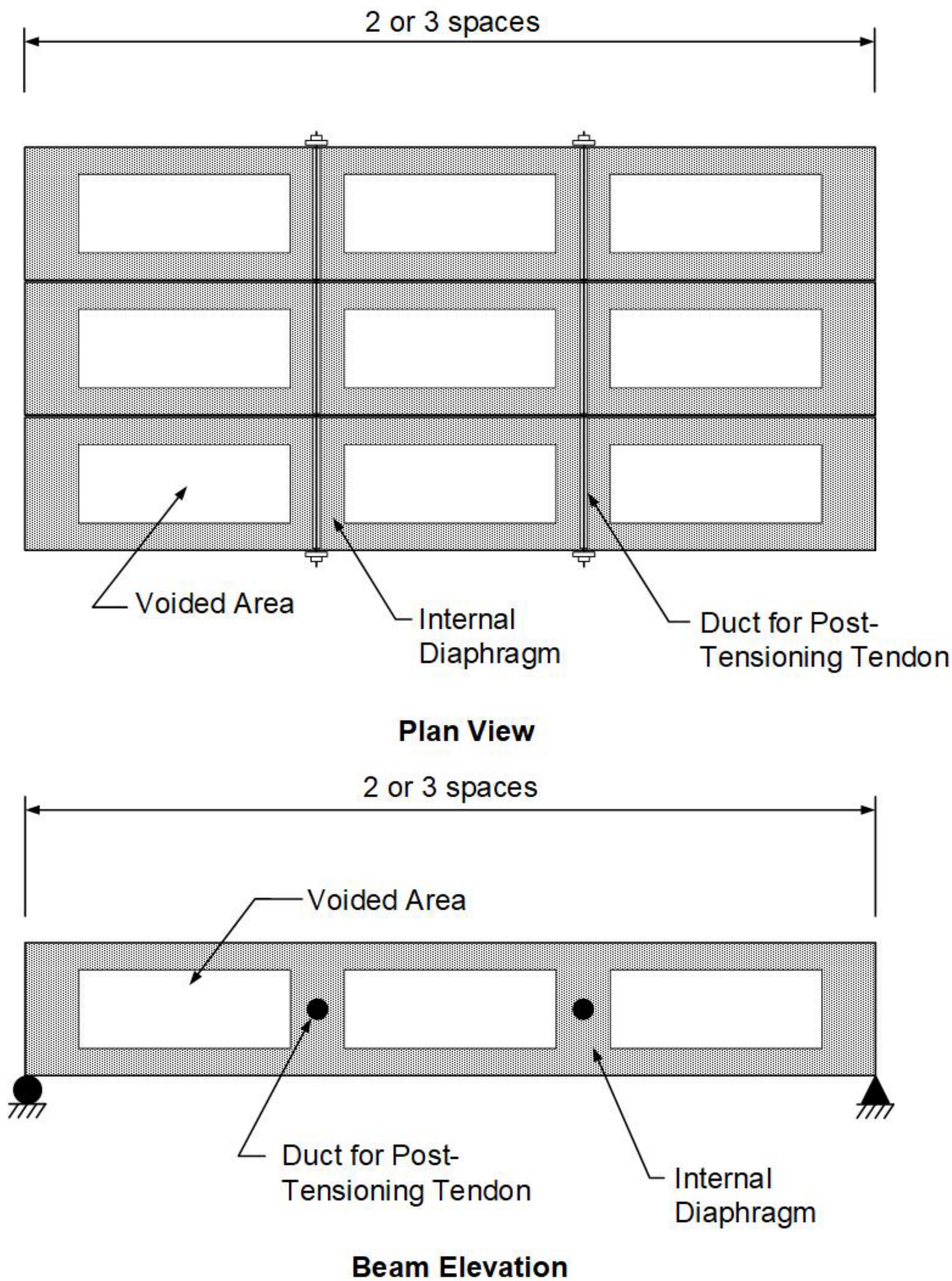


Figure 9.1.38 Schematic of Internal Diaphragms and Post-Tensioning

Steel Reinforcement

Primary reinforcement consists of main tension steel and shear reinforcement or stirrups. Main tension steel consists of high strength prestressing strands placed in the flange and lower web of the box beam.

Depending on the age of the structure, the strand size will likely be 1/4, 3/8, 7/16, ½, or 0.6 inch in diameter and spacing is typically 2 inches apart (see Figure 9.1.39).

Mild steel stirrups are placed vertically in the web at various spacings as necessary to provide adequate shear reinforcement (see Figure 9.1.39). Mild steel stirrups are more closely spaced near beam ends and typically 60 ksi. Older designs may utilize 40 ksi reinforcement.

The current practice is to install the prestressing strands inside the shear stirrups (see Figure 9.1.39). Older designs (1960s) called for the stirrups to be placed between the prestressing strand rows.

Transverse post-tensioning through the diaphragms helps maintain monolithic action between the adjacent box beams by creating a clamping action for engagement of the shear keys. Temperature and shrinkage reinforcement consisting of mild steel is placed longitudinally in the beam webs and top flange.

Composite strands made of carbon fiber or glass fiber are an alternative to steel but are not commonly used. Refer to Chapter 7 for more information on composite strands.

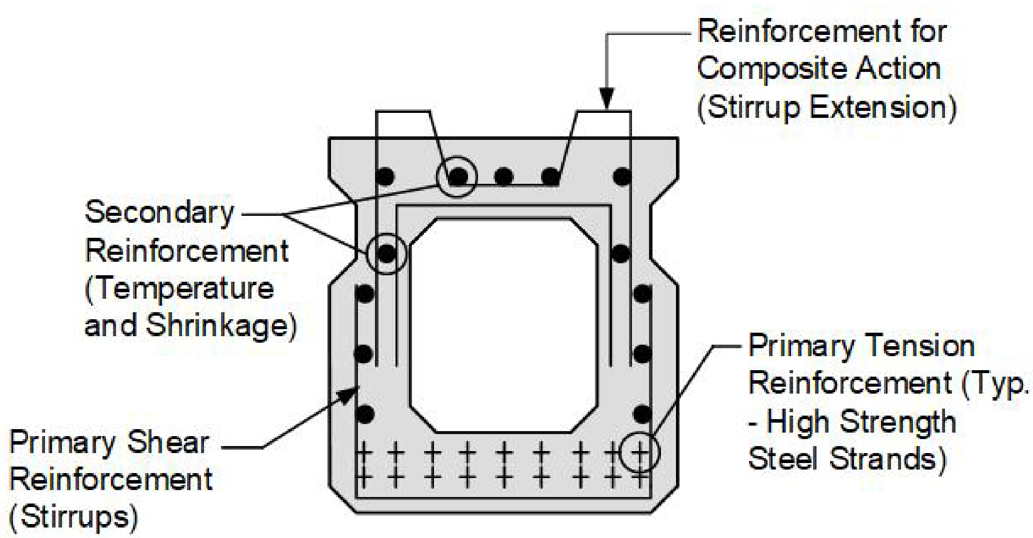


Figure 9.1.39 Steel Reinforcement of a Prestressed Box Beam

9.1.10 Precast Prestressed Double Tee Beams

Design Characteristics

A prestressed double tee beam, as the name implies, resembles two adjacent capital letter T's (see Figure 9.1.40). The horizontal section is called the top flange, and the two vertical leg sections are called the webs or stems. The Northeast Extreme Tee (NEXT) beam is the regional standard for double tee beams unless a state agency has adopted their own specification. This type of bridge beam is mostly used in short spans or in situations where short, obsolete bridges are to be replaced.

Figure 9.1.40 Precast Double Tee Beam Section

Prestressed concrete double tee beams have a monolithic top flange and stem design that allows the top flange to act integrally with the stems to form a superstructure member. 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 9.1.41).

This type of construction was originally used for buildings and is quite common in parking garages. It has been adapted for use in highway structures with double and triple stems.

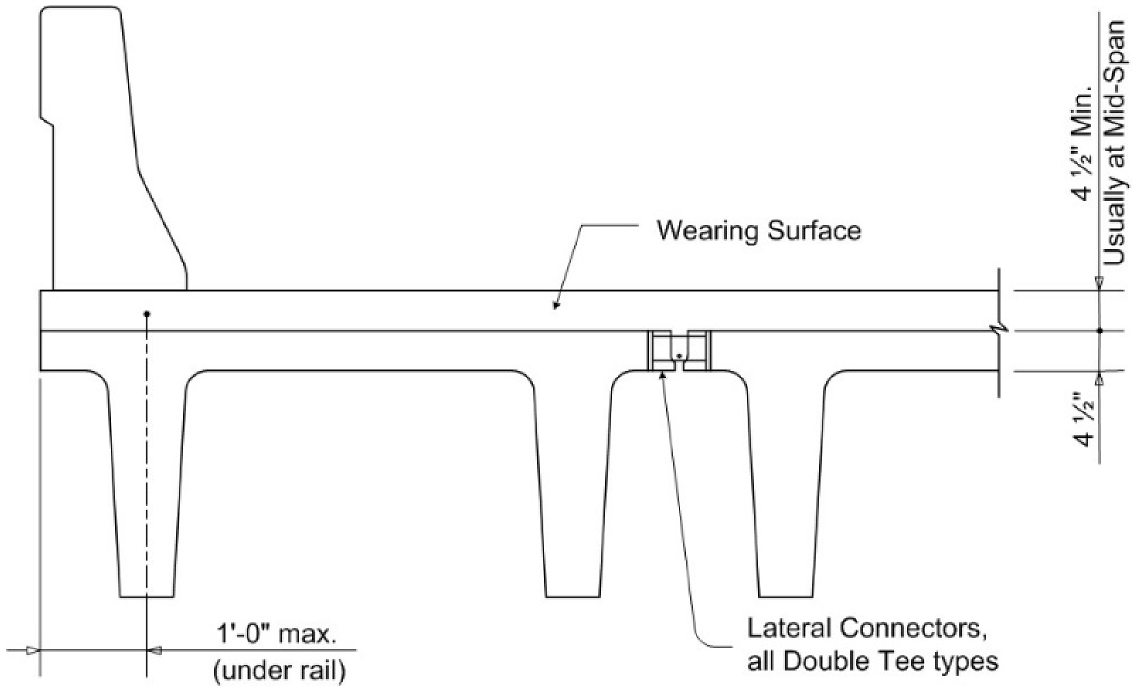


Figure 9.1.41 Prestressed Double Tee Beam with Lateral Connectors

Prestressed double tees have a typical stem depth of 12 to 34 inches. The average flange width is 8 to 10 ft, with a typical span length of approximately 25 to 55 ft. Prestressed double tees can be used in spans approximately 80 ft long with stem depths up to 5 ft and flange widths up to 12 ft. 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 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 9.1.42). This occurs so that the beam end can sit flush on the bearing seat.

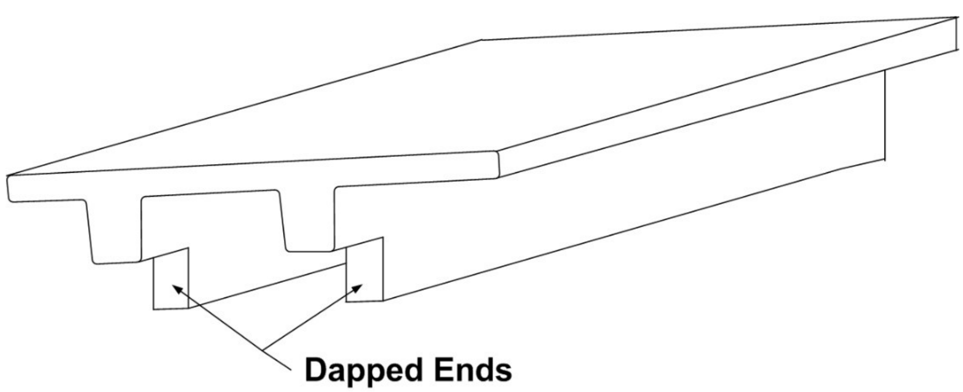


Figure 9.1.42 Dapped End of a Prestressed Double Tee Beam

The top flange of the prestressed double tees can act as the integral wearing surface or be overlaid. In NEXT Beams, for example, the difference is distinguished in the name of the section. A D Beam (Deck Beam) has an integral full-depth flange that acts as the structural bridge deck, whereas an F Beam (Flange Beam) has a partial-depth flange. The thin flange serves as the formwork for a conventional reinforced concrete deck (see Figure 9.1.43). Bituminous, or asphalt, can also be applied as the wearing surface.

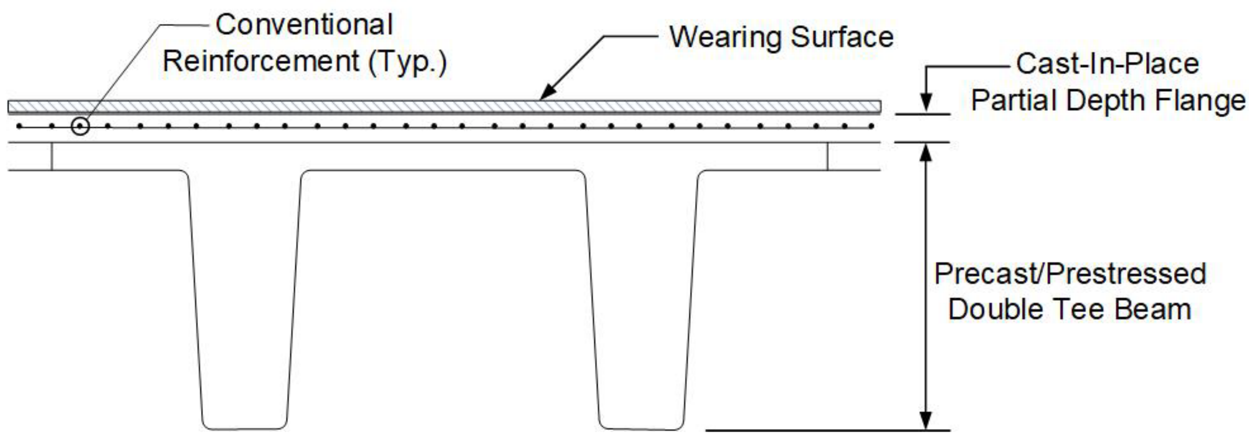


Figure 9.1.43 NEXT Beam

Primary Members

The primary members of a prestressed double tee beam superstructure, like precast tee beams, are the stems and the top flange.

Steel Reinforcement

The primary tension and shear steel reinforcement consists of prestressing strands and reinforcing bars, sometimes referred to as mild reinforcement (see Figure 9.1.44). The prestressing strands are placed longitudinally in each stem. When the double tees are to be continuous over two or more spans, ducts 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. The primary reinforcement for the top flange section of a prestressed double tee beam follows the reinforcement pattern of a typical concrete deck (refer to Chapter 8).

In some wider applications, the top 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. They can be post-tensioned together in a similar fashion as slab beam bridges (see Figure 9.1.32).

The secondary, or temperature and shrinkage, reinforcement is placed longitudinally on each side of each stem and top flange. In some newer designs, welded-wire-fabric is used as the secondary and shear reinforcement in the stems. The vertical wires in the welded-wire-fabric act as the shear reinforcement and the longitudinal wires perform as the secondary reinforcement.

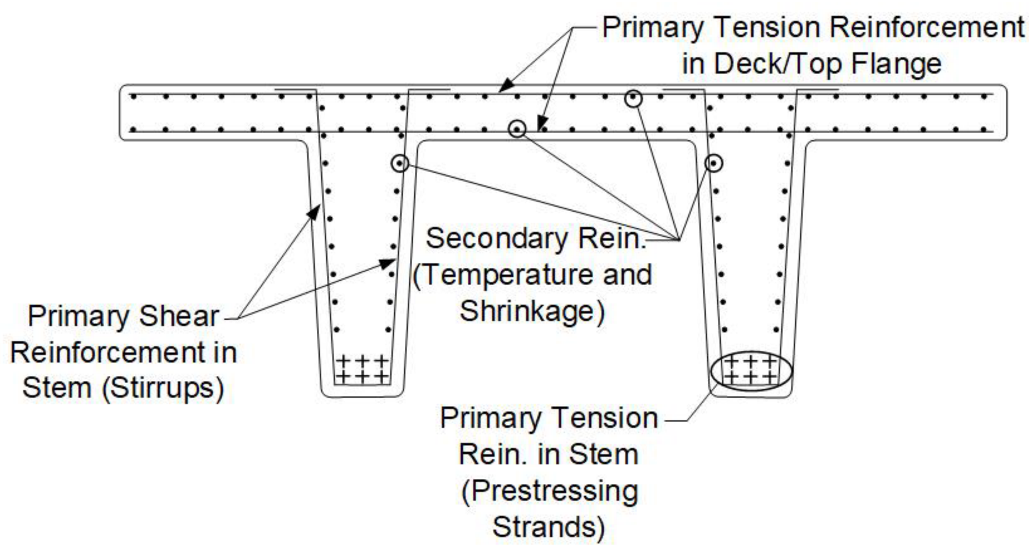


Figure 9.1.44 Steel Reinforcement in a Prestressed Double Tee Beam

9.1.11 Precast Prestressed I-Beams, Bulb Tee Beams, Deck Bulb Tee Beams

Design Characteristics

Prestressed I-beams and deck bulb tee beams have been used since the 1950s. These beam types are often used because of their material saving shapes.

With prestressed I-beams and bulb tees, most of the concrete mass is in the top and bottom flanges and away from the neutral axis of the beam. The I- or T- shape provides a designer with enough cross-section to place the desired amount of reinforcement while reducing the amount of concrete needed (see Figure 9.1.45).

Figure 9.1.45 Prestressed I-beam Bridge

Common prestressed concrete I-beam shapes used by State highway agencies are the AASHTO-PCI-certified precast shapes (see Figure 9.1.46) (AASHTO-PCI). Some highway agencies have also developed variations of the AASHTO-PCI-certified I-beam shapes to accommodate their particular needs.

Prestressed I-beams are generally used in spans ranging from 40 to 200 ft. They are generally the most economical at spans from 60 to 160 ft.

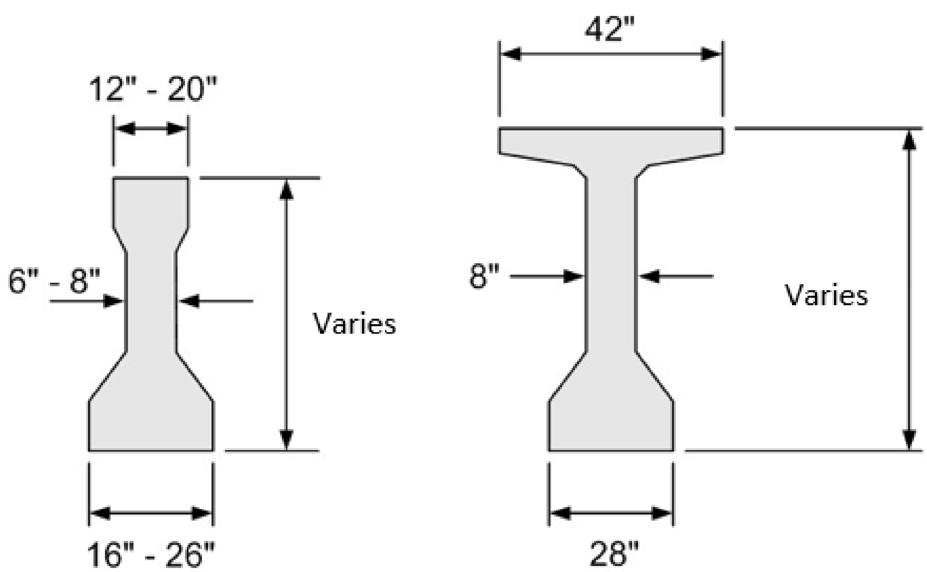


Figure 9.1.46 Cross Section - AASHTO I-beams

Originally developed from the AASHTO-PCI-certified Type V and VI shapes, AASHTO-PCI-certified bulb tee shapes utilize a more efficient cross section with fewer prestressing strands to achieve comparable span lengths between traditional AASHTO-PCI-certified I-beams (see Figure 9.1.47 and Figure 9.1.48). Due to the reduced volume of concrete used in fabrication, bulb tee beams have reduced material costs, lower shipping weights, and increased stability during transportation. Economical span lengths range between 80 and 160 ft. When higher strength concrete is used, maximum span lengths can approach 200 ft.

Bulb tee beams are also suited for span continuity using post-tensioning techniques (presented later in this chapter). As with AASHTO-PCI-certified I-beams, some highway agencies have also developed variations of the AASHTO-PCI-certified bulb tee shapes, including bulb tee girders, variable depth bulb tee beams, and bulb tees with increased web depths and/or top flange widths.

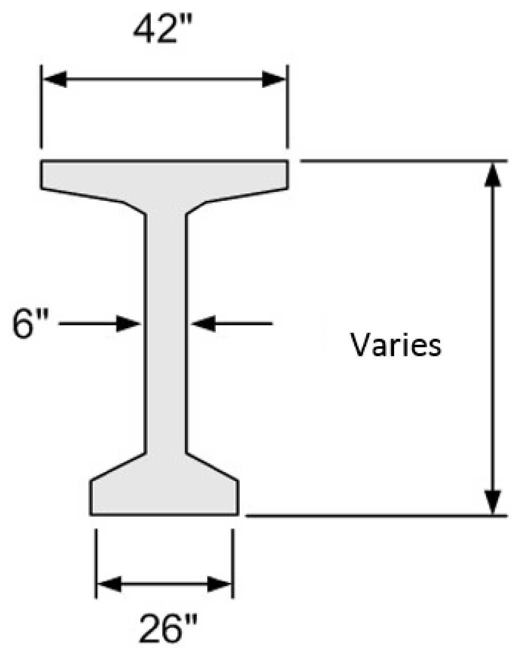


Figure 9.1.47 Cross Section - AASHTO-PCI Beam Bulb Tee

Figure 9.1.48 Placement of an AASHTO-PCI Bulb Tee Beam

Deck bulb tee beams, a variation of AASHTO-PCI-certified bulb tee shapes, were developed to provide a beam cross section that would not need a separately placed concrete deck, but would generally have a flexible or rigid overlay placed on the top flange. Using similar geometry as the AASHTO-PCI-certified bulb tee, with the exception of the top flange width (see Figure 9.1.49), these PCI Deck bulb tee beams may be a viable alternative to adjacent box beam superstructures.

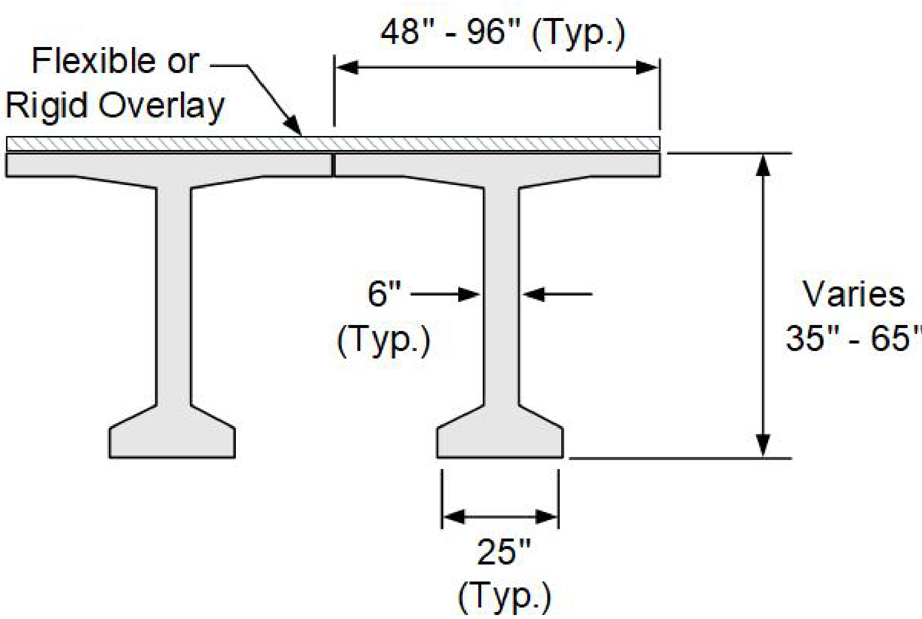


Figure 9.1.49 Cross Section - PCI Deck Bulb Tee Beam

Economical span lengths range between 80 and 160 ft, with maximum span lengths of 200 ft 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 ft. PCI 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 chapter.

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 Figure 9.1.50 and Figure 9.1.51).

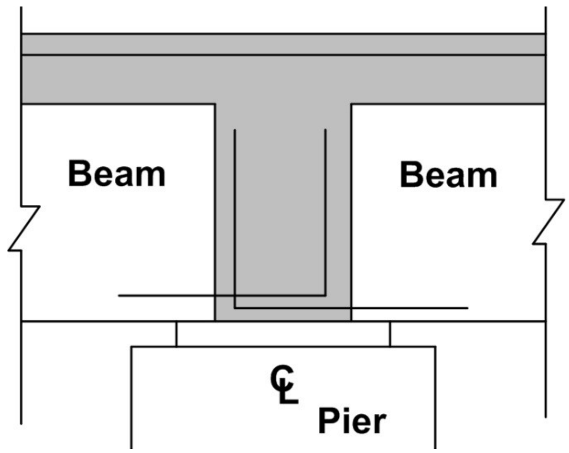


Figure 9.1.50 Continuous Prestressed I-beam Schematic

Figure 9.1.51 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 are stressed for continuity. Cast-in-place concrete diaphragms are framed around the beams at the abutments and piers.

The deck is secured to and can be made composite with the prestressed beam by the use of extended stirrups that are cast into the I-beam or bulb tee beam (see Figure 9.1.52). Some designs may utilize a reinforced concrete deck with the deck bulb tee for composite action. Note that deck bulb tee beams by themselves (including overlays) are not considered to be composite because the top flange (deck), web and bottom flange, or "bulb" are constructed of the same material.

Primary Members

The primary members are the prestressed beams. The beams carry load from the deck to the substructure units.

Steel Reinforcement

Primary reinforcement consists of main tension steel and shear reinforcement or stirrups.

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 fabricated in standard sizes of 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 that are placed in ducts cast into the beam web (see Figure 9.1.52 and Figure 9.1.53).

Mild steel stirrups are vertical in the beam and placed throughout the web at various spacings (see Figure 9.1.52 and Figure 9.1.53). Welded wire fabric also may be used for shear reinforcement in these beams.

Composite strands made from carbon or glass fibers have been used as a substitute for steel prestressing strands to address concerns with steel corrosion, although not common.

Secondary reinforcement includes mild steel temperature and shrinkage reinforcement that is longitudinal in the beam. (see Figure 9.1.52 and Figure 9.1.53). Bulb tee beams also typically contain transverse temperature and shrinkage steel in the top flange.

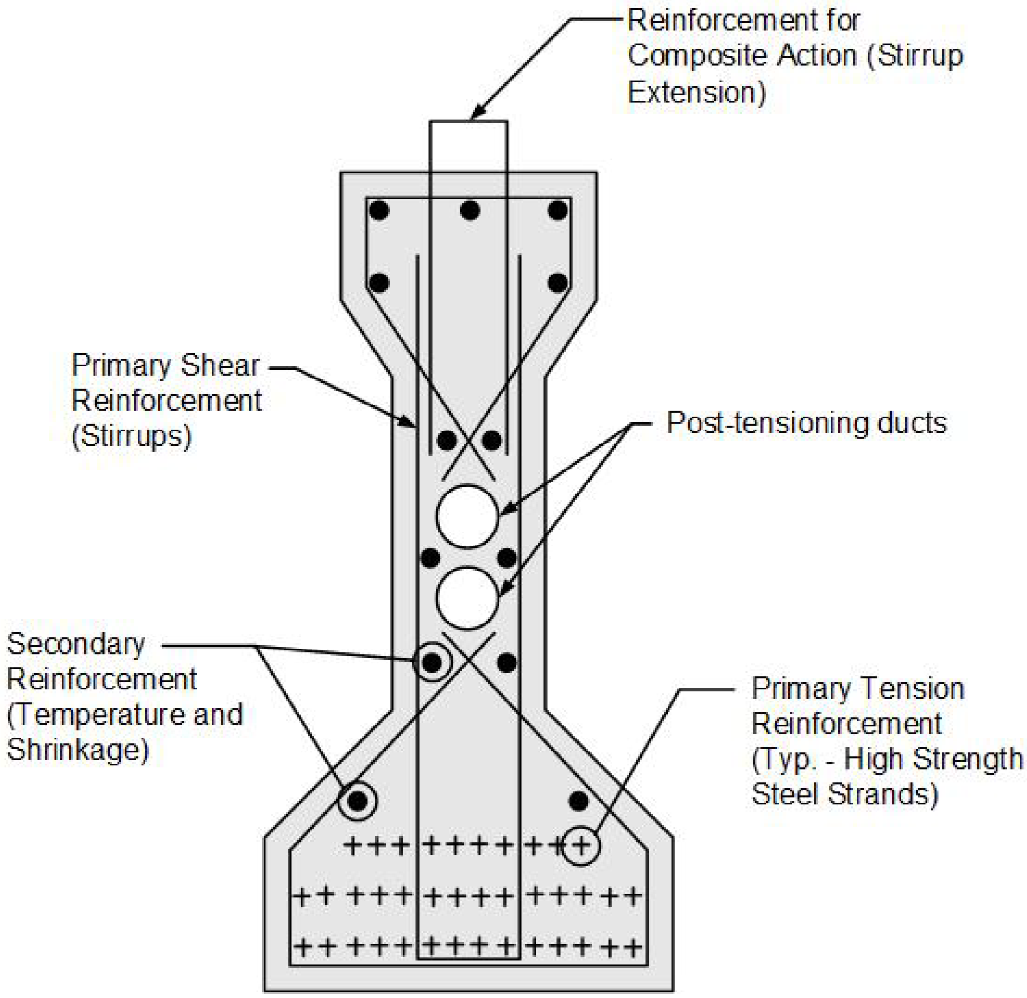


Figure 9.1.52 Steel Reinforcement in a Prestressed I-beam

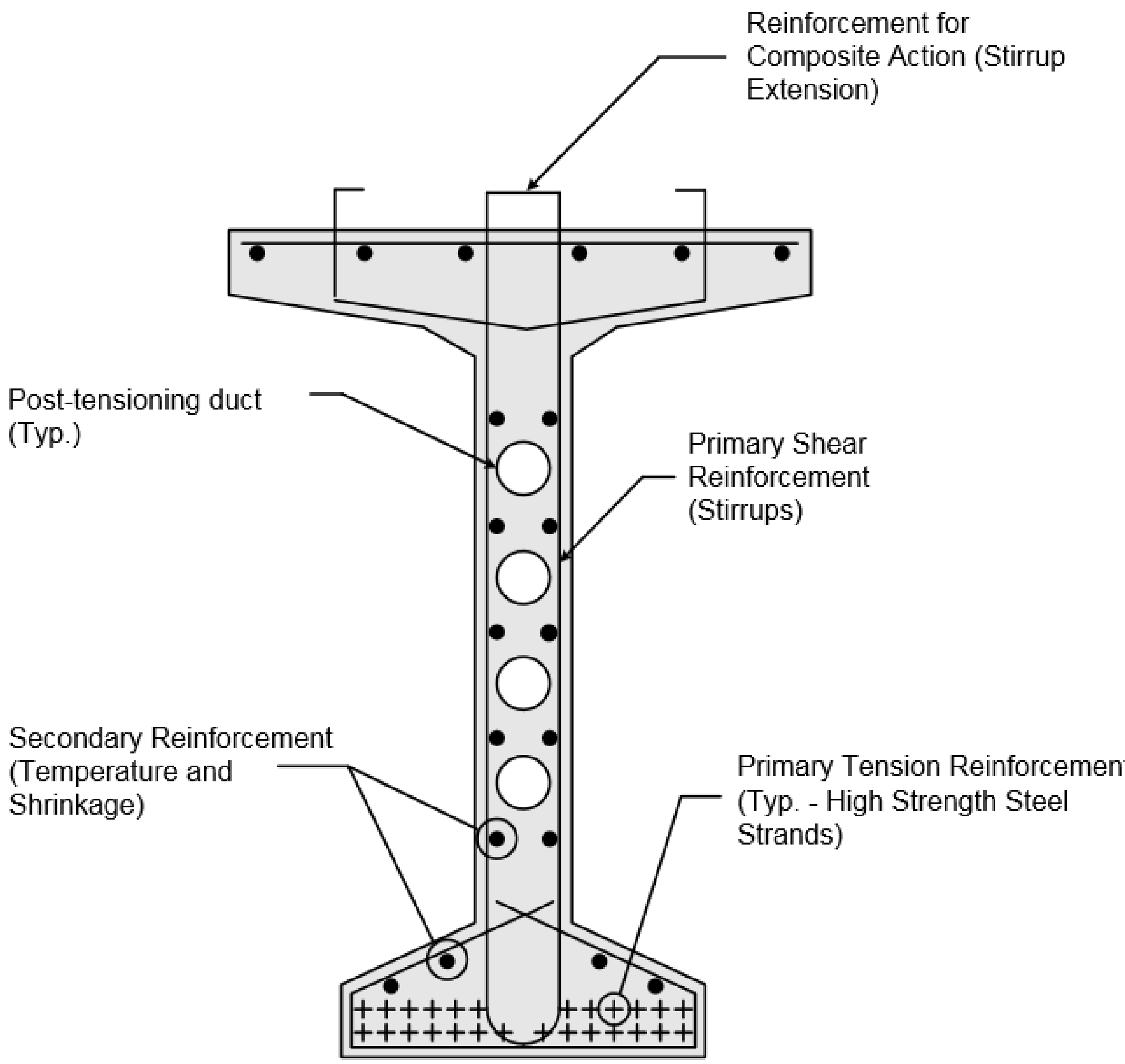


Figure 9.1.53 Steel Reinforcement in a Prestressed Bulb Tee Beam

9.1.12 Cast-In-Place and Precast Box Girders

Design Characteristics

Concrete box girders are typically trapezoidal with cantilevered top flanges that combine mild steel reinforcement and high strength post-tensioning tendons into a cross section capable of accommodating an entire roadway width. Concrete box girders can be cast-in place, precast segmental or cast-in-place segmental with single or multiple cells. (see Figure 9.1.54 and Figure 9.1.55).

Figure 9.1.54 Precast Segmental Concrete Box Girder Bridge

Figure 9.1.55 Cast-in-place Concrete Box Girder Bridge

For wide roadways, the box portion may have internal webs and is referred to as a multi-cell box girder (see Figure 9.1.56). Concrete box girder bridges are designed as single span or continuous multi-span structures. Spans can have a straight or curved alignment and are generally in excess of 150 ft.

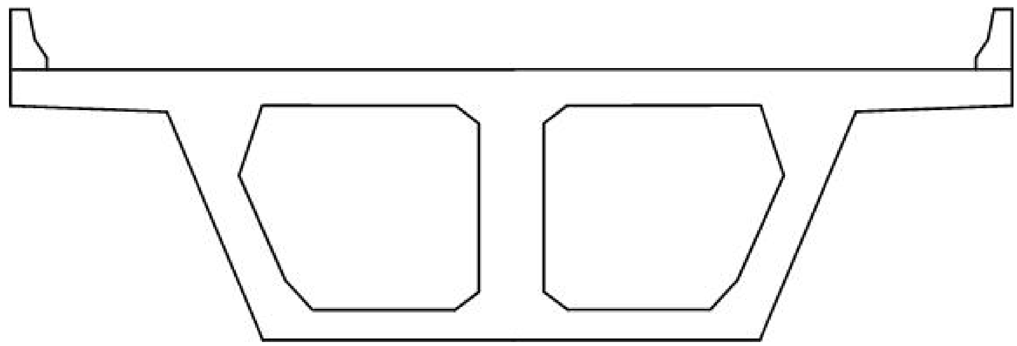


Figure 9.1.56 Cross Section - Multi-cell Box Girder

Many current box girders are built using segmental construction. A segmental concrete bridge is fabricated piece by piece. These pieces, or segments, are post-tensioned together during construction of the bridge (see Figure 9.1.57 and Figure 9.1.58). The superstructure can be constructed of precast concrete or cast-in-place concrete segments. Several characteristics are common to precast segmental bridges:

- Used for medium and long span bridges (spans can be as short as 130 ft).
- 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 9.1.57).
- 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 should be supported from previously erected segments during construction.

Figure 9.1.57 Adjacent Single Cell Segmental Boxes with Closure Pour

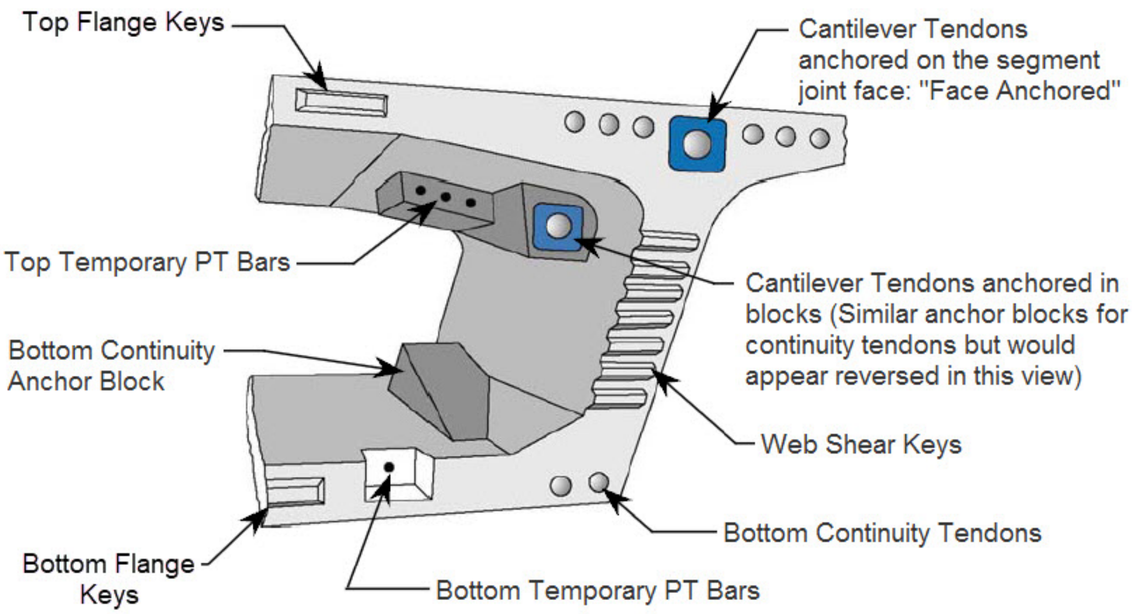


Figure 9.1.58 Typical Features of Precast Cantilever Box Girder Segments

Primary Members

For box girder structures, the primary superstructure member is the box girder itself. When the box girder design is used, the top flange or deck, the bottom flange, and webs are all primary members of the box girder (see Figure 9.1.59 and Figure 9.1.60). The top flange is considered an integral deck member.

The interior webs and interior diaphragm are primary members since they help support the top flange.

Any exterior diaphragms between tangent box girders are considered secondary members; exterior diaphragms between curved box girders are considered primary members.

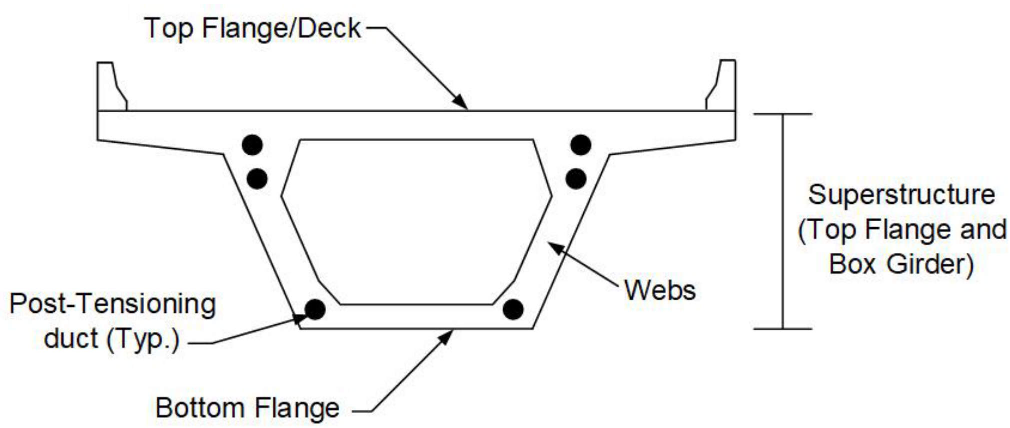


Figure 9.1.59 Basic Members of a Single Cell Concrete Box Girder

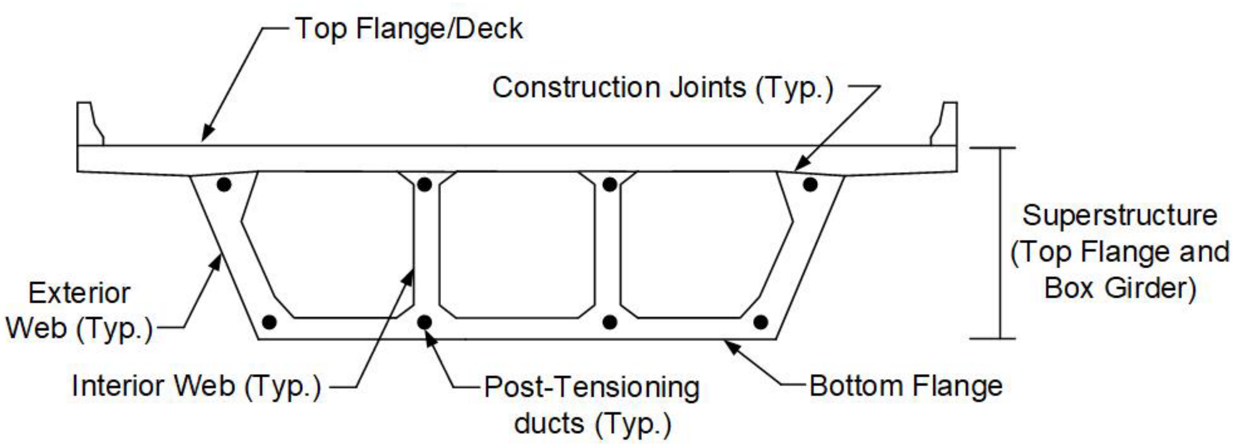


Figure 9.1.60 Replaceable Deck of a Multiple Cell Cast-in-place Box Girder

Steel Reinforcement

Box girder structures use a combination of primary mild steel reinforcement and high strength post-tensioning and pre-tensioning steel tendons to resist tension and shear forces (see Figure 9.1.61).

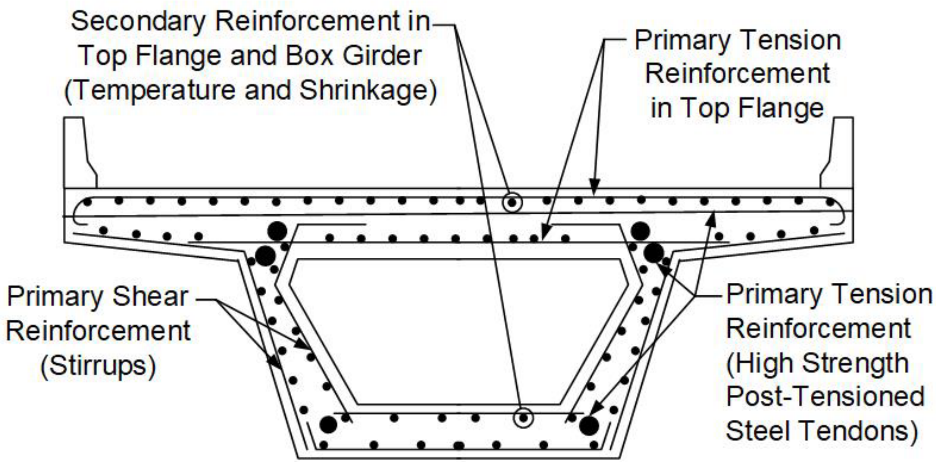


Figure 9.1.61 Steel Reinforcement in a Concrete Box Girder

Tension reinforcement is provided in the top and bottom flanges of the box girder as necessary (bottom flange at midspan in areas of positive moment and top flange over interior supports in areas of negative moment) to resist flexure. However, because of the design span lengths, mild steel reinforcement does not typically have sufficient strength to resist all of the tension forces. To reduce these tensile stresses to the anticipated levels for prevention of concrete cracking, prestressing of the concrete is introduced through post-tensioning. Galvanized metal and plastic ducts are placed in the forms at the desired location of the tendons. When the concrete has cured to the design strength, or the segmental boxes are erected and ready for the next phase of construction, the tendons are installed into the ducts, tensioned, and then grouted.

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 into block-outs in the edges of the 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 may be needed. This reinforcement is provided in the form of additional stirrups.

The secondary (temperature and shrinkage) reinforcing steel is oriented longitudinally in the deck, webs, and flanges in the box girder. The primary and secondary reinforcing steel for the deck portion of the girder is the same as a standard concrete deck (see Figure 9.1.61).

Special “confinement” reinforcement may also be needed at the anchorage locations to prevent cracking due to the large transfer of force to the surrounding concrete (see Figure 9.1.62).

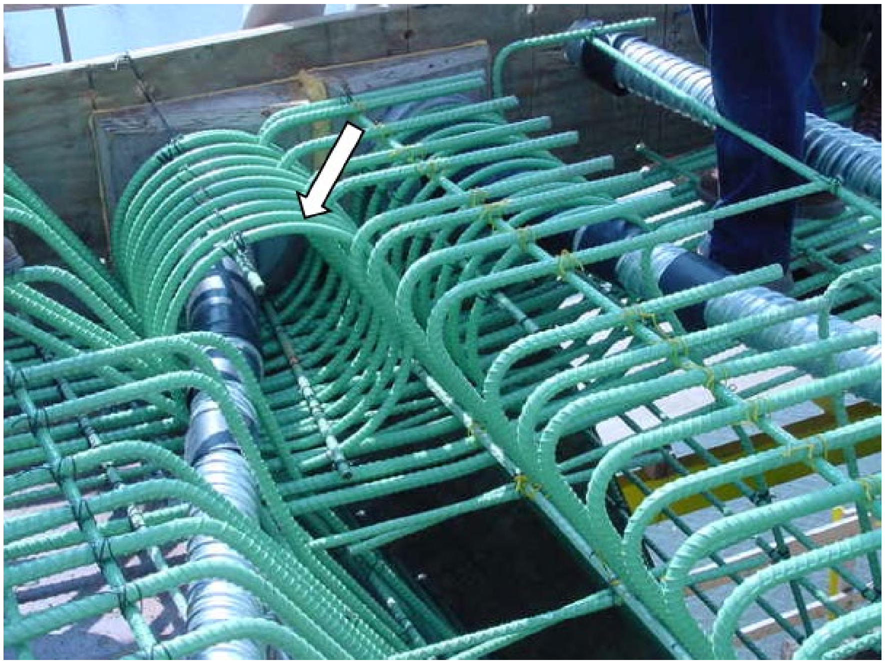


Figure 9.1.62 Post-tensioning Anchorage Reinforcement Before Concrete Placement

9.1.13 Secondary Members

Although secondary members do not resist primary loads, such as vehicular live load, they are important features for construction and the global stability of the bridge. Secondary members also distribute secondary live load forces such as wind. For all concrete superstructure types, there are a few common member types that are considered secondary.

Diaphragms

Diaphragms are common secondary superstructure members on concrete beam bridges. End diaphragms can provide restraint, act as a backwall, or support the free edge of the beam top flange if present. They may be full or partial depth (see Figure 9.1.63).

Intermediate diaphragms may also be present in longer span bridges and are usually at the half or third points along the span (see Figure 9.1.64).They can be used to compensate for torsional forces. Intermediate diaphragms are usually partial depth.

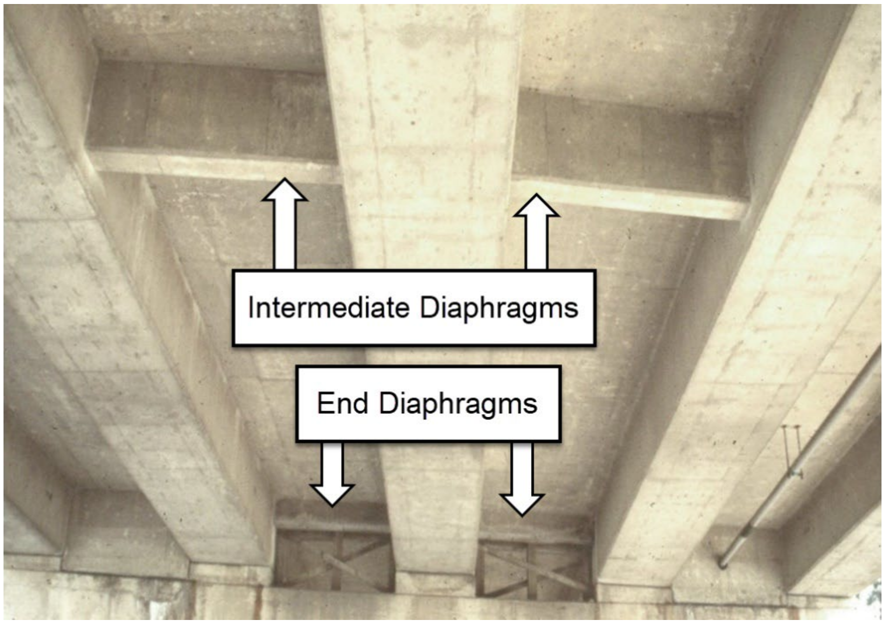


Figure 9.1.63 End and Intermediate Diaphragms on a Spread Box Beam Bridge

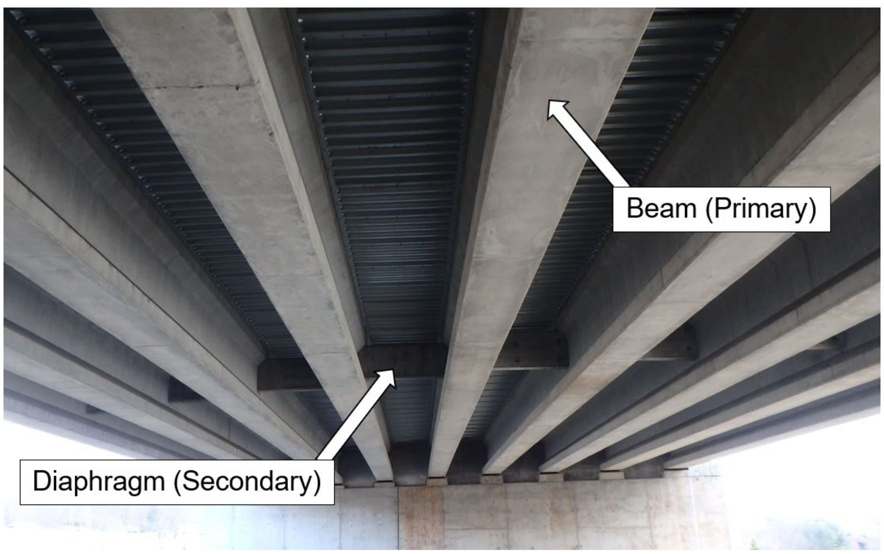


Figure 9.1.64 Intermediate Diaphragms on an I-Beam Bridge

Diaphragms may be present between concrete frame legs but are not common to the bridge type. Adjacent box beam bridges are another superstructure type that typically does not have secondary members.

Struts

The secondary members of an open spandrel arch bridge are the arch struts, which are transverse members connecting the arch ribs. Arch struts provide stability against lateral forces and reduce the unsupported compression length between supports (see Figure 9.1.22) for the open spandrel arch ribs.

Section 9.2 Overview of Common Deficiencies

9.2.1 Concrete Deficiencies

Common deficiencies that may occur on conventional or prestressed reinforced concrete superstructures include:

- Cracking (structural).
- Cracking (non-structural).
- Scaling.
- Delamination.
- Spalling.
- Chloride contamination.
- Freeze-thaw.
- Efflorescence.
- Alkali silica reactivity (ASR).
- Ettringite formation.
- Honeycombs.
- Pop-outs.
- Wear.
- Collision damage.
- Abrasion.
- Overload damage.
- Reinforcing steel corrosion.
- Carbonation.
- Loss of prestress.
- Independent beam action.

Refer to Chapter 7 for a detailed explanation of the properties of concrete, types and causes of concrete deficiencies.

Section 9.3 Inspection Methods

9.3.1 Visual and Physical Inspection

It is important to utilize inspection methods to fully ascertain conditions, including the causes and importance of each deficiency, so that all findings can be accurately noted and critical findings can be identified. The inspection of concrete for surface deficiencies is primarily a visual activity of looking at all surfaces for deficiencies such as cracking, scaling, and spalling. It is also important to scan each component from end to end to observe any negative or downward camber, misalignment, or other signs of movement.

Physical inspection methods often are employed to supplement visual inspection methods. Physical methods for concrete are primarily used to detect delaminated areas by sounding and to measure defects such as cracks, spalls, scaling, and other defects. Connections at tie bolts and transverse stressed rods can be tapped with a hammer to determine integrity and tightness of the connections.

Refer to Chapter 7 for detailed procedures of visual and physical inspection methods.

9.3.2 Advanced Inspection

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

There are several advanced methods for concrete inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified nondestructive evaluation (NDE) technician include:

- Electrical Resistivity (ER).
- Galvanostatic Pulse Measurement (GPM).
- Ground Penetrating Radar (GPR).
- Half-Cell Potential (HCP).
- Impact Echo (IE).
- Infrared Thermography (IT).
- Linear Polarization (LPR).
- Magnetic Flux Leakage (MFL).
- Magnetometer (MM).
- Rebound and penetration methods.
- Ultrasonic Pulse Velocity (UPV).
- Ultrasonic Pulse Echo (UPE).
- Ultrasonic Surface Waves (USW).

Other methods that may be considered destructive testing methods include:

- Concrete cores.
- Borescopes.
- Moisture content.
- Petrographic examination.
- Alkali-Silica Reaction (ASR) evaluation.
- Reinforcing steel sample.
- Rapid chloride permeability testing.

All advanced methods are described in detail in Chapter 17.

Section 9.4 Inspection Areas and Deficiencies

Before inspecting specific areas below the bridge, the concrete superstructure should be inspected for general global alignment. Sight along the curb line, top of railing or the bottom of the superstructure and look for vertical or horizontal misalignment. Both sides of expansion joints should be viewed to check if they line up properly. Vertical misalignment, horizontal misalignment or sagging may be an indication of several structural issues. For instance, sagging may be evidence of prestress strand relaxation. Misalignment of expansion joints may be evidence of differential substructure settlement or deficiencies in the bearing devices. Also, the secondary members should be inspected. Distorted secondary members may be evidence of primary member differential movement or overstress. Misalignment may be indicators of potential deficiencies. Visual, physical, or advanced inspection methods may be beneficial in evaluating the following inspection areas and deficiencies.

9.4.1 Bearing Areas

Bearing areas are where one member “sits” on top of another member. Typically, the bearing areas for a bridge are where the superstructure bears on to the substructure supports, usually via a bearing device.

Deficiencies may be caused by deck joint water leakage or restriction of thermal movement due to a faulty bearing mechanism leading to cracking, spalling, and corrosion. Spalling could also be caused by poor quality concrete placement, including insufficient rebar cover. Bearing areas should be examined for cracking, delamination, and spalling where friction from thermal movement and high edge or bearing pressure could overstress the concrete. The condition and operation of any bearing devices should be checked. Crushing near the bearing seat should be checked. Rust stains, which indicate corrosion of steel reinforcement, should be inspected. The effects of temperature, creep, and concrete shrinkage may produce undesirable conditions at the bearings. The bearing areas and the bearings should be checked for proper movement and movement capability.

The bottom of prestressed box or voided slab beams may be checked 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 undergoing freezing and thawing. Rust staining around the cracks should be inspected. Rust staining is an indication of reinforcement corrosion and possible section loss.

The floor system/bent cap interface is also exposed to bearing loading. Bearing areas should be examined as described in this section.

For typical bearing areas and potential defects, see Figure 9.4.1 through Figure 9.4.8.

Figure 9.4.1 Bearing Area: Cast-in-Place Slab

Figure 9.4.2 Bearing Area: Cast-in-Place Concrete Tee Beam Bridge

Figure 9.4.3 Bearing Area: Spalling and Exposed Tee Beam Reinforcement

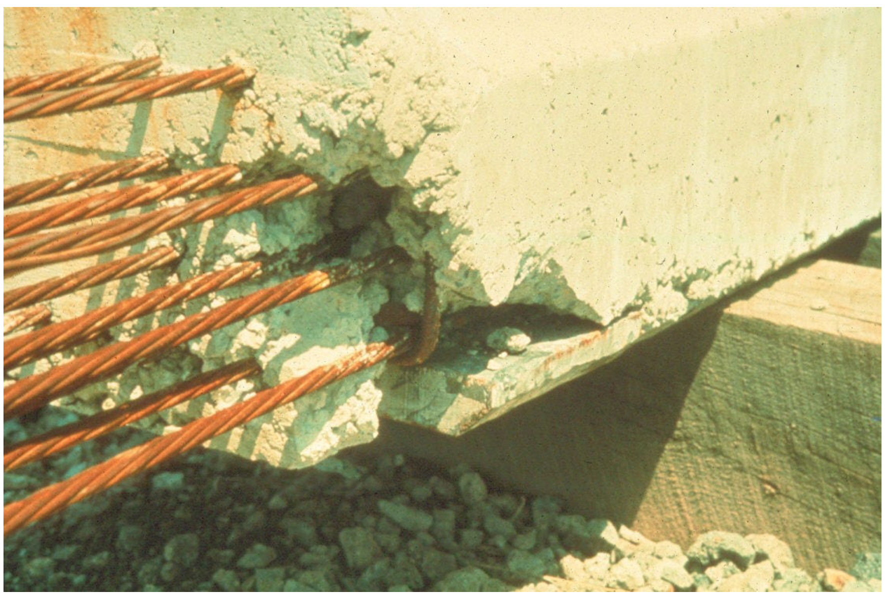


Figure 9.4.4 Bearing Area: Spalling Due to Poor Concrete and Steel Placement

Figure 9.4.5 Bearing Area: Spalled Beam Ends with Exposed Prestressing Reinforcement

Figure 9.4.6 Bearing Area: Longitudinal Cracks in Web of Box Beam

For arches, the arch/thrust block interface has the greatest bearing load magnitude (see Figure 9.4.7). Arches and the thrust block interface should be inspected for loss of cross section of the reinforcement bars at any spalls. The arch should be examined for any longitudinal cracks. These may indicate an overstress condition.

The arch/spandrel column interface has the second greatest bearing load magnitude. This interface should be examined for reinforcement cross-section loss at any spalls. Horizontal cracks in the columns within several feet from the arch should be checked. These indicate excessive bending in the column, which is caused by overloads and differential arch rib deflection (see Figure 9.4.7).

The spandrel column/cap interface has the third greatest bearing load magnitude. This area should be inspected for loss of section at spalled areas. The column may be examined for cracks which begin at the inside corner and propagate upward. These indicate differential arch rib deflections (see Figure 9.4.8).

The arch ring should also be examined for unsound concrete. Rust stains, cracks, discoloration, crushing, and deterioration of the concrete should be examined. The interface between the spandrel wall and the arch may be checked for spalls that could reduce the bearing area. Any transverse cracks found on the arch should be investigated, which indicate an overstress condition.

Figure 9.4.7 Bearing Area: Arch/Thrust Block Interface

Figure 9.4.8 Bearing Area: Spandrel Column/Arch Rib Interface

9.4.2 Shear Zones

Areas near the supports should be investigated for shear cracking. The presence of transverse cracks on the underside of the superstructure member near supports or diagonal cracks on the sides of the member indicate the onset of an overstress for shear. Cracks may represent a loss in shear capacity and should be carefully measured. Shear reinforcement in high stress areas should be inspected for exposure and potential section loss on the stirrups.

For typical shear areas and potential deficiencies, see Figure 9.4.9 through Figure 9.4.17.

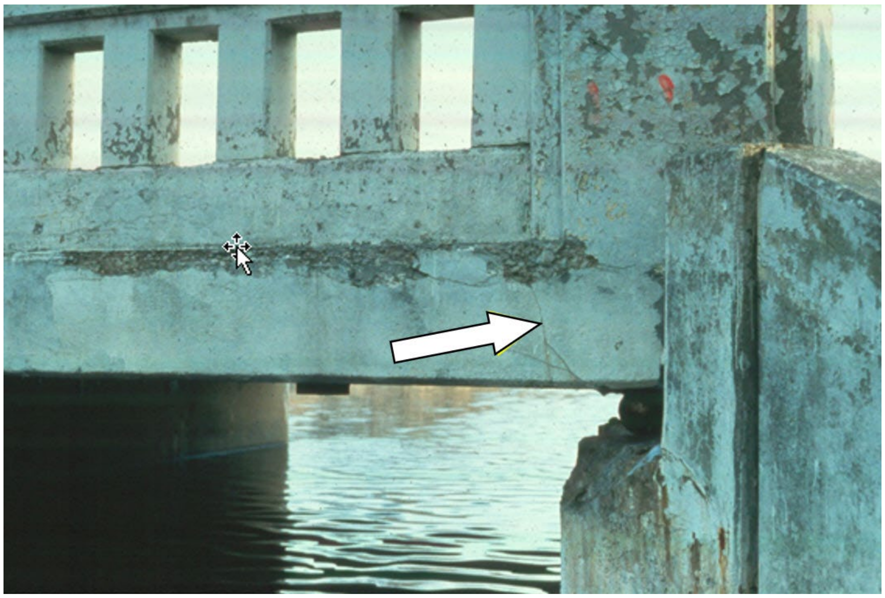


Figure 9.4.9 Diagonal Shear Cracks Close to the Ends of a Slab Bridge

Figure 9.4.10 Exposed Shear Reinforcement at End of Box Beam

Figure 9.4.11 Shear Zone of Cast-in-Place Concrete Tee Beam Bridge

Shear cracks at the ends of the spandrel bent caps should be checked. When arch ribs are connected with struts, the arches near the connection should be examined for diagonal cracks due to torsional shear. These cracks indicate excessive differential deflection in the arch ribs. Also, the floor system may be investigated for shear cracks in a fashion similar to tee beams and girders.

Areas near the supports where the frame beams or slab meet the frame legs or abutments should be inspected. Shear cracks in the frame beams or slab (beginning at the frame legs and propagating upward toward mid-span) should be checked. The frame legs should be inspected for diagonal cracks that initiated at the frame beam/slab or footing (see Figure 9.4.12).

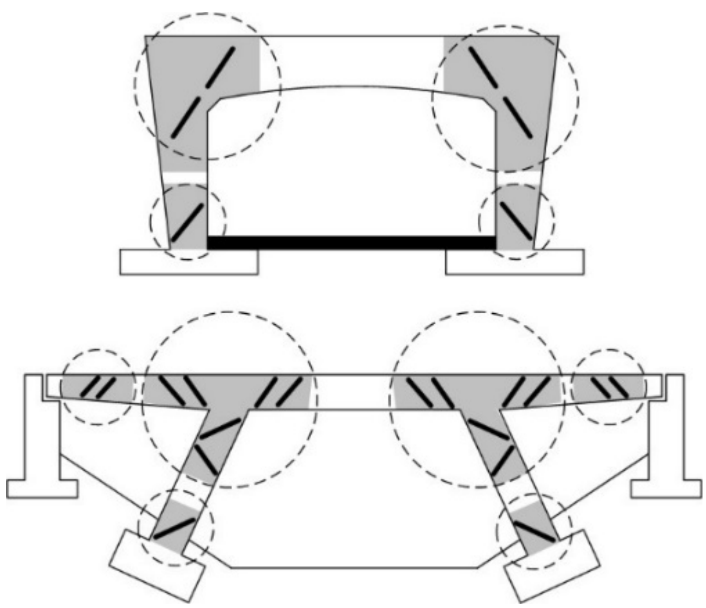


Figure 9.4.12 Shear Zones in Single Span and Multi-span Frames

The top and bottom surface of slab, box section, or wearing surface should be inspected for longitudinal reflective cracking which may indicate leakage and broken connections between the superstructure members (see Figure 9.4.13). These problems indicate failed shear keys and that the superstructure units are no longer tied together or acting monolithically (see Figure 9.4.14). Differential deflection of adjacent superstructure members under live load should be checked.

Figure 9.4.13 Shear Key Leaking Joint Between Adjacent Box Beams

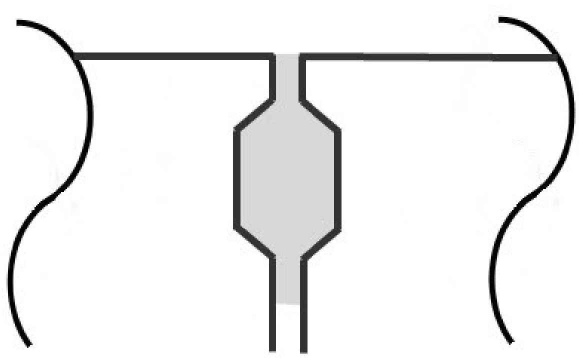


Figure 9.4.14 Shear Key Joint with Non-Shrink Grout Schematic

For dapped-end double tee beams, diagonal shear cracks in the reduced depth section that sits on the bearing seat should be checked. At the full depth-to-reduced-depth vertical interface, vertical direct shear cracking should be investigated. At the bottom corner where the reduced section meets the full depth section, diagonal shear corner cracks should be inspected (see Figure 9.4.15).

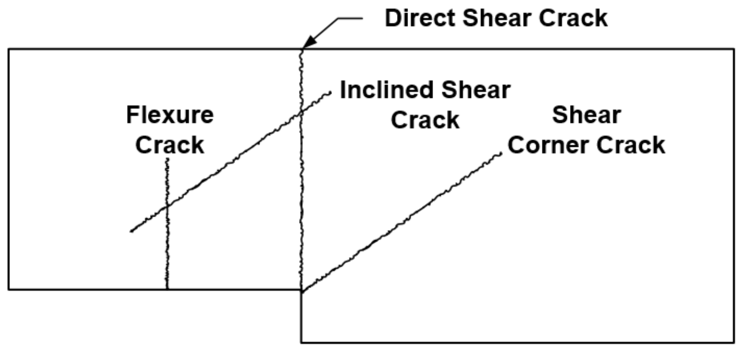


Figure 9.4.15 Crack Location for Dapped End Double Tee Beams

Box girder ends and the sections close to piers should be checked for diagonal shear cracks in webs. These web cracks project diagonally upward at approximately a 45-degree angle from the support toward midspan (see Figure 9.4.16).

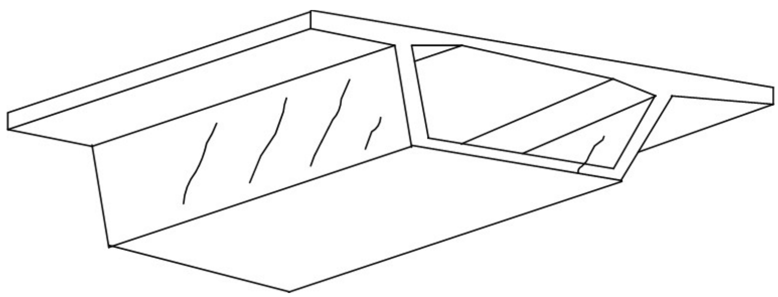


Figure 9.4.16 Box Girder Cracks Induced by Shear

Torsion and shear cracking occurs 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 produces a helical configuration if torsion alone was present. Bridge structures most often do not experience torsion alone; rather bending, shear and torsion occur simultaneously. In this event, cracking is more pronounced on one side of the box girder due to the additive effects of all forces (see Figure 9.4.17).

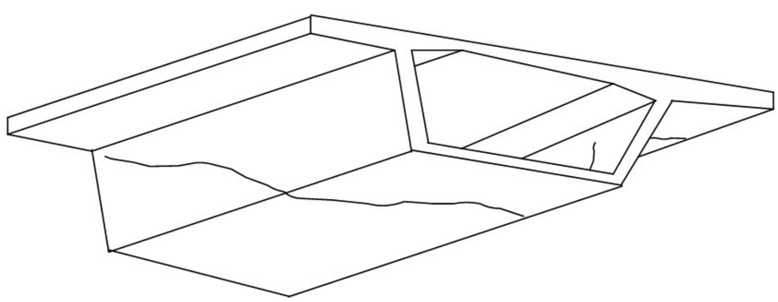


Figure 9.4.17 Box Girder Cracks Induced by Torsion and Shear

The interior and the exterior surfaces of the box girder should both be inspected. The inspection methods for shear zones in segmental box girder bridges are typically the same as for concrete box girder bridges.

9.4.3 Tension and Flexure Zones

Cracking, rusting of primary reinforcement, and other defects in tension zones could indicate serious issues or critical findings. Tension zones should be examined for flexure cracks perpendicular to tensile forces, vertical on the sides of the member, and transverse across the superstructure member. The tension zones tend to be at midspan along the bottom of the member for both simple and continuous span bridges. Additional tension zones are on top of the element over the piers for continuous spans. Transverse/vertical cracks may indicate overstress due to tensile forces caused by bending stresses. Efflorescence from cracks and, more significant, the discoloration of the efflorescence caused by rust stains from the reinforcing steel should be checked. In severe cases, the reinforcing steel may become exposed due to spalling. Reinforcing steel section loss may decrease live load capacity and the section loss should be measured and documented to update the load rating.

For prestressed members, inspectors should examine for superstructure sagging or reinforcement section loss.

Similar tension zones should be checked for floorbeams and stringers if present. For open spandrel arch bridges, the tension areas of the spandrel bent caps and columns (i.e., mid-span at the bottom and ends at the top) should be inspected. The tension areas in the floor system should also be checked.

The spandrel walls should be inspected. Cracks, movement, and general deterioration of the concrete should be investigated.

The bottom of the prestressed superstructure sections should be checked 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/relaxation of prestress. Fine cracks may be difficult to detect with the naked eye. All cracks should be measured with an optical crack gauge or crack comparator card to document and track crack size and length growth.

The top of the superstructure sections (if exposed) near the ends should be examined for tensile cracks due to prestress eccentricity. This indicates excessive prestress force. If the top flange has a wearing surface applied, cracks in the wearing surface may be checked. Cracks in the wearing surface may be an indication that the superstructure is overstressed or that water is getting to the superstructure.

A string line may be used or site down the bottom edge of the superstructure member. Superstructure members with negative camber may be an indication of prestress loss or prestressed reinforcement section loss.

The superstructures should be inspected for exposed strands. Longitudinal cracks on the bottom flange of adjacent or spread box beams can be an indication of prestressing steel corrosion and section loss. Prestressed strands can corrode rapidly and fail abruptly; therefore, any exposure may be significant.

Water freezing in the voids can cause longitudinal cracks. Skewed solid or voided slab beams may exhibit longitudinal cracks due to uneven prestressing force in the strands.

Any exposed tension reinforcement should be checked and any section loss observed on the prestressing tendons should be documented. Pitting or section loss may decrease live load capacity. Exposed prestressing tendons are susceptible to stress corrosion and sudden failure.

Flexure cracks can appear in the top flange at pier locations and on the bottom flange at mid-span regions. The extent of cracking depends on the intensity of the bending being induced. Flexure cracks will typically propagate to the neutral axis or to an area around the half-depth of the section. Flexural cracks found in post-tensioned members should be examined carefully.

For typical tension areas and potential deficiencies, see Figure 9.4.18 through Figure 9.4.29.

Figure 9.4.18 Flexure Cracks on a Tee Beam Stem

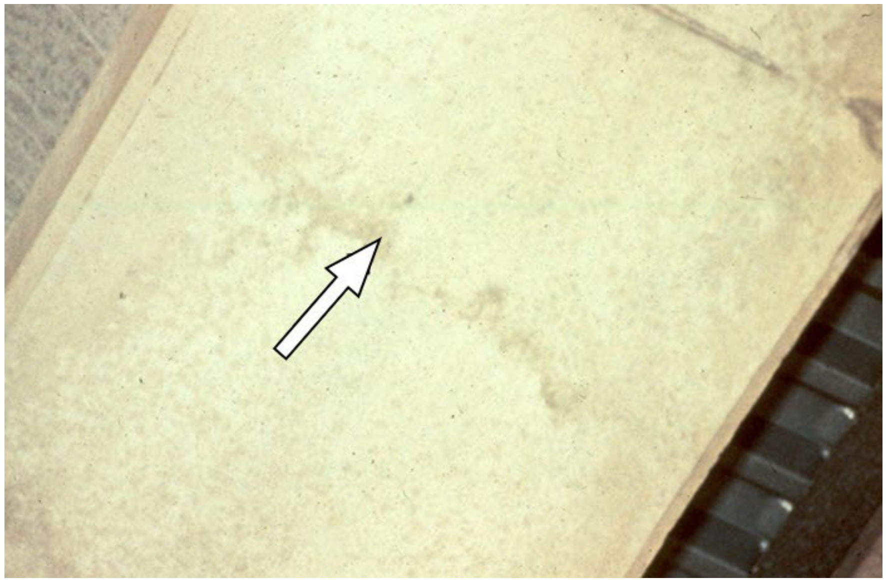


Figure 9.4.19 Flexure Crack on Bottom Flange of Prestressed Box Beam

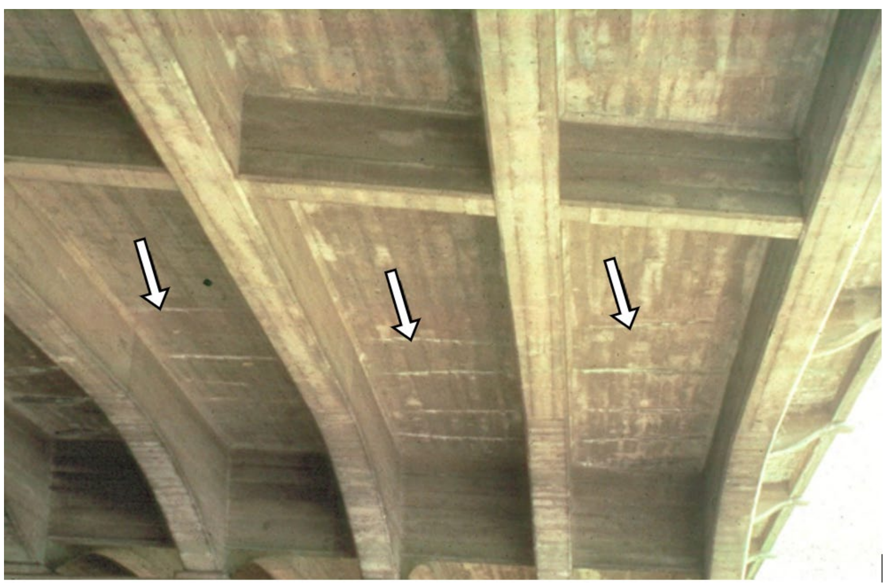


Figure 9.4.20 Flexure Cracks in Tee Beam Top Flange/Deck

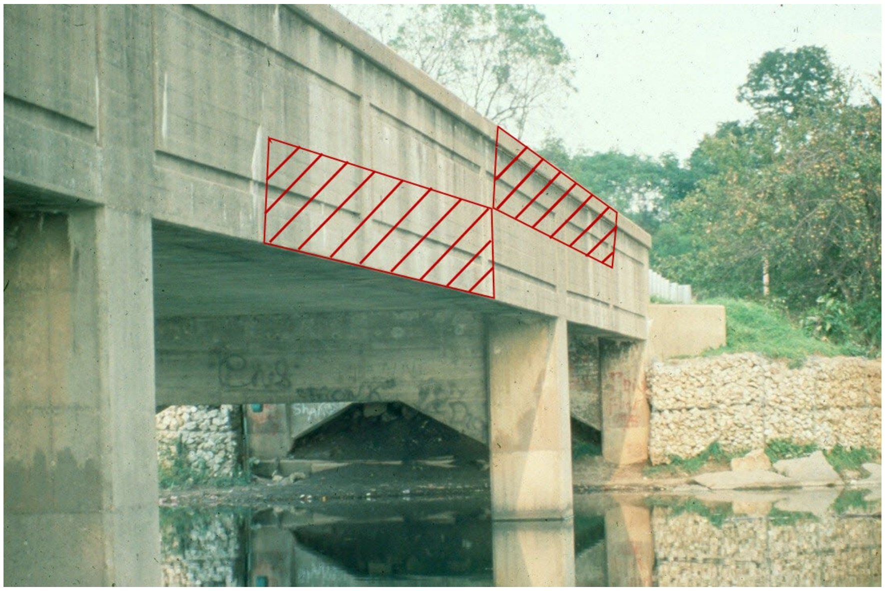


Figure 9.4.21 Through Two-Girder Bridge Elevation with Tension Zones Indicated

Figure 9.4.22 Slab Max. Moment Area: Delamination, Efflorescence, Rust Stains

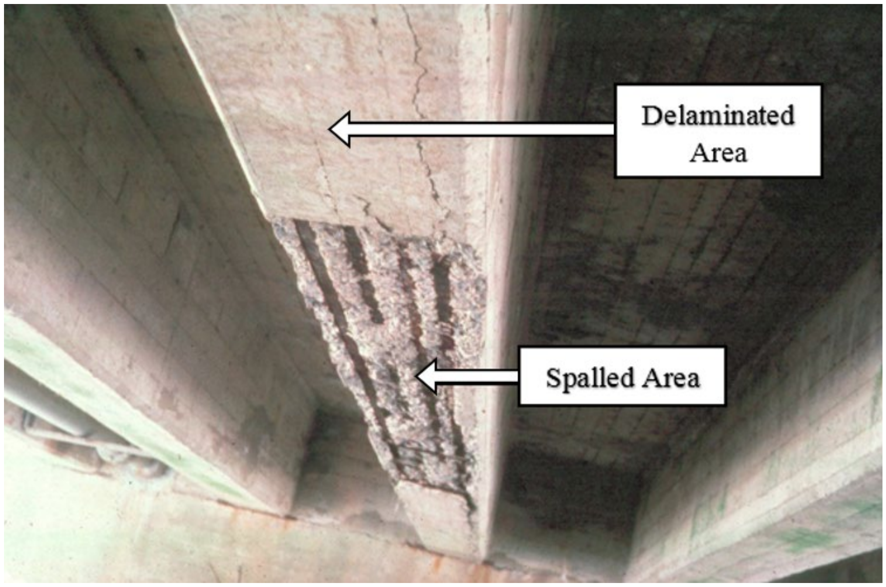


Figure 9.4.23 Delaminated and Spalled Tee Beam with Corroded Primary Reinforcement

Figure 9.4.24 Deteriorated Arch/Spandrel Wall Interface

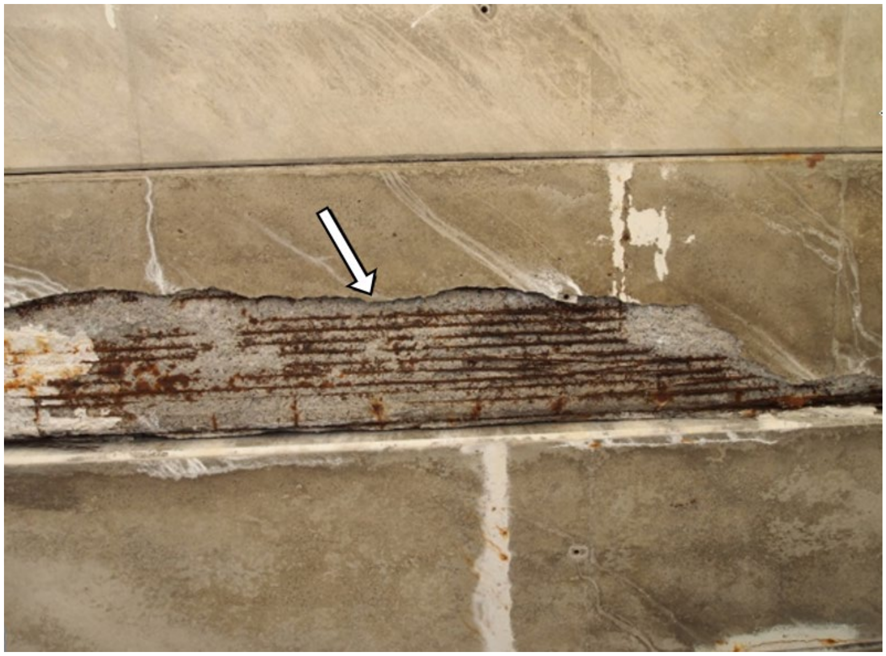


Figure 9.4.25 Spall and Exposed Corroded Reinforcement Strands, Mid-Span of Box Beam

The loading in prestressed concrete box girders may be extremely complicated due to non-symmetric dead loads, centrifugal forces, and temperature/shrinkage. (see Figure 9.4.26 and Figure 9.4.27).

Similar to a deck supported by longitudinal superstructure members, the top side of the top flange should be inspected for longitudinal flexure cracking directly over interior and exterior girder webs. Inside the box, the bottom of the top flange may be examined for longitudinal flexure cracking between the girder webs. These longitudinal cracks are caused by overstressing of the deck. Any efflorescence or leakage through the top flange should be documented.

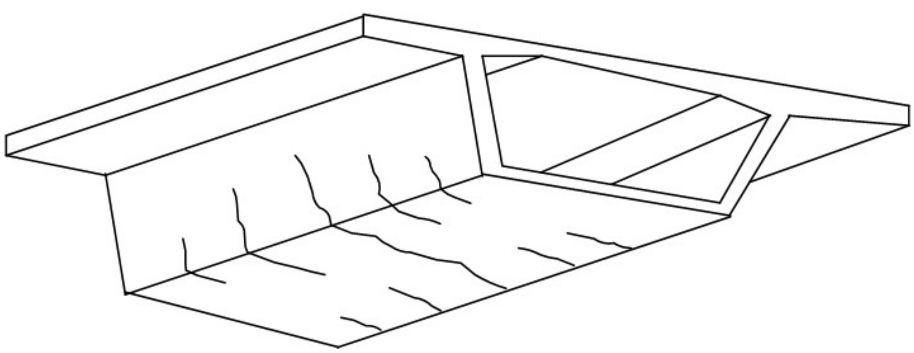


Figure 9.4.26 Box Girder Cracks Induced by Flexure (Positive Moment)

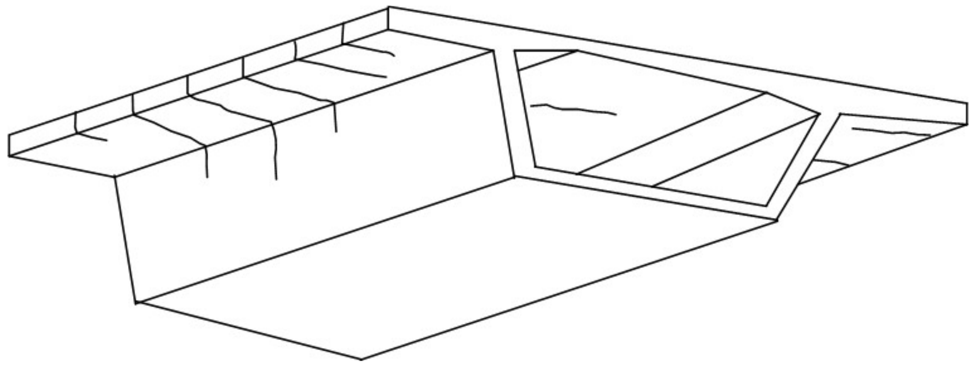


Figure 9.4.27 Box Girder Cracks Induced by Flexure (Negative Moment)

Flexure-shear cracks can appear close to pier support locations. They initiate on the bottom flange and are oriented transversely to the longitudinal axis of the bridge. The cracking propagates up the webs approximately 45 degrees to the horizontal and toward mid-span (see Figure 9.4.28).

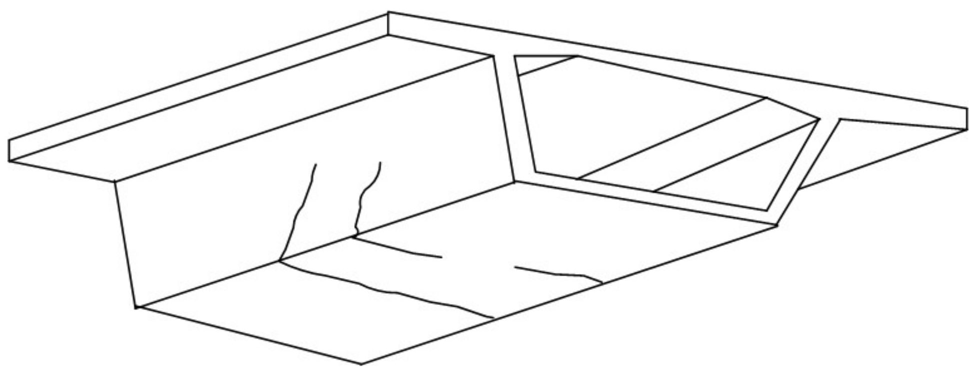


Figure 9.4.28 Box Girder Cracks Induced by Flexure-Shear

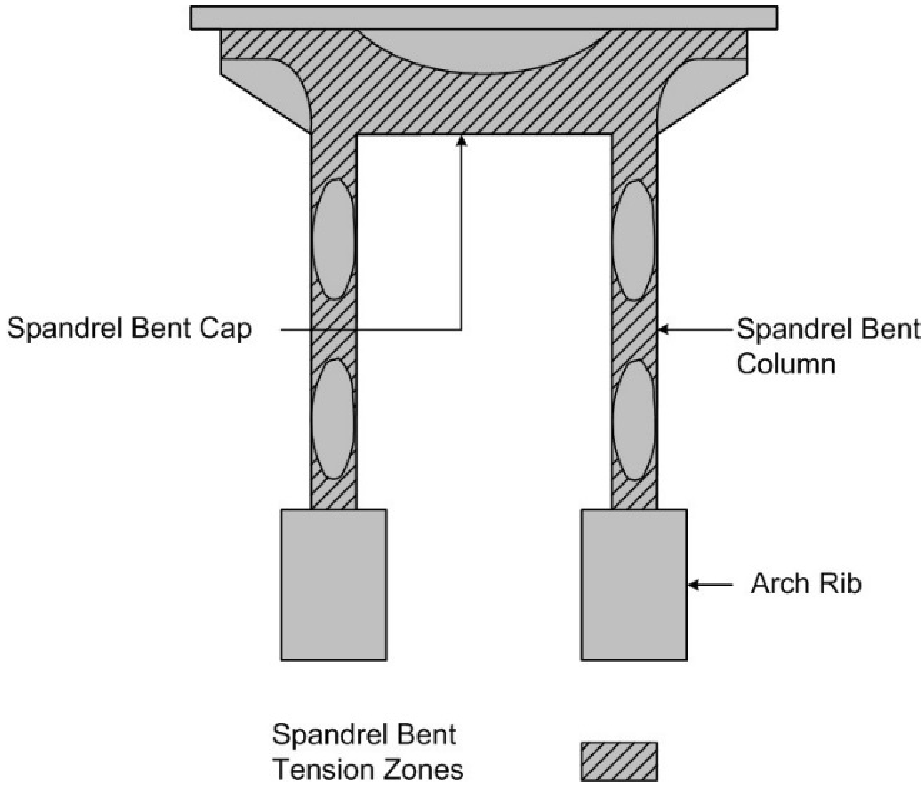


Figure 9.4.29 Spandrel Bent Tension Zones

9.4.4 Areas Exposed to Drainage

Areas exposed to roadway drainage should be investigated for deteriorated concrete. This includes the entire riding surface of the deck slab or top flanges, particularly around scuppers or drains. Spalling or scaling may also be found along the curb line and fascias.

If the roadway surface is bare concrete, any delaminated areas, scaling, and spalls should be checked. The curb lines are the most common location for water ponding and salt intrusion. If the top flange has an asphalt wearing surface, indications of deteriorated concrete such as reflective cracking and depressions in the wearing surface should be checked.

For typical areas exposed to drainage and potential deficiencies, see Figure 9.4.30 through Figure 9.4.41.

Figure 9.4.30 Asphalt Covered Tee Beam Flange

Figure 9.4.31 Deteriorated Tee Beam Stem Adjacent to Drain Hole

Figure 9.4.32 Interior Face of a Through Girder with Scaling

The seam or joint between adjacent precast beams should be inspected for leakage. Leakage generally indicates a broken shear key between the beams (see Figure 9.4.33). If signs of leakage are present between beams, the beams should be closely observed for differential deflection under live load (see Figure 9.4.34). Also, beam ends may be checked for concrete deterioration due to leaking joints.

Figure 9.4.33 Joint Leakage Between Channel Beams

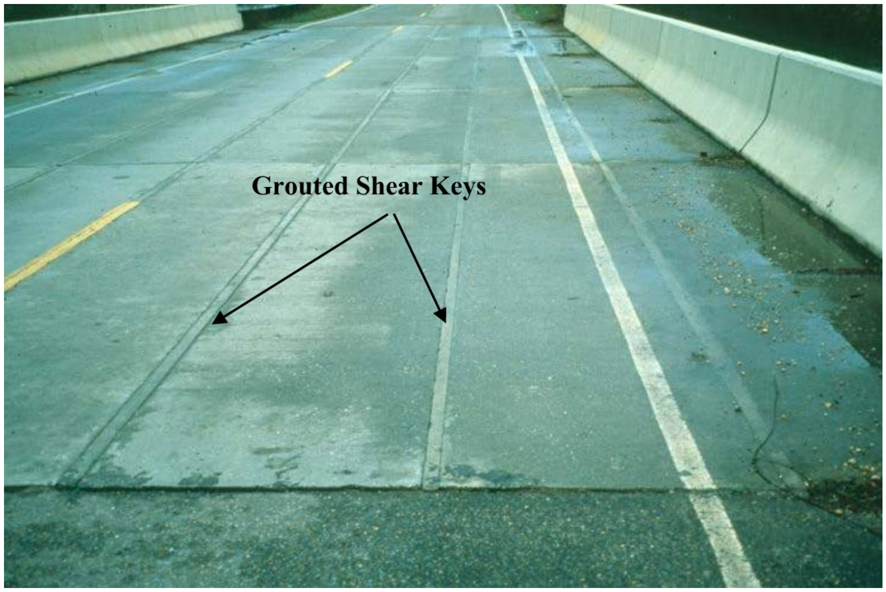


Figure 9.4.34 Top Surface of Deck of Precast Channel Beam Bridge

The web or stem tie-bolts should be checked for tightness and corrosion (see Figure 9.4.35 and Figure 9.4.36). Signs of corrosion should not be confused with the epoxy used for the bolt. Corrosion may be an indication of leaking through a broken shear key.

Figure 9.4.35 Transverse Tie Bolt with Epoxy

Figure 9.4.36 Stem Tie Bolts

For an open spandrel arch, the areas exposed to drainage and roadway runoff should be checked. Members beneath the floor system are prone to scaling, spalling, and chloride contamination (see Figure 9.4.37).

Figure 9.4.37 Scaling and Contamination on an Arch Rib Due to a Failed Drainage System

For a closed spandrel arch, it should be verified that the weep holes are working properly. Also, it should be checked that surface water drains properly and does not penetrate the fill material. Longitudinal joint areas of adjacent slab or frame beams may be checked for leakage and concrete deterioration (see Figure 9.4.38). Slab or frame beam ends should be checked for deterioration due to leaking deck joints at the abutments. It should also be checked to see if weep holes are functioning.

Figure 9.4.38 Longitudinal Joint Between Slab Frames

The ends of beams may be deteriorated due to water leaking through the deck expansion joints and should be checked (see Figure 9.4.39).

Figure 9.4.39 Water Leakage at Deck Joint Between Spans

Joints between slab beams and adjacent box beams should be examined for leakage and rust stains. Reflective cracking in the traffic surface and differential beam deflection under live load should be investigated. These problems indicate that the shear key between boxes or slab beams has been broken and that they are acting independently of each other (see Figure 9.4.40). 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. These issues may indicate reduction in load capacity and therefore should be considered serious.

Figure 9.4.40 Joint Leakage and Rust Staining

Drain holes should be checked for proper function as accumulated water can freeze and crack the beam. Older prestressed beams used cardboard to form the voids. The wet cardboard can clog the drain holes. If possible, unplug the drain holes if found clogged. If unable to open, request assistance from maintenance forces.

Drain holes are typically provided at the low point of each prestressed box girder “cell” (see Figure 9.4.41). 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. These devices should be inspected for missing screens or clogging due to debris.

Figure 9.4.41 Concrete Box Girder Drain Hole with Screen

9.4.5 Areas Exposed to Traffic

Any areas damaged by vehicle or vessel collision should be checked. This is especially important for two-girder bridges due to the low number of load paths. The number of exposed and severed reinforcing bars or strands with section loss as well as areas of spalled and delaminated concrete should be documented. The loss of concrete due to such an accident may not be serious unless the bond between the concrete and steel reinforcement is affected. If the top flange is exposed to traffic, signs of wear may be examined, especially at the wheel path locations.

For typical areas exposed to traffic and potential deficiencies, see Figure 9.4.42 through Figure 9.4.44.

Figure 9.4.42 Collision Damage to Tee Beam Bridge over a Highway

Figure 9.4.43 Inspectors Evaluating Collision Damage on Prestressed Concrete I-beam

Figure 9.4.44 Box Beam Damage Caused by Vehicle Striking Attached Railing

9.4.6 Areas Previously Repaired

Any previous repairs should be examined thoroughly. Repaired areas should be determined to be sound and functioning. Effective repairs such as patching and epoxy injection of cracks are usually limited to protection of exposed tendons and reinforcement (see Figure 9.4.45).

Figure 9.4.45 Collision Damage Repair on Prestressed Concrete I-Beam

9.4.7 Acute Angles on Skewed Bridges

Skewed concrete bridges should be examined for lateral displacement and deformation of bearings. The acute corners of skewed bridges should be inspected for cracking due to point loading and insufficient reinforcement, as well as honeycombing.

Skewed prestressed concrete bridges should be examined for lateral displacement and cracking of acute corners due to unsymmetrical strand release and insufficient reinforcement.

9.4.8 Thermal Effect

Thermal cracks are caused by non-uniform temperatures between two surfaces of the concrete superstructure. Cracking will typically be transverse in the thinner regions of concrete superstructure members and longitudinal near changes in cross section thickness such as the box girder top flange/web interface (see Figure 9.4.46 and Figure 9.4.47).

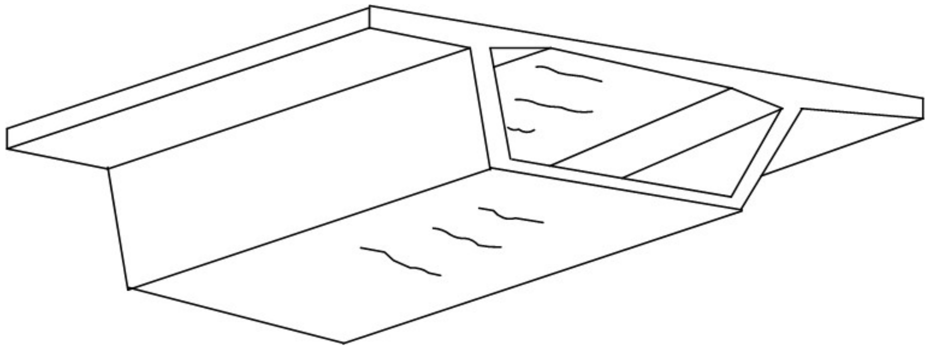


Figure 9.4.46 Thermally Induced Transverse Cracks in Box Girder Flanges

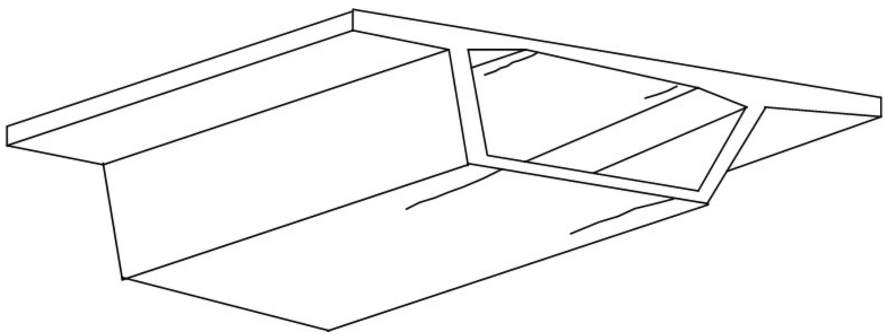


Figure 9.4.47 Thermally Induced Longitudinal Cracks in Box Girder Flanges

9.4.9 Prestressed-Induced Forces

Cracking can occur along any of the lines of prestressing tendons in areas such as the end anchorages and deviation blocks and even in the web or flanges. For this reason, it is important for the inspection team to be aware of where tendons are arranged in the beam (see Figure 9.4.49). This cracking may be the result of 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 duct around the tendon have been reported. These voided areas can fill with water, which may accelerate corrosion of the post-tensioning strands. In colder climates, the water may freeze and burst the duct 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, usually by the vacuum grouting method.

Figure 9.4.48 Post-tensioning Tendon Duct

Anchor Blocks

Anchor blocks (or blisters) contain the termination of the post-tensioning tendons. Very large concentrated loads are developed within these blocks. They tend to crack if not properly reinforced or if there are voids adjacent to the post-tensioning tendons. The cracking may be more of a splitting failure in the web and would be oriented in the direction of the post-tensioning tendon (see Figure 9.4.49 and Figure 9.4.51).

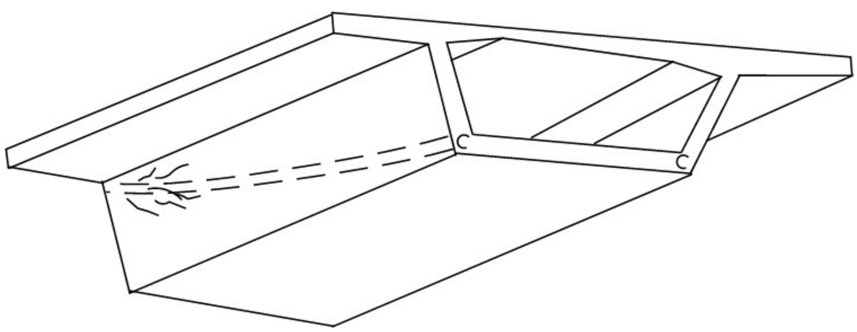


Figure 9.4.49 Web Splitting near an Anchorage Block

Deviation Blocks

Deviation blocks allow longitudinal post-tensioning tendons to change direction or angle within the box girder. 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 9.4.51). As with anchor blocks, carefully examine deviation blocks since these are points of very high stress concentrations and are subject to delamination, spalling and cracking.

Internal Diaphragms

End diaphragms at the piers and abutments serve to stiffen the box section and to distribute the large bearing reaction loads. Tendon anchorages within the diaphragm can 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 should be closely inspected (see Figure 9.4.51).

9.4.10 Segmental Box Girder Joint

Inspect transverse joints between segmental box girder sections for crushing and movement of the shear keys. Areas where closure joints or segments were poured in place should receive close observation. These areas sometimes are regions of tendon anchorages and couplers. The stress concentrations in these areas are very different from those 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.

Joints between the segments should be examined for any signs of distress, leakage, or infiltration (see Figure 9.4.50 and Figure 9.4.51).

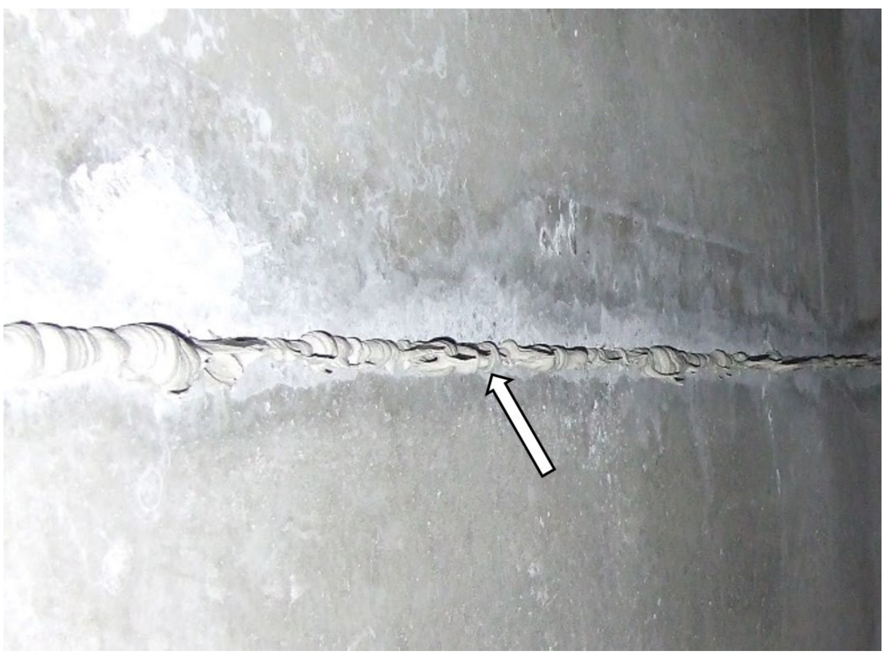


Figure 9.4.50 Close-up View of Box Girder Top Flange Joint with Epoxy Joint Material

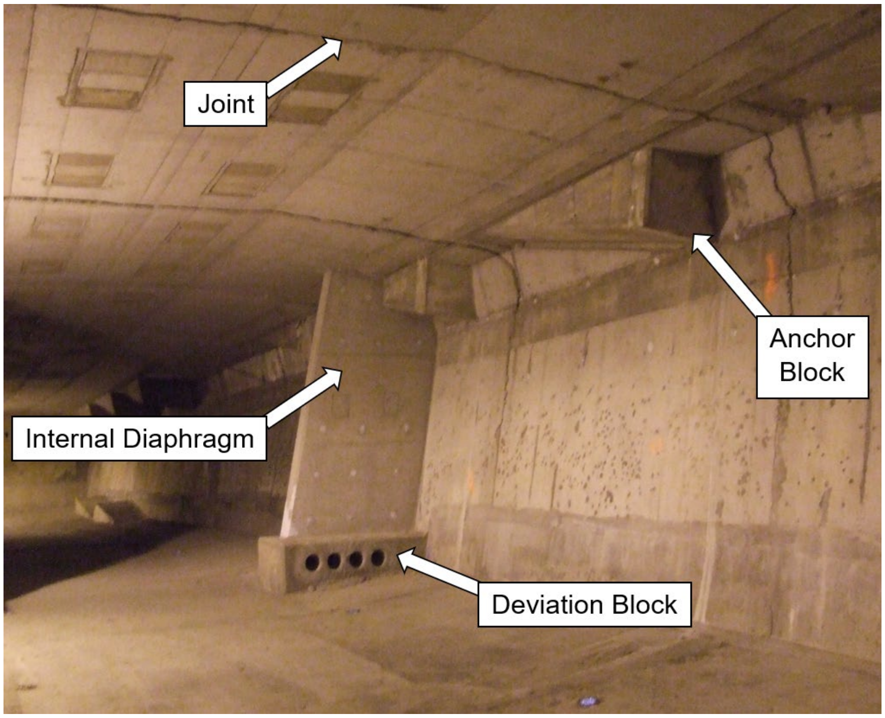


Figure 9.4.51 Box Girder Joint Between Segments Shear Keys and Deviation Block

9.4.11 Miscellaneous Areas

Surface Irregularities

Form marks caused by burlap folds used in the old vacuum curing process should be inspected. This dates the beam construction to the early 1950s and should alert the inspection team to possible deficiencies common in early box beams, such as inadequate or non-existent drainage openings and strand cover.

Unintentional Load Path

Older cast-in-place box girder interiors should be inspected to verify that inside forms left in place do not provide unintentional load paths, which may result in overloading members of the concrete box (see Figure 9.4.52). Loads from the deck may be directly transferred to the bottom flange. This was not the intent of the original design and this bottom flange may now be overstressed.

Figure 9.4.52 Interior Formwork Left in Place

Structure Alignment

An engineering survey should be performed at the completion of construction and a schedule for future surveys established. The results of these surveys should aid the bridge engineer in assessing the behavior and performance of the bridge. Inspectors should establish permanent survey points at each substructure and at each mid-span. Likewise, several points should be set at each of these locations in the transverse direction across the top flange (see Figure 9.4.53). During the inspection:

- The girder should be inspected for the proper camber by sighting along the fascia of the bottom flange.
- On curved box girders, irregularities in the superelevation of the flanges, could indicate torsional distress, and should be checked.

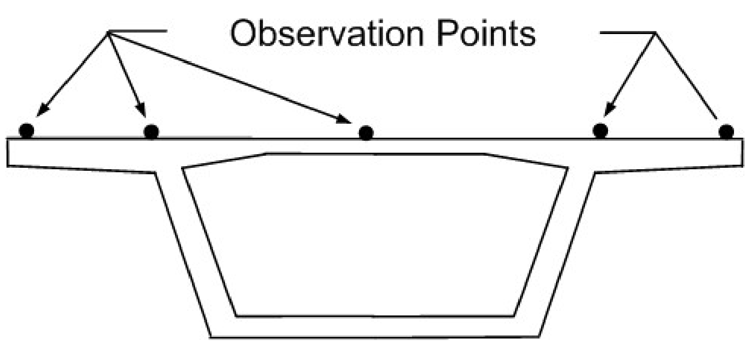


Figure 9.4.53 Location of Observation Points Across Box Girder Top Flange

Along Tendon Profile

Post-tensioning tendons can be aligned vertically, horizontally, 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 members that contain these tendons along the tendon's profile. 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.

Unusual noises should be investigated, 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 have been installed on or within the bridge.

9.4.12 Secondary Members

Secondary members typically span between primary members. Some examples include arch struts and diaphragms spanning between conventionally reinforced tee beam members, prestressed AASHTO wide flange beams, or arch struts between arches. Diaphragms are at the ends of spans or at the half or third points in a span. (see Figure 9.4.54 through Figure 9.4.56).

Figure 9.4.54 Inspection of Arch Strut (Secondary Member)

Figure 9.4.55 End Diaphragm (Secondary Member)

Figure 9.4.56 Intermediate Diaphragm (Secondary Member)

Diaphragms should be inspected 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 primary superstructure members.

Section 9.5 Lessons Learned

9.5.1 Lake View Drive Bridge over Interstate I-70

General

Adjacent box beam bridges in service that are not considered composite have unique characteristics that are to be considered during in-service inspection. The following case history illustrates the potential for hidden deterioration. When combined with other factors, these deficiencies can lead to failure. In December 2005, the fascia beam of the Lake View Drive Bridge over Interstate 70 in Southwestern Pennsylvania failed near mid-span (see Figure 9.5.1). A forensic investigative study conducted by Lehigh University revealed a number of factors that contributed to the failure. The information in this section comes from the investigation report *Forensic Evaluation of Prestressed Box Beams from the Lake View Drive over I-70 Bridge (FHWA-PA-2006-017-EMG002 / ATLSS Report 06-13).*

Figure 9.5.1 View Northeast of I-70 EB from Beneath Span 3

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 composite 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.

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 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 in the bottom flange and into the webs (see Figure 9.5.2).

Collapsed Beam Findings

Samples were taken from both the failed beam and adjacent beams after the collapse. Material testing confirmed that the strengths of both the concrete and prestressing steel were greater than the design values. 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.

Documented collision damage on the fascia beam existed near the point of fracture and resulted in several bottom strands being severed 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 39 strands in Beam 1 failed primarily due to environmental corrosion, 19 of which were not visible (see Figure 9.5.2). 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 9.5.3). Corrosion was also simultaneously transferred laterally to adjacent strands by corroding shear reinforcement stirrups (see Figure 9.5.4). 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. Strand corrosion resulted in a direct reduction in structural capacity.

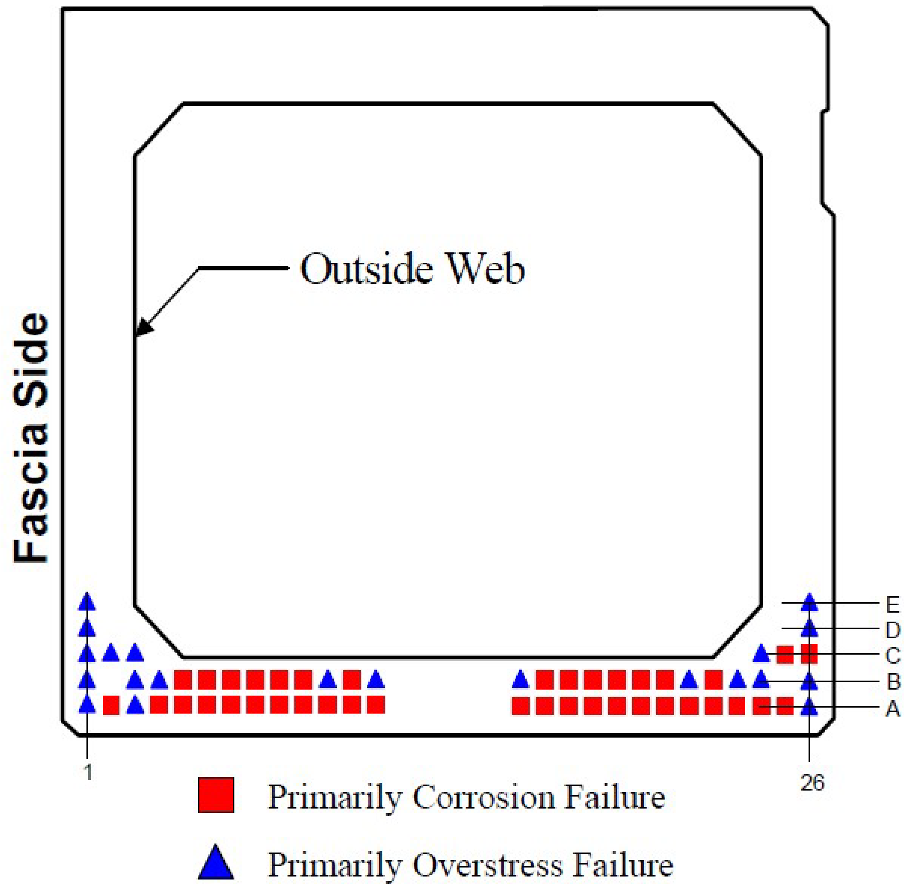


Figure 9.5.2 **Beam 1 Post-Collapse Prestressing Strand Wire Fracture Assessment**

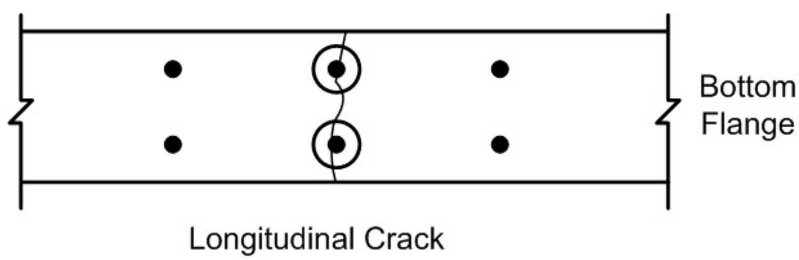


Figure 9.5.3 Prestressing Strands Corroded due to Crack Location

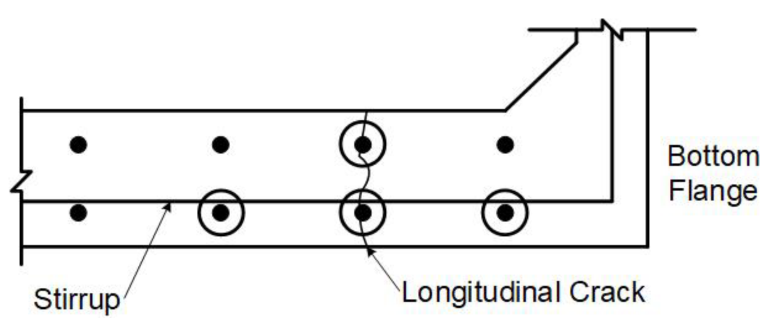


Figure 9.5.4 Adjacent Prestressing Strands and Shear Reinforcement Corrosion

In the past, longitudinal cracks were not considered a significant deficiency by PennDOT since they are parallel to the primary reinforcing strands. This Post Collapse Study showed that one crack, thought to be insignificant, led to up to four strands with environmental corrosion and section loss (see Figure 9.5.5).

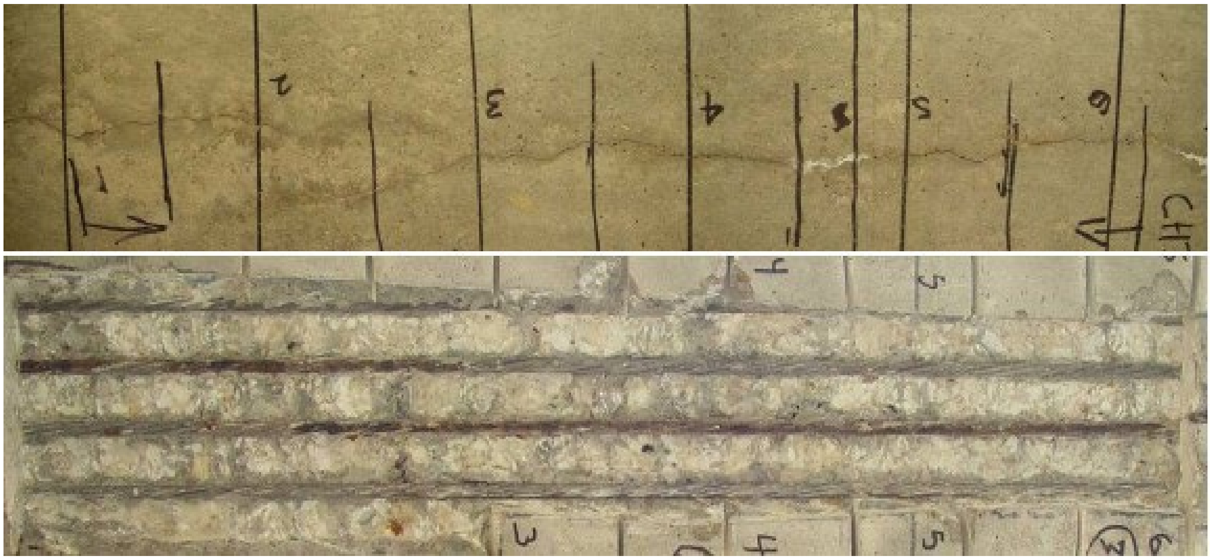


Figure 9.5.5 Longitudinal Cracks (Top) and Corresponding Corroded Strands (Bottom)

Unforeseen Fabrication Problems

Unforeseen fabrication problems also contributed to the Lake View Drive Bridge failure. The Post Collapse Study found:

- The minimum bottom flange thickness was found to be 86 percent of the design thickness.
- The average concrete cover over the strands was measured 0.87 inch average compared to 1- 9/16 inches as noted in the design drawings.
- The minimum wall thickness was measured to be 66 percent of the design value. The study concluded that the cardboard formwork likely 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 clogged, resulting in an accumulation of moisture, and even standing water.

Inspectors could not determine these fabrication problems without extensive use of advanced testing methods. Prestressed concrete beams are currently constructed to tighter tolerances than 1960 (the date of fabrication for the beams in the Lake View Drive Bridge).

Summary

In response to the findings from the study, as outlined in *Forensic Evaluation of Prestressed Box Beams from the Lake View Drive over I-70 Bridge (FHWA-PA-2006-017-EMG002 / ATLSS Report 06-13)*, a revised and more detailed set of inspection methods and State condition rating assessment were developed by PennDOT and incorporated into the Pennsylvania Department of Transportation's Publication 238 *Bridge Safety Inspection Manual* for all adjacent box beam bridges that are not considered composite. The condition of only one prestressed adjacent box beam is now considered in the bridge's evaluation.

The barrier joint location should be the focus of inspection 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. Deflection joints in the parapet were closed to prevent water leakage on the beam fascia.

Examination of fascia beams should be observed for the presence of cracking at drainage locations where runoff can come in contact with the sides and bottoms of the beams.

The effects of any longitudinal cracking in bottom strand locations, likely resulting in corrosion of non-visible strands including those above the bottom row, should be accounted for in the condition assessment of adjacent box beam bridges that are not considered to be composite.

Section 9.6 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of concrete bridges. The two major rating systems, that are used to collect data for the National Bridge Inventory currently in use are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI)* for component **condition level** rating (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

This page intentionally left blank.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 10 TABLE OF CONTENTS

CHAPTER 10 LIST OF FIGURES

Chapter 10 Inspection and Evaluation of Steel Superstructures

Section 10.1 Characteristics

10.1.1 Introduction

There are a wide variety of steel bridge superstructures that may be selected based on span length and site conditions. This section outlines the design characteristics of the different structure types and details the typical configuration of each type. It also includes a discussion of some members that are commonly utilized in multiple types of steel bridges. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

10.1.2 Rolled Beams

Design Characteristics

Rolled steel beams are one of the two basic steel superstructure types. Although many designers and inspectors use the terms “beam” and “girder” interchangeably, there is a noted difference. In steel fabrication, the word “beam” refers to rolled shapes. The word “girder” refers to fabricated members. Girders are fabricated from web and flange plates, which are connected by welding, riveting, or bolting.

A steel rolled beam bridge is a configuration of parallel rolled beams with a deck placed on top of the beams. A separate floor system is not present for this type of bridge. The most common use of this superstructure type is for simple spans, with typical span lengths from 30 to 50 ft (see Figure 10.1.1). Continuous span designs have also been used, some of which, on rare occasions, incorporate pin-and-hanger connections (see Figure 10.1.2).

Figure 10.1.1 Simple Span Rolled Beam Bridge

Figure 10.1.2 Continuous Span Rolled Beam Bridge with Pin and Hanger

Rolled beams are generally compact sections that satisfy ratios for the flange and web thicknesses to help prevent buckling. Rolled beams come in various 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. 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). In the past, rolled beams with a depth greater than 36 inches were not generally available, but are now available from some mills as deep as 44 inches. Also, rolled beams may have bearing stiffeners but typically do not include intermediate web stiffeners.

In continuous girder designs, additional girder strength is necessary 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) may have negative moments larger than the adjacent positive moments. Typically, the girder depth used in the positive moment region cannot sufficiently resist the negative moment, so the web depth needs to be increased (refer to Chapter 6). Instead of increasing the depth for the full length of the girder, the girder is haunched at the intermediate supports. To haunch a rolled beam, the bottom flange is separated from the web and an insert plate of the needed depth is welded in place (see Figure 10.1.3).

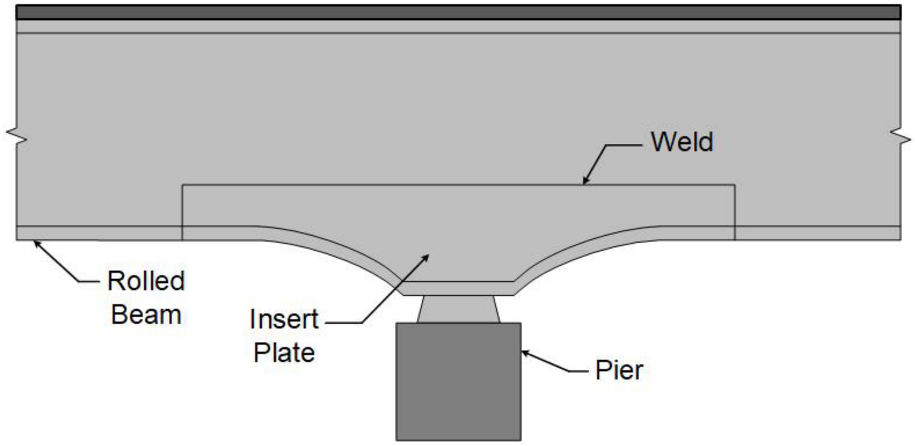


Figure 10.1.3 Web Insert Plate for Rolled Beam Bridge

Historically, another common method of economically increasing the capacity of a rolled beam bridge was to weld partial length cover plates to the flanges (see Figure 10.1.4). The cover plates increased a beam's bending strength, but this practice created a fatigue-prone detail in the tension flange. Since the cover plates were attached by welding, fatigue cracking has been found to occur in the beam flanges at the ends of partial length cover plates.

Figure 10.1.4 Rolled Beam Bridge with a Partial Length Cover Plate

Primary Members

Primary members are designed to resist live loads from trucks and dead loads. The primary superstructure members in a rolled beam bridge are the beams themselves. The beams carry all primary loadings to the substructure. Secondary members for steel bridges are described in Section 10.1.14.

10.1.3 Plate Girders

Design Characteristics

Plate girders have similar characteristics to rolled beams; however, there are some primary differences that make the superstructure type unique. Plate 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. Plate girders generally have both bearing stiffeners and intermediate web stiffeners.

The plate girder bridge is like the rolled beam bridge in appearance but, plate girders are often larger than those that could be provided by the rolling mills. Older plate girders are riveted or bolted built-up members consisting of angles and plates (see Figure 10.1.5). In a riveted or bolted built-up member, the angles are considered part of the flange. Today's plate girders are usually welded plate members (see Figure 10.1.6).

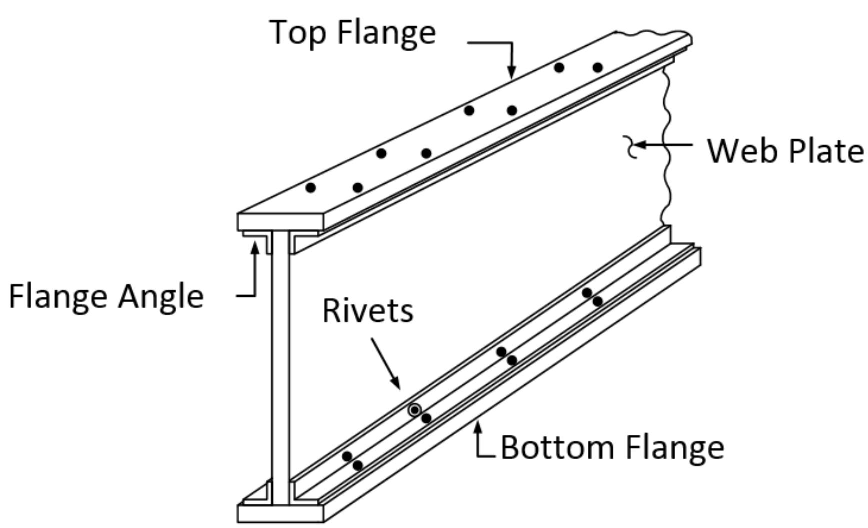


Figure 10.1.5 Built-up Riveted Plate Girder

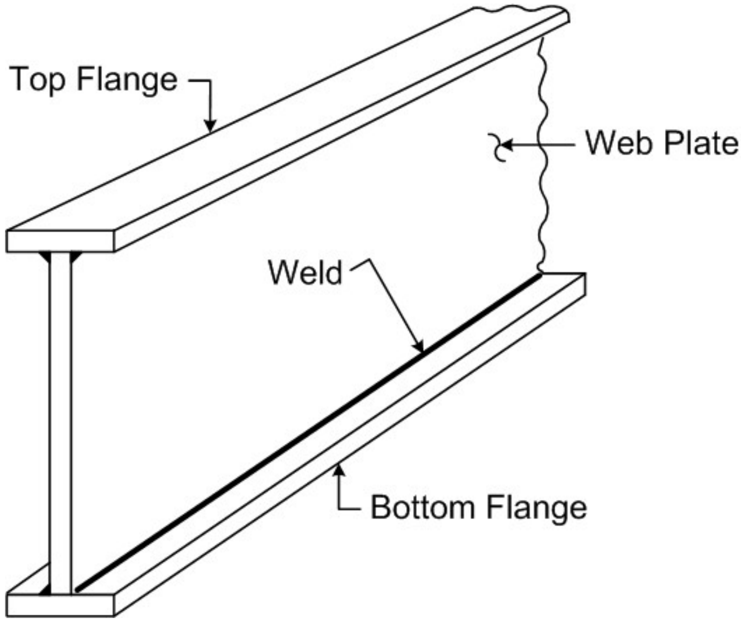


Figure 10.1.6 Built-up Welded Plate Girder

This bridge type can be found in simple and continuous span designs (see Figure 10.1.7), and it is widely used when curved bridges are needed due to site conditions, or necessary road geometry (see Figure 10.1.8). Continuous welded plate girder bridges have been built for spans of over 500 ft.

Figure 10.1.7 Continuous Span Plate Girder Bridge

Figure 10.1.8 Curved Plate Girder Bridge

Pin-and-hanger connections are found in older girder bridge construction (see Figure 10.1.9). Because pin-and hanger connections are constructed below expansion joints, this type of connection is not used in modern bridge design practices due to excessive corrosion within the assembly. The deterioration may lead to failure of the connection and girders. Pin-and-hanger assemblies are discussed in detail in Section 10.1.10.

Figure 10.1.9 Plate Girder Bridge with Pin and Hanger Connection

Sometimes both types of superstructure, rolled beams and plate girders, can be used on the same bridge (see Figure 10.1.10). The shorter approach spans are typically rolled beams and the longer main spans generally utilize plate girders.

Figure 10.1.10 Combination Rolled Beams and Plate Girders

Plate girders may also use haunched girders to increase negative moment capacity (see Figure 10.1.11). The top and bottom flange plates are welded or riveted to the variable web plate.

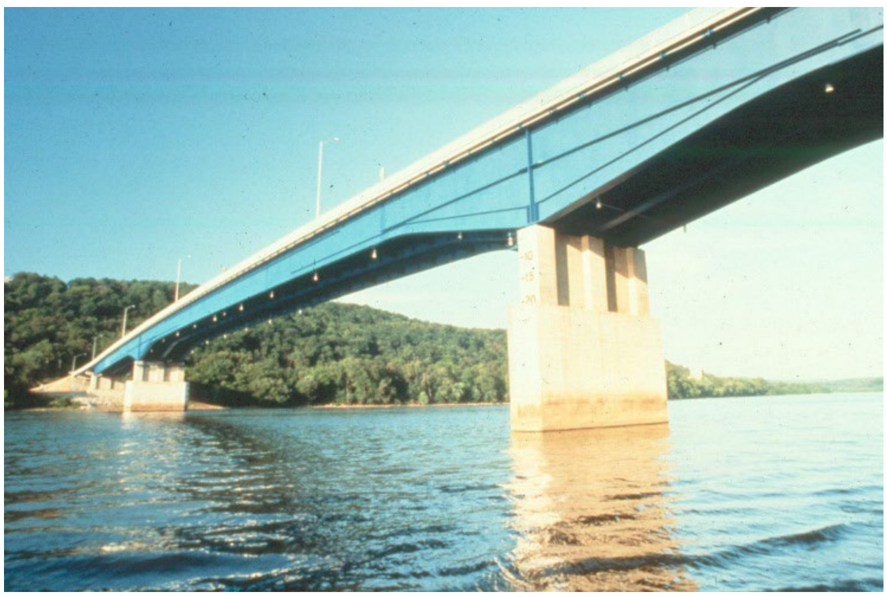


Figure 10.1.11 Fabricated Variable Depth Girder Bridge

As plate 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 transverse (vertical) or longitudinal (horizontal) (see Figure 10.1.12). 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.

Figure 10.1.12 Primary Members with Welded and Riveted Web Stiffeners

Primary Members

The primary members of a plate girder bridge are the fabricated girders, as well as the diaphragms and cross-frames on a curved bridge. On curved bridges, the cross-frames carry the torsional load between the girders and are therefore primary members. Full depth cross-frames are most often used with curved girders instead of diaphragms (see Figure 10.1.13). On straight girder bridges, diaphragms, cross-frames, and lateral bracing are considered secondary members. Secondary members for steel bridges are described in Section 10.1.14.

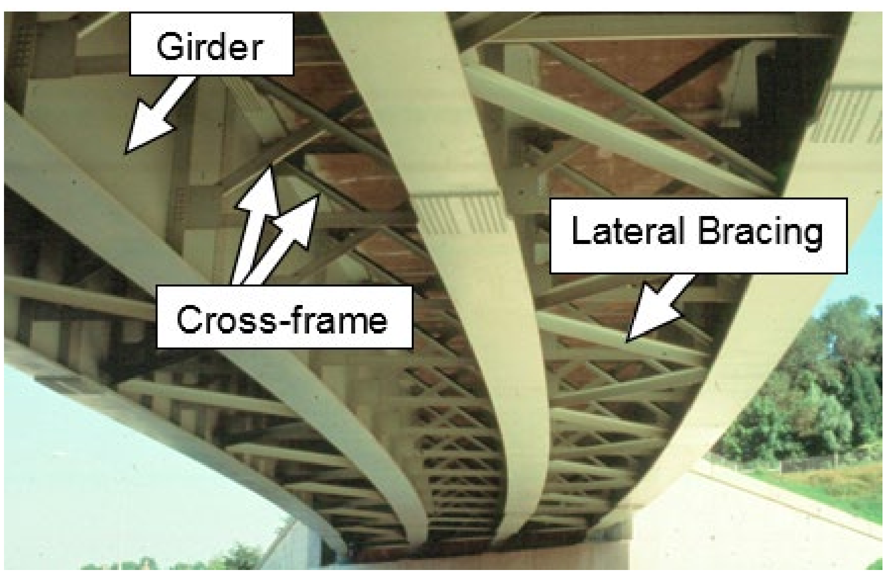


Figure 10.1.13 Curved Girder Bridge

10.1.4 Box Girders

Design Characteristics

A box girder bridge is supported by one or more steel box girder members. Steel box webs and flanges 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. Box girder bridges are generally used in simple spans of 75 ft or more (see Figure 10.1.14) and in continuous spans of 100 ft or more. They are frequently used for curved bridges due to their torsional rigidity (see Figure 10.1.15).

Figure 10.1.14 Simple Span Box Girder Bridge

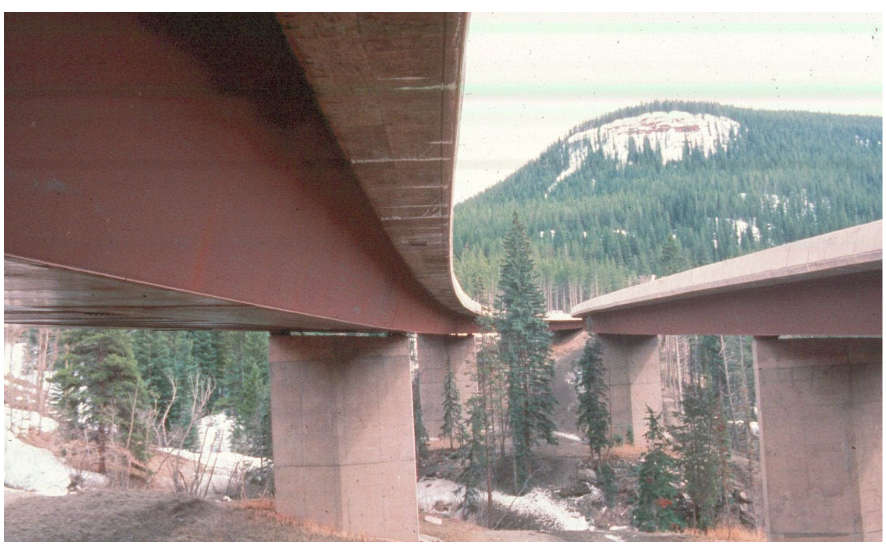


Figure 10.1.15 Curved Box Girder Bridge

A box girder bridge can use a single box configuration (see Figure 10.1.16) or have multiple (spread) boxes in its cross section (see Figure 10.1.17). Several factors such as deck width, span length, terrain, and even aesthetics can all play a role in determining which configuration is used.

Figure 10.1.16 Single Box Girders

Figure 10.1.17 Spread Box Girders

The webs and bottom flange of large box girders may be stiffened in areas of high stress. This is accomplished in part by stiffeners inside the box member. Similar to plate girder bridges, the stiffeners are designed to help the webs and bottom flange resist buckling due to compression from bending, torsional and shear forces. The stiffeners limit the unsupported length of the web and bottom flange, which results in increased stability of the web and bottom flange plates. Box girders may also incorporate diaphragm, cross-frame, and top flange lateral bracing systems. External diaphragms and cross-frames may be used between box girders (see Figure 10.1.18 and 10.1.14).

Box girders typically have an opening or access door to allow the bridge inspection team to examine the inside of the box (see Figure 10.1.19). Box girders are typically considered confined spaces. Refer to Chapter 2 for explanation of confined spaces and inspection needs.

Figure 10.1.18 External Diaphragm between Spread Steel Box Girders

Figure 10.1.19 Inspection Access Door in a Box Girder

Box girder bridges may have nonredundant steel tension members (NSTMs) depending on the number of box girders in the span. If the span has one box girder, and often when there are only two box girders, then the span is nonredundant and the box girders are NSTMs. Structural analysis can determine if, and where, box girders are NSTMs on a bridge. Refer to Chapter 7 for additional information on NSTMs and fracture.

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. (see Figure 10.1.20).

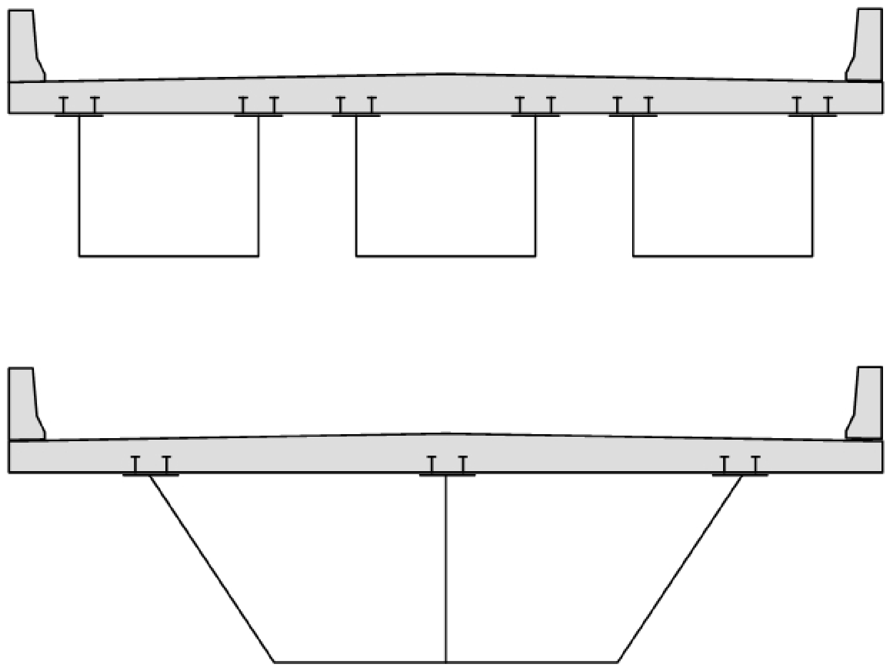


Figure 10.1.20 Box Girder Cross Sections with Composite Deck

Primary Members

The primary members of a box girder bridge are the box girders and, on a curved bridge, the diaphragms, and cross-frames. On a straight bridge, the diaphragms and cross-frames are secondary members. Secondary members for steel bridges are described in Section 10.1.14.

10.1.5 Girder-Floorbeam System Bridges

Design Characteristics

Girder-floorbeam bridges are typically constructed with two plate girders. Shorter spans may utilize rolled members. However, unlike the rolled beam or plate girder bridges, the girder-floorbeam bridge typically has only two main girders and a floor system of transverse floorbeams and smaller longitudinal stringers (if present) between them. The floor system supports the deck and the girders support the floor system. Reference Section 10.1.9 for a detailed description of floor systems.

Girder-floorbeam 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 girder-floorbeam system bridges. Girder-floorbeam bridges are classified as deck girder or through girder systems. In a deck girder system, the deck is supported by the floor system and top flanges of the main girders (see Figure 10.1.21). Floor systems are found not only in deck girder and through girder systems, but in other bridge types as well.

Figure 10.1.21 General View of a Deck Girder Bridge

In a through girder system, the deck is supported by the floor system between the girders, which extend above the deck (see Figure 10.1.22). The portion of the girder that is above the deck also serves as the bridge railing.

Figure 10.1.22 Through Girder Bridge

Though few through girders are constructed today, they were commonly used prior to the early 1950s. Since many through girder bridges were constructed in the 1940s and 1950s, they are commonly riveted. Their most common use was where vertical underclearance was a concern, such as over railroads (see Figure 10.1.23).

Figure 10.1.23 Through Girder Bridge with Limited Underclearance

A rare type of through girder has three or more girders, with the main girders separating the traffic lanes (see Figure 10.1.24). These structures are most likely converted railroad or trolley bridges.

Figure 10.1.24 Through Girder Bridge with Three Girders

Primary Members

The primary members of a girder-floorbeam system bridge are the girders, floorbeams, and also stringers if present. Secondary members for steel bridges are described in Section 10.1.14.

10.1.6 Trusses

Design Characteristics

Metal truss bridges have been built since the early 1800s. 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 10.1.25). The original metal trusses were made of wrought iron, then cast iron, and now 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. Due to higher labor costs, more truss spans are fabricated in the shop instead of assembling the individual members at the site. This fact, in addition to a propensity for NSTMs and fatigue-prone details, susceptibility to debris collection, and corrosion problems, has limited the use of trusses to major river crossings.

The superstructure of a truss bridge usually consists of two parallel trusses but can have more. The trusses are the main load-carrying members on the bridge and support the floor system that spans between the truss lines. Reference Section 10.1.9 for more details on floor systems. There are three types of trusses, grouped according to their position relative to the bridge deck (see Figure 10.1.26).

Figure 10.1.25 Simple Span Truss

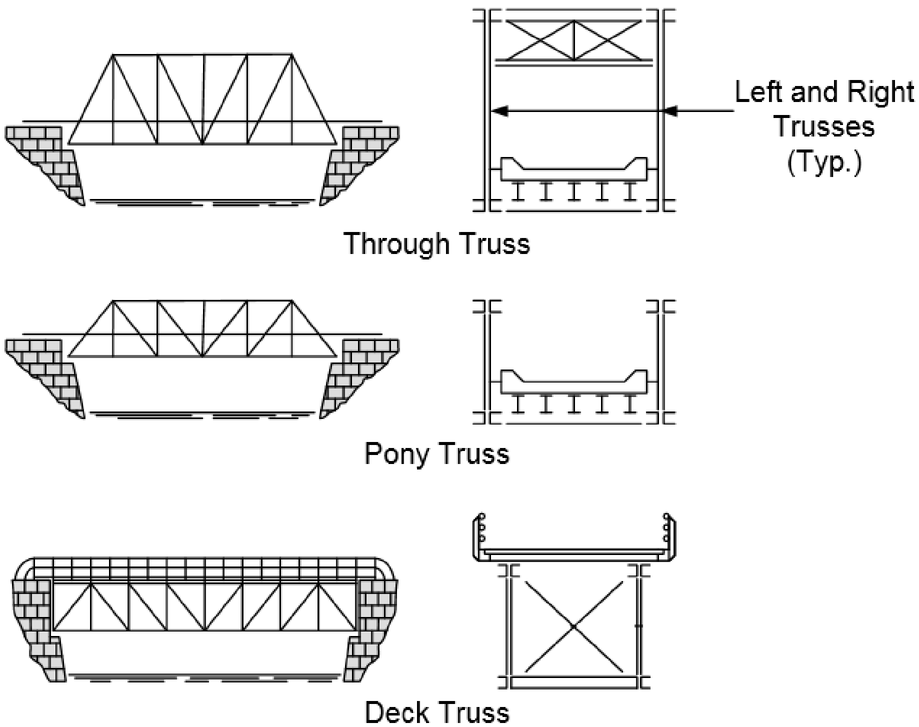


Figure 10.1.26 Through, Pony, Deck Truss Comparison

In a through truss, the roadway is placed between the trusses (see Figure 10.1.27). Through trusses are constructed when underclearance is limited.

Figure 10.1.27 Through Truss

A pony or "half-through" truss has no overhead bracing members connecting the two trusses (see Figure 10.1.28). 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 rolled beam or plate girder bridges.

Figure 10.1.28 Pony Truss

On a deck truss, the roadway is placed on top of the trusses (see Figure 10.1.29). Deck trusses have unrestricted horizontal clearances and can readily be widened. For these reasons, they tend to be preferred over through trusses when underclearance is not a concern.

Figure 10.1.29 Deck Truss

Trusses, in most applications, are primary load-carrying members. However, they can also be used as members in floor systems, in arches, and as stiffening trusses in suspension bridges and arch bridges (see Figure 10.1.30 and Figure 10.1.31). Trusses are also commonly used for movable bridge spans because they are lightweight and have higher overall stiffness (see Figure 10.1.32).

Figure 10.1.30 Suspension Bridge with Stiffening Truss

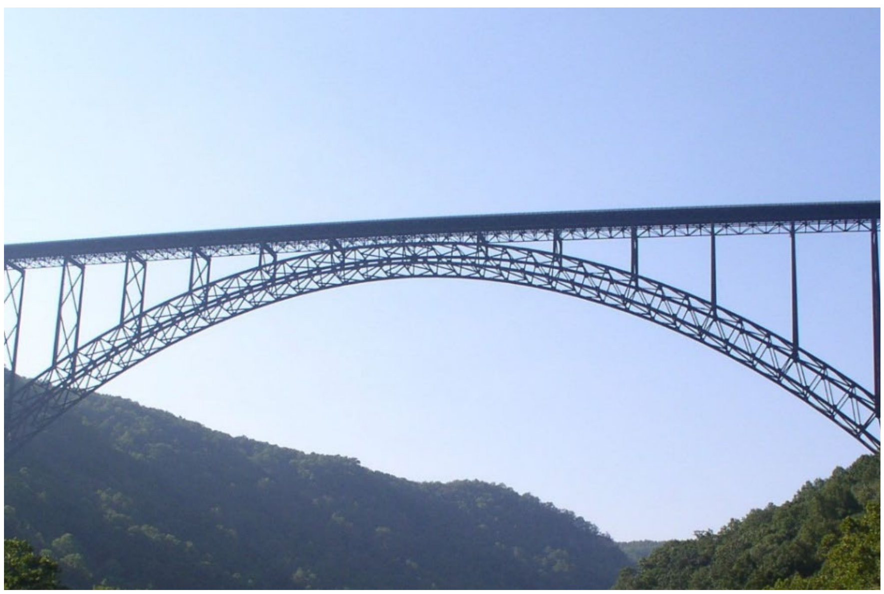


Figure 10.1.31 Deck Arch Bridge with Stiffening Truss in Main Arch

Figure 10.1.32 Vertical Lift Bridge and Approaches using Truss Members

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 10.1.33).

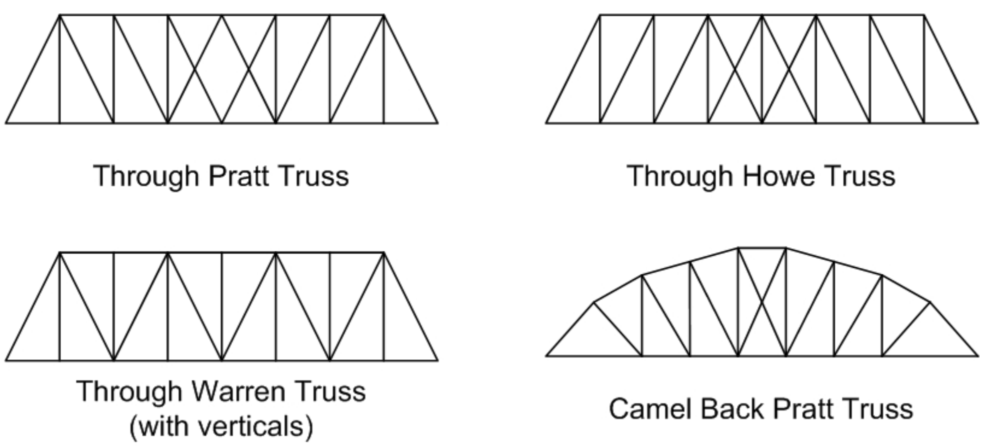


Figure 10.1.33 Various Truss Designs

Trusses have been constructed for short to very long spans, using simple and continuous designs (see Figure 10.1.34 to Figure 10.1.36).

Figure 10.1.34 Single Span Camel Back Pratt Truss

Figure 10.1.35 Multiple Simple Span Pony Truss

Figure 10.1.36 Continuous Through Truss

Cantilevered trusses often incorporate a "suspended" or "drop-in" span between two cantilevered spans (see Figure 10.1.37 and Figure 10.1.38). The suspended span behaves as a simple span and is connected to cantilevered spans with pins or pin-and-hanger connections. The back span on a cantilever truss is called the anchor span.

Figure 10.1.37 Cantilever Deck Truss

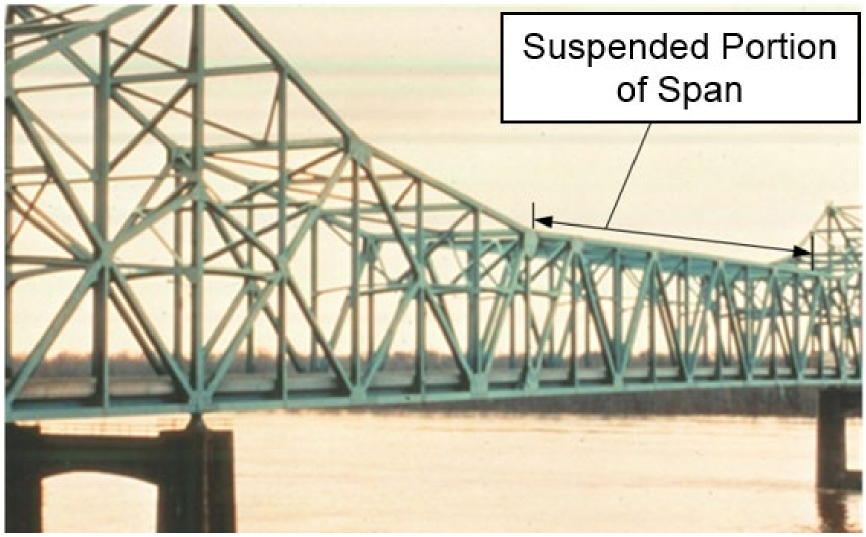


Figure 10.1.38 Cantilever Through Truss

Truss members are divided in to three groups:

- Top or upper chord members.
- Bottom or lower chord members.
- Diagonals and verticals.

See Figure 10.1.39 for truss members, floor systems, and various bracing configurations.

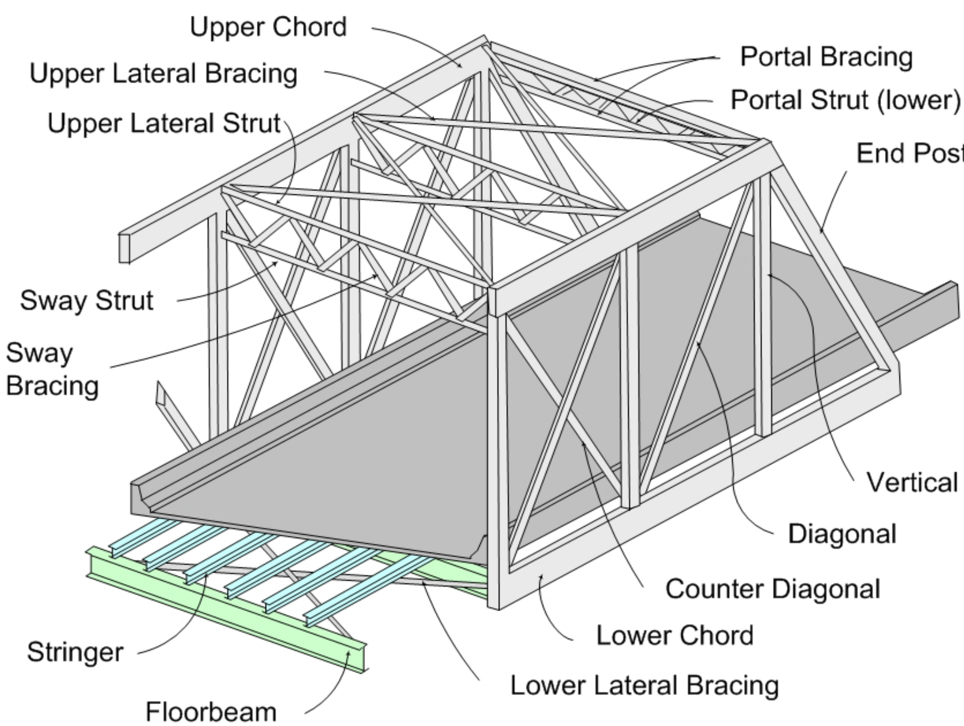


Figure 10.1.39 Truss Members, Floor System, and Bracing

Floor systems are described in detail in Section 10.1.9. Bracing (secondary members) is described in detail in Section 10.1.14.

Truss members, including eyebar plates, can be fabricated from rolled shapes (see Figure 10.1.40).

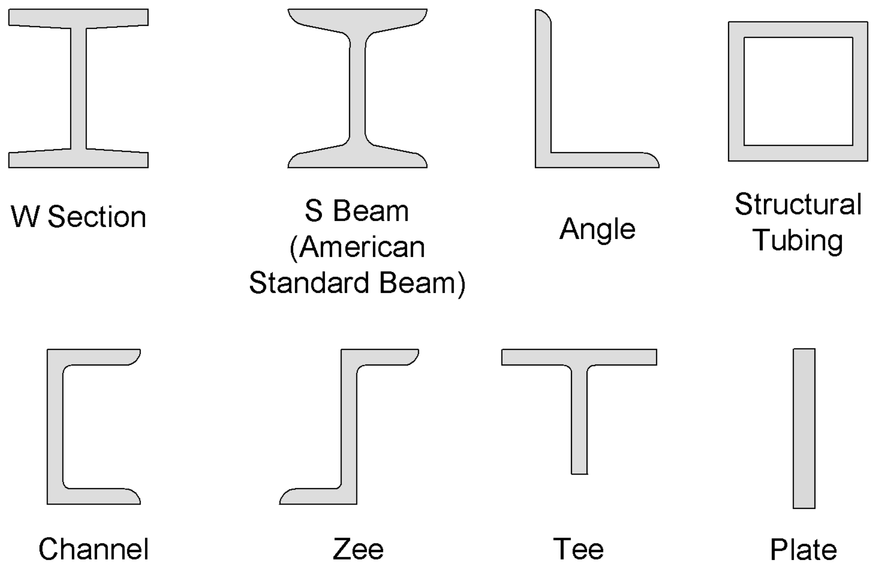


Figure 10.1.40 Rolled Steel Shapes

Trusses also utilize built-up sections. Built-up sections are fabricated by bolting, riveting, or welding rolled shapes (see Figure 10.1.41). Built-up sections can also be custom designed to be efficient for expected design loads. Built-up sections 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; however, they can be vulnerable to internal corrosion not readily visible to inspectors.

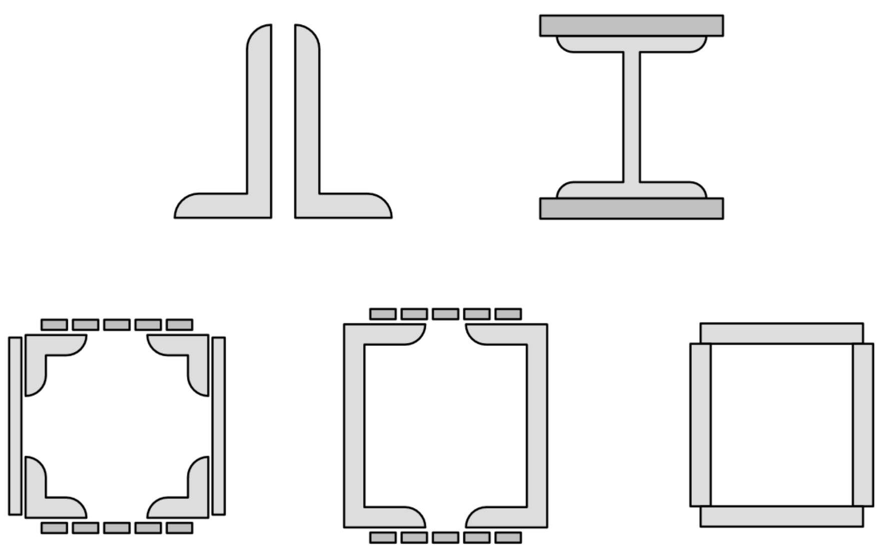


Figure 10.1.41 Built-Up Sections

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 10.1.42). On a simple span truss, the bottom chord is always in tension, and 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).

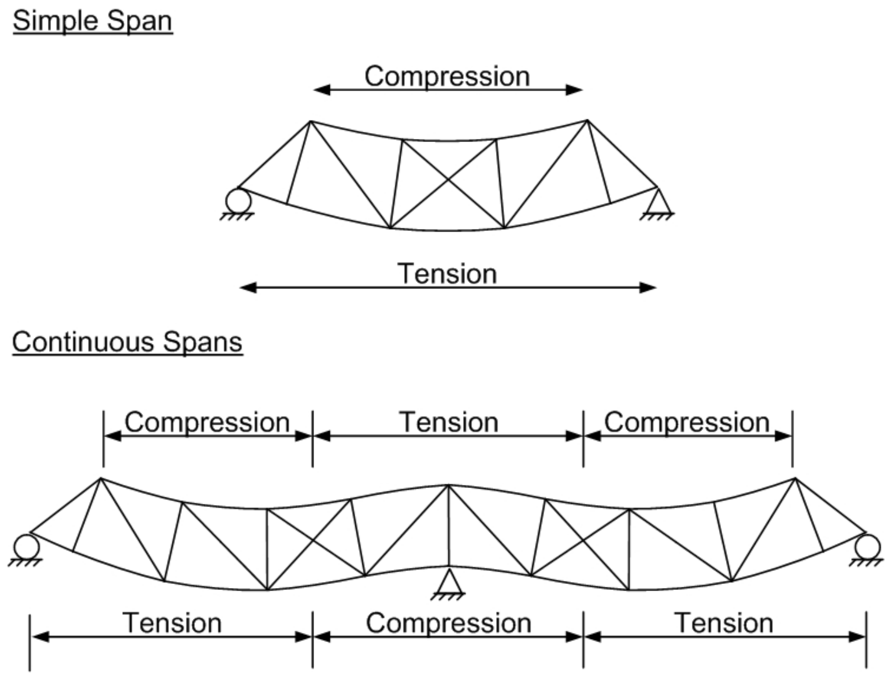


Figure 10.1.42 Axial Loads in Truss Chord Members

As truss bridge spans increase, cantilever and continuous designs are typically used, creating negative moment regions. The top chord of a truss is in tension over an intermediate support similar to the top flange on a girder (see Figure 10.1.42). It is common to find varying depth trusses on large structures, with the greatest depth at the intermediate supports where the moments are the largest (see Figure 10.1.27 and Figure 10.1.36).

Diagonal and Vertical Web Members

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

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 10.1.43). 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.

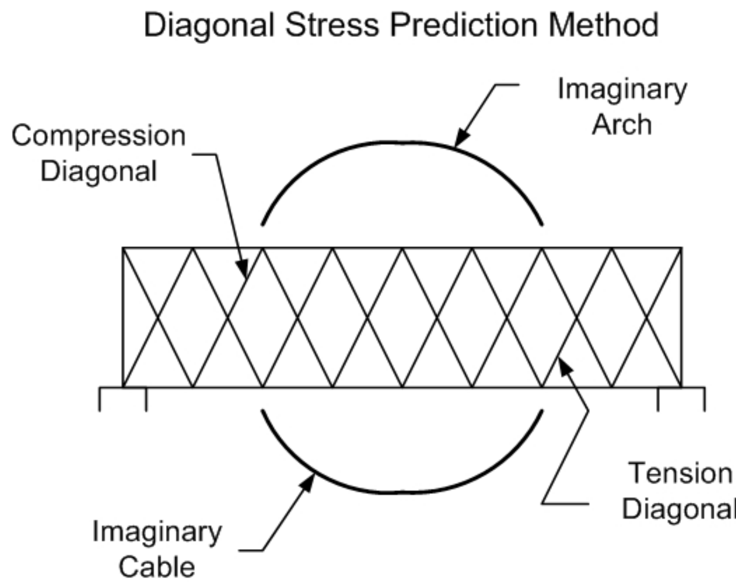


Figure 10.1.43 "Imaginary Cable – Imaginary Arch"

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 needs 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 truss 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 found 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 10.1.39 of a sample truss schematic showing diagonals in a simply supported truss.

There is 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 10.1.44). Verticals that have two diagonals at the same end are similar to the force in the diagonal closest to midspan (see Figure 10.1.45). Verticals that have counters on both ends are in compression (see Figure 10.1.46).

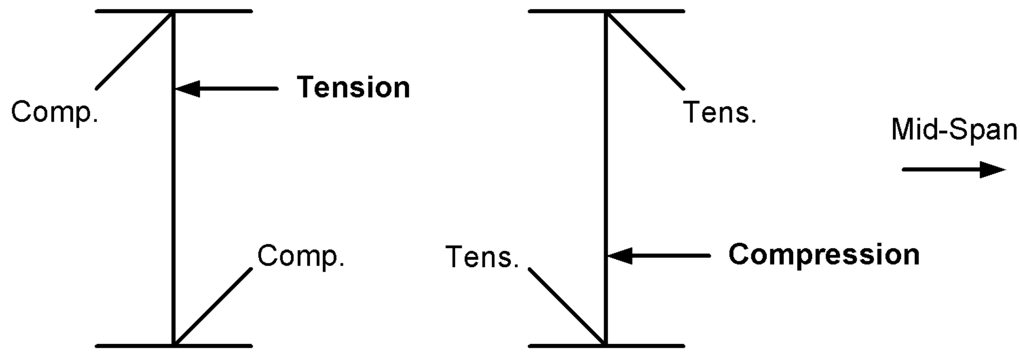


Figure 10.1.44 Vertical Member Stress Prediction Method – Opposite Diagonals

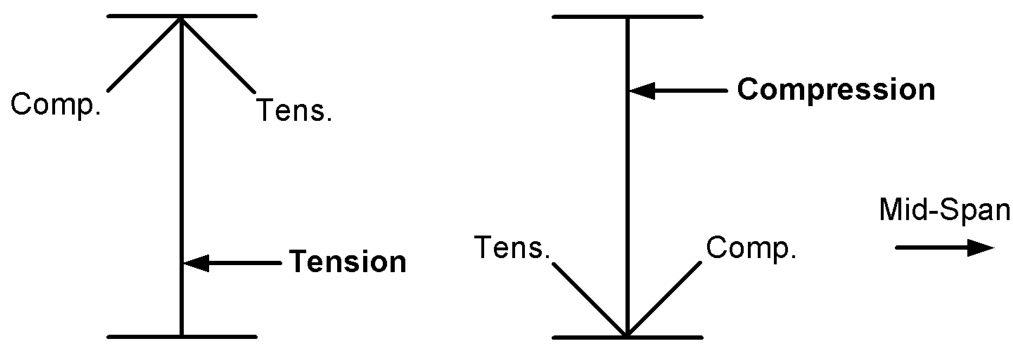


Figure 10.1.45 Vertical Member Stress Prediction Method – Diagonals Same End

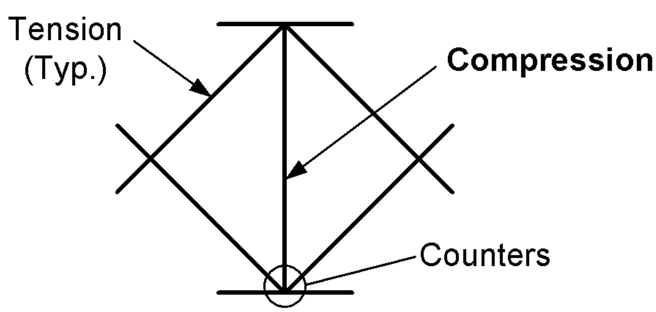


Figure 10.1.46 Vertical Member Stress Prediction Method - Counters

See Figure 10.1.39 of a sample truss schematic showing verticals in a simply supported truss.

Panel Points

A panel point is the location where the truss members are connected. 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 1950s and early 1960s, as high strength bolts began to replace rivets. Rivets were used during shop fabrication. Bolts were used to complete the connection in the field (see Figure 10.1.47). See Section 10.1.11 for detailed information on gusset plates.

Figure 10.1.47 Truss Panel Point using Rivets and Bolts

Some older trusses have pins at panel point connections (see Figure 10.1.48).

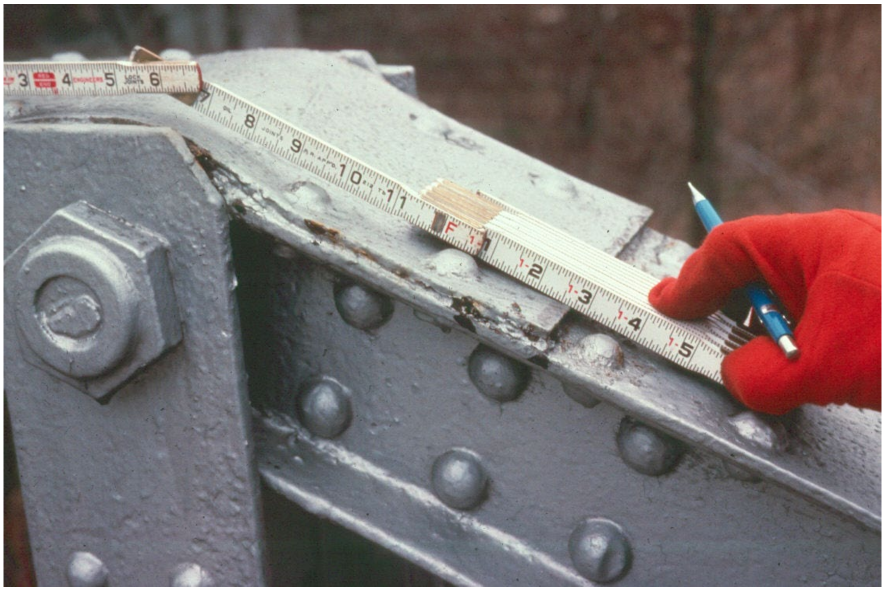


Figure 10.1.48 Pin Connected Truss

Panel points are often numbered to help orient the inspection documentation. The letter U for upper chord, the letter L for lower chord, or the letter M at a middle connection, typically designates a panel point. Additionally, the panel points are numbered from bearing to bearing, typically beginning with 0 (zero). Most trusses begin with panel point L0. Some deck trusses may begin with U0. Upper and lower panel points of the same number are always in a vertical line with each other (e.g., U6 is directly above L6) (see Figure 10.1.49 and Figure 10.1.50).

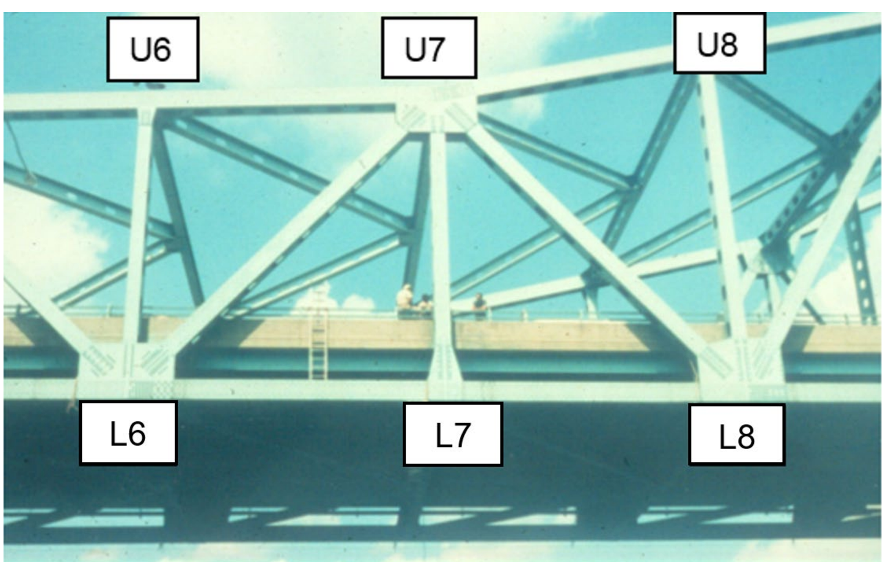


Figure 10.1.49 Through Truss Panel Point Numbering System

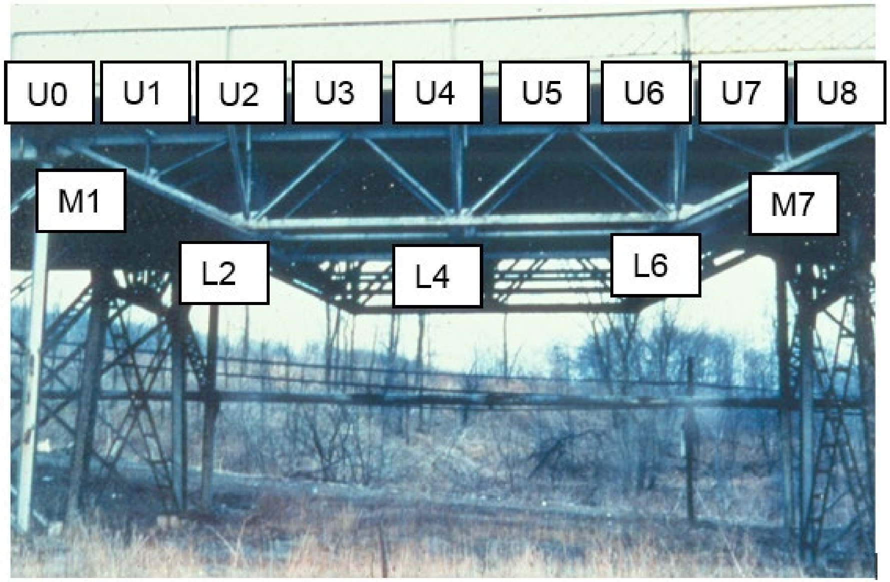


Figure 10.1.50 Deck Truss Panel Point Numbering System

A panel is the space, or horizontal distance, between panel points. Truss panels are typically 20 to 50 ft long. 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. The truss design also needed 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 can subdivide the panels. The midpoint of each diagonal can be braced with a downwardly inclined sub-diagonal in the opposite direction and with a sub-vertical down to the lower chord. For larger trusses with long compression members, the member may be braced at mid-length with a horizontal or diagonal member. 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 1870s, use this method (see Figure 10.1.51).

Figure 10.1.51 Pennsylvania Truss with Midpoint Panels

Primary Members

The primary members in a truss bridge are the truss members (chords, diagonals, verticals) and the floor system (floorbeam and stringers, if present). Gusset plates that connect primary members are also considered primary members and are described in Section 10.1.11. Secondary members are described in detail in Section 10.1.14.

10.1.7 Arches

Design Characteristics

Arches are a unique structure type, as they are longitudinal members resisting vertical loads primarily in compression. Arch bridges have been built since Roman times, but metal arch bridges have only been constructed since the late 1800s. Arch bridges generally demand strong foundations to resist the large concentrated thrust loads.

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. The steel arch is designed to resist a load combination of axial compression and bending moment.

Arches are divided into two main types: deck and through arches (see Figure 10.1.52 and Figure 10.1.53). In a deck arch, the arch members are below the roadway, whereas for a through arch, the arch members extend above the deck.

Figure 10.1.52 Deck Arch Bridge

Figure 10.1.53 Through Arch Bridge

Deck Arches

The open spandrel steel 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. The roadway is situated between the arches on a through arch bridge. The tied arch bridge internally distributes the horizontal thrust that would typically go in the substructures into the tie girder.

Open spandrel steel arches receive traffic loads through spandrel bents that support a deck and floor system. Through arches and tied arches receive traffic loads through the hangers that support the deck and floor system. Steel arches can be used in very long spans, measuring up to 1700 ft.

The arch members are called ribs and can be fabricated I-shapes, boxes, or truss shapes. The arches are classified as either solid ribbed, braced ribbed, or spandrel braced (see Figure 10.1.54, Figure 10.1.55, and Figure 10.1.56). 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 10.1.56).

Figure 10.1.54 Solid Rib Deck Arch Bridge

Figure 10.1.55 Braced Rib Deck Arch Bridge

Figure 10.1.56 Spandrel Braced Deck Arch Bridge with Six Arch Ribs

An arch with a pin at each end of the arch is called a two-hinged arch (see Figure 10.1.57). 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 rare. Foundation conditions, in part, dictate the need for hinges. Three-hinged arches, for example, are not significantly affected by small foundation settlements.

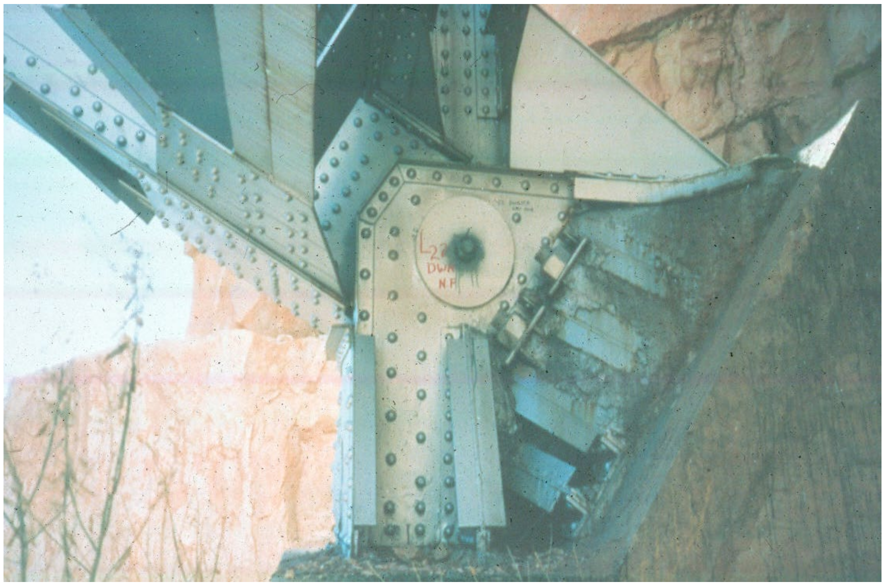


Figure 10.1.57 Hinge Pin for Spandrel Braced Deck Arch

Through Arches

Like deck arches, through arch bridges are considered simple spans because of the basic arch function and have large thrust blocks to transmit the arch reactions to the foundations. Through arches typically utilize two or three hinged systems.

The through arch is constructed with the crown of the arch above the roadway (see Figure 10.1.58). The deck is hung from the arch by wire rope cables or eyebars in the through arch portion of the bridge. In a true or tied arch, all the load from deck will be transferred via suspenders. A modified through arch has a spandrel at the end and the load is transferred to the arch by spandrel columns (see Figure 10.1.59).

Figure 10.1.58 Braced Rib Through Arch Bridge

Figure 10.1.59 Modified Through Arch Bridge

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 has a single curve defining the arch shape. 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.

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 that connects the ends of the arch, like the string on an archer bow (see Figure 10.1.60). 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 ends of the arch are no longer supported horizontally, causing the arch to collapse.

Design plans are generally necessary 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 10.1.60 Three-Span Tied Arch Bridge

The tie member is a fabricated I or box member or consists of truss members. The tie is supported by hangers, which usually consist of wire rope cable, but can also be eyebars or built-up members.

Primary 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. Any gusset plates connecting primary members are also considered to be primary. The floorbeams and stringers (if present) are considered primary members (see Figure 10.1.61). Floor systems are described in Section 10.1.9.

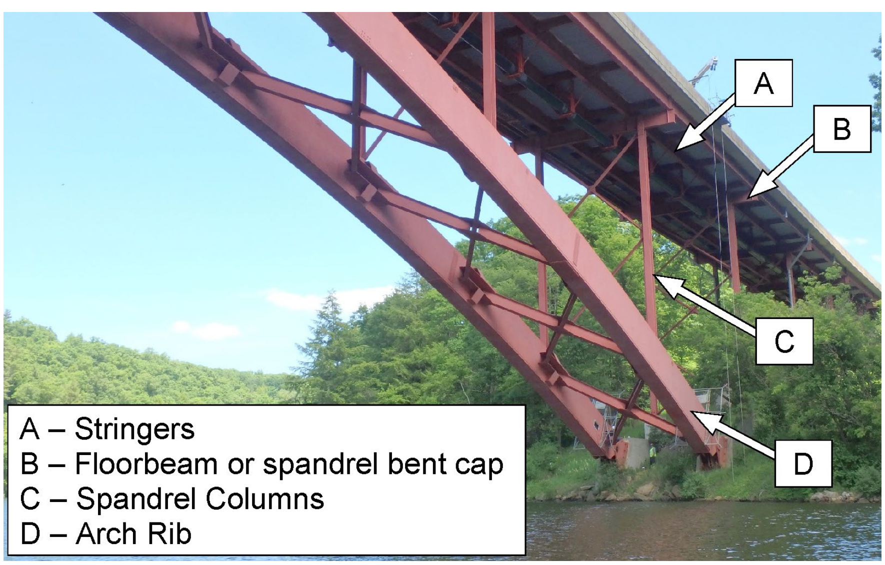


Figure 10.1.61 Solid Rib Deck Arch Primary Members

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.

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 (including any truss webbing in a braced rib), hangers and floor system including floorbeams and stringers (if present) (see Figure 10.1.62).

In the same manner as a deck arch, the traffic loads are supported by the deck and the load from the deck is transmitted to the floor system. The stringers and floorbeams resist the load in bending and shear. The load is then 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.

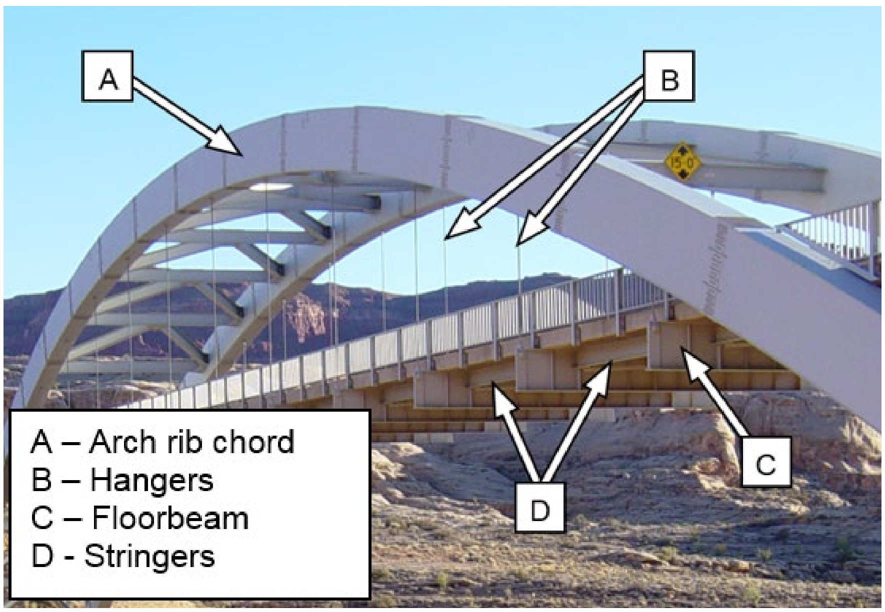


Figure 10.1.62 Through Arch Primary Members

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 10.1.63).

In a tied arch bridge, the load transfer acts the same as a through arch, except that the tie girder also carries the horizontal thrust from the arch.

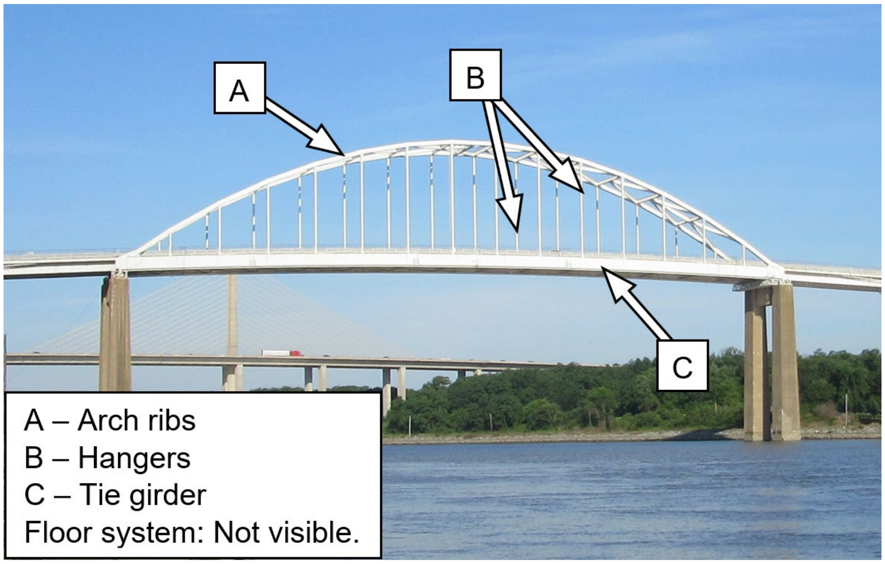


Figure 10.1.63 Tied Arch Primary Members

The secondary members for arch bridges consist of sway bracing and lateral bracing and are detailed in Section 10.1.14.

10.1.8 Rigid Frames

Design Characteristics

A rigid frame consists of horizontal members rigidly attached to vertical or inclined members without the use of bearings (see Figure 10.1.64). In a steel rigid frame bridge structure, the frame sides or "legs" replace intermediate supports. Because the legs contribute to the structure's overall capacity, increased span lengths and material savings can be realized.

Figure 10.1.64 Rigid K-frame Bridge Constructed of Two Frames

The superstructure of a rigid frame bridge can be constructed of two frames similar to a two-girder bridge (see Figure 10.1.64) or of multiple frames in the same manner as a rolled beam or plate girder bridge (see Figure 10.1.65). These frames can be thought of as fabricated plate girders with attached legs.

Figure 10.1.65 Rigid Frame Bridge Constructed of Multiple Frames

Rigid frames are not referred to as having a single, simple, multiple, or continuous spans. Horizontally curved steel rigid frames are not common for bridge construction.

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 approximately 7 to 20 ft on centers, depending on loads, span lengths, and type of floor system. Steel rigid frames can be economical for spans from 50 ft to over 200 ft. Standard abutments and expansion bearings support the ends of the frame girders. Many rigid frames have a floor system (reference Section 10.1.8) which transfers the traffic loading to the frame girders.

Steel rigid frame bridges are multi-span structures and are commonly referred to as "K-frame" bridges (see Figure 10.1.64). 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. The selection of a rigid frame bridge is often based on aesthetics.

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 10.1.66). Since there are no bearings between the legs and girder portion of the frame, bending forces are transferred between the girder and the legs (see Figure 10.1.66).

Figure 10.1.66 Connection Between Legs and Girder Portion

Each portion of the frame resists various levels of stress due to moment and shear. Tension zones are found throughout the frames, depending on geometry (see Figure 10.1.67).

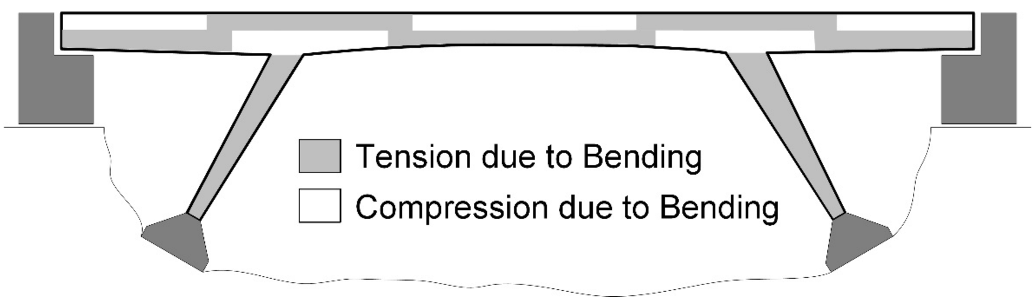


Figure 10.1.67 Stress Zones in a Rigid Frame

In some designs, a triangular frame configuration can be used. For very long spans, two K-frames can be connected end-to-end (see Figure 10.1.68). 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 and share the same bearing. This type of configuration is known as a delta frame, as in the Greek letter Delta. The leg connections form an inverted triangle with the girder portion of the frame.

Figure 10.1.68 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 the bearings which are hinges (see Figure 10.1.69).

Figure 10.1.69 Rigid Frame Bearings

Steel rigid frames may have up to three different types of stiffeners (see Figure 10.1.70).

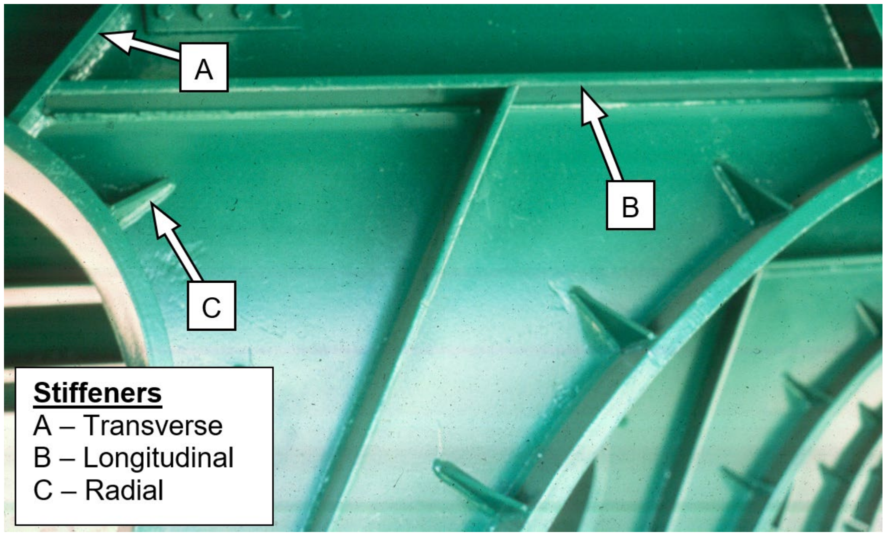


Figure 10.1.70 Transverse, Longitudinal, and Radial Stiffeners on a Frame Knee

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

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 compression. 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 are placed perpendicular along the frame knee bottom flange radius. The radial stiffeners are welded to the flange and web at designed spacings. This type of stiffener stiffens the web against radial compression forces in the knee.

Primary Members

For steel rigid frame bridges, the primary members are the frames, floorbeams, and stringers (if present). The frame is commonly broken down into the following five members:

- 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, transverse stiffeners, and diaphragms. Reference Section 10.1.14 for detailed information.

10.1.9 Floor Systems

Introduction

The following steel superstructures may use floor systems:

- Girder.
- Trusses.
- Arches.
- Frame.

Floor systems are common on multiple types of steel superstructures. They are generally utilized when there are two main load-carrying members, with the floor system positioned between them. The floor system serves to support the deck and transfer vehicle loads to the girders, trusses, arches, or frames.

Floorbeams and Stringers

There are generally two types of floor systems: the floorbeam system and the floorbeam-stringer system. The configuration varies slightly depending on the superstructure type.

The floorbeam system consists of floorbeams, which are perpendicular to traffic and connect to the superstructures at panel points. The deck is supported by the floorbeams, which in turn transmit the loads to the remaining superstructure. The floorbeams can be either rolled beams, plate girder or fabricated cross frames (see Figure 10.1.71).

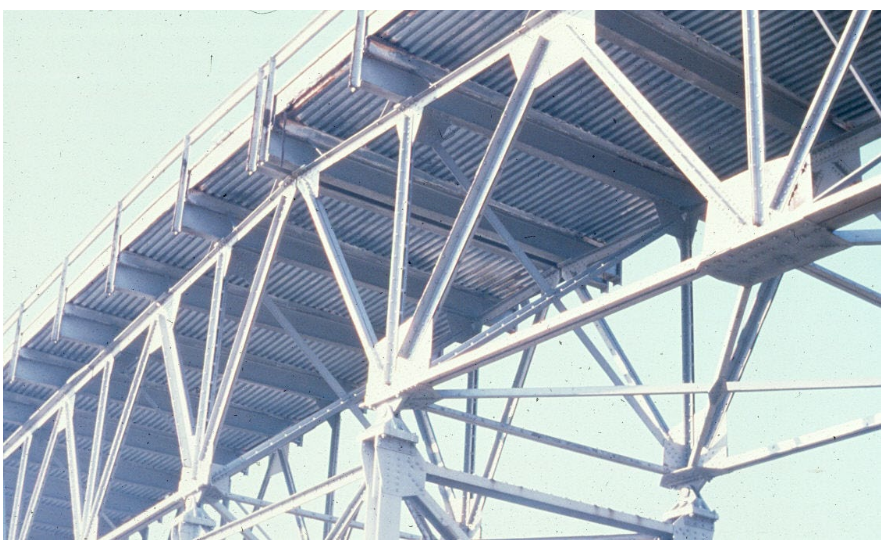


Figure 10.1.71 Floorbeam Floor System

The floorbeam-stringer system consists of floorbeams connected to the superstructure panel points and longitudinal stringers, parallel to the main superstructure members, connected to or supported by the floorbeams (see Figure 10.1.72). 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. The stacked configuration reduces fatigue-prone details that may lead to fatigue cracking but increases the overall depth of the superstructure (see Figure 10.1.73).

Floor systems (floorbeams and stringers) are subjected to bending and shear stresses. Continuous stringers help distribute the bending stresses throughout the stringer, resulting in a reduced maximum positive bending moment in the stringers between the floorbeams. Since floorbeams are often at panel points of a superstructure, the numbering of the members should coincide with the adjacent panel points (see Figure 10.1.72). Reference Section 10.1.11 for typical panel point and gusset plate configuration and naming convention.

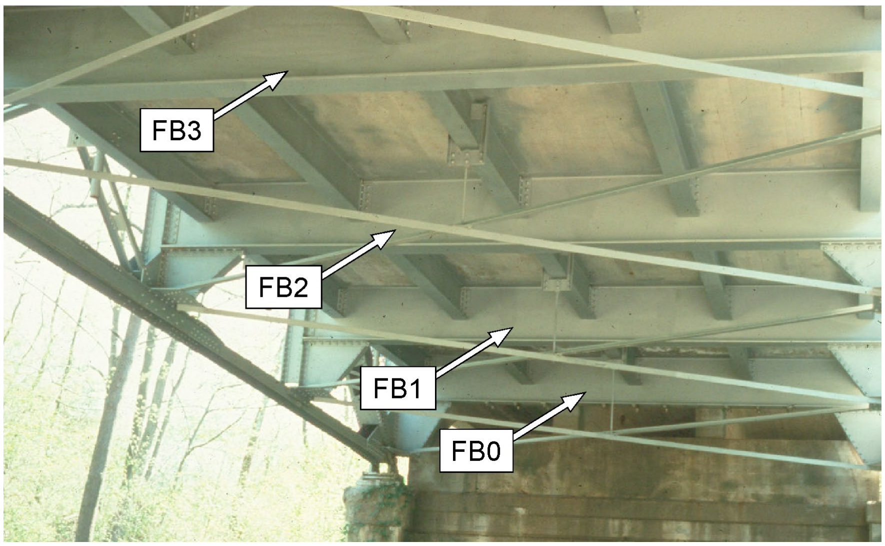


Figure 10.1.72 Floor System with Stringers Connected to Floorbeam Webs

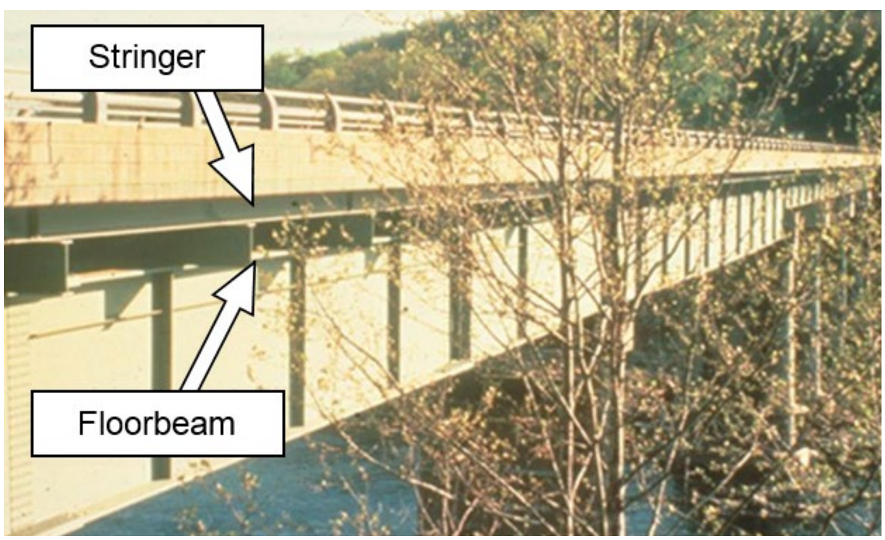


Figure 10.1.73 Floor System with Stacked Floorbeam Stringer Configuration

10.1.10 Pin and Hanger Assemblies

Design Characteristics

Pin-and-hanger assemblies are devices that utilize two pins with connecting hangers in bridges to permit longitudinal expansion movement and rotation (see Figure 10.1.74 and Figure 10.1.75).

Figure 10.1.74 Typical Pin-and-Hanger Assemblies at Locations Relative to Piers

Pin-and-hanger joints are usually only 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 10.1.74). 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.

Figure 10.1.75 Pin-and-Hanger Assembly

If only rotation of the joint is desired and not longitudinal expansion movement, one pin is used (see Figure 10.1.76).

Figure 10.1.76 Single Pin Assembly

Pin-and-hanger assemblies may experience forces that were not accounted for in the original design. The hangers, or links, are designed for axial tension forces only; (see Figure 10.1.77) however hangers see both axial tension and bending. In-plane bending results from binding on the pins due to corrosion between the pin and the hanger (see Figure 10.1.78). Out-of-plane bending (perpendicular to the wide face) results from misalignment, pack rust, skewed geometry, or improper erection.

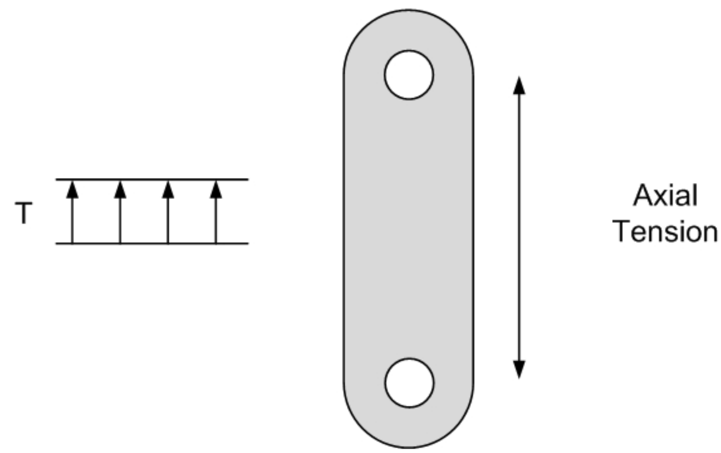


Figure 10.1.77 Design Stress in a Hanger Link (Tension Only)

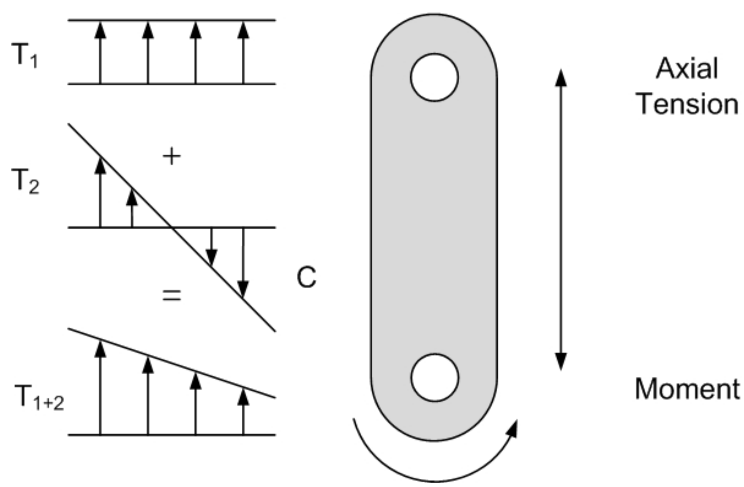


Figure 10.1.78 Actual Stress in a Hanger Link (Tension and In-Plane Bending)

Pins are designed to resist shear and bearing on the full thickness of the hanger (see Figure 10.1.79). 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 10.1.80). Section loss in the pin may cause a loss of bearing areas between the pin, the hangers, and the web. This loss can cause unsymmetrical loading which results in possible out-of-plane bending in the web and hanger. Vertical misalignment of the beams can also cause undesired out-of-plane bending in the hangers.

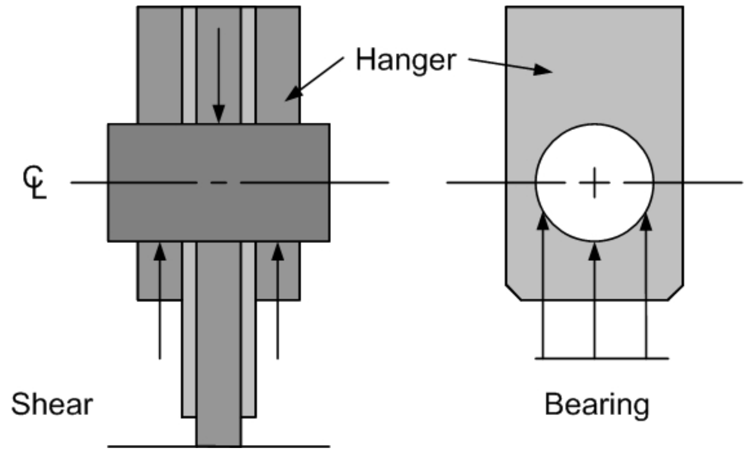


Figure 10.1.79 Design Stress in a Pin (Shear and Bearing)

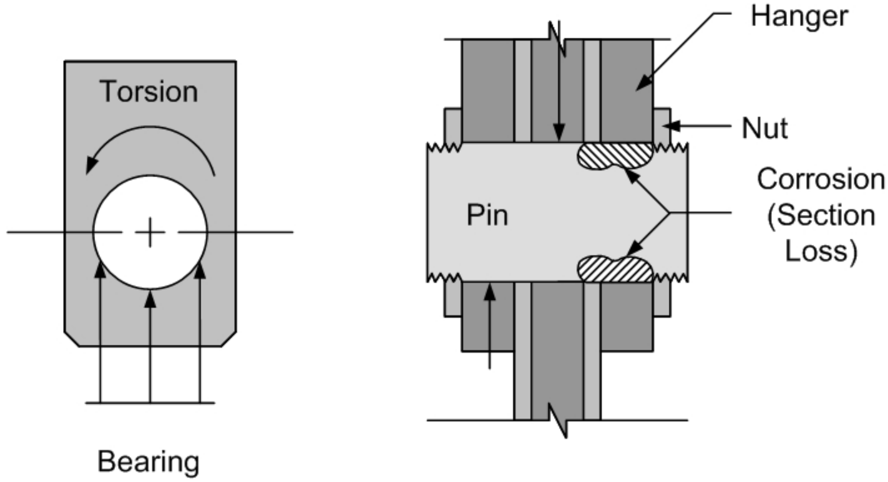


Figure 10.1.80 Actual Stress in a Pin (Shear, Bearing, and Torsion)

Although pin-and-hanger and single pin assemblies are no longer being designed, bridges with these details are still in service and will remain for the foreseeable future. Therefore, it is very important to give special observation to these details during inspection. These details are commonly retrofitted to ensure safety.

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 (see Figure 10.1.81).
- Underslung catcher (see Figure 10.1.82).
- Seated beam connection.
- Continuity (field splice).
- Stainless steel replacements.
- Non-metallic inserts and washers.

Both rod and saddle and underslung catcher systems are added to the structure and only carry load if the pin or hanger fails. The gap between the "catcher" and the girder should 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. An inspector should find out the relative design positions of the components and measure the critical points in the field for comparison.

Figure 10.1.81 Rod and Saddle Retrofit

Figure 10.1.82 Underslung Catcher Retrofit

The seated beam connection completely replaces the pin-and-hanger assemblies. Vacant pin holes may be left under some schemes. Inspection of these details should be the same as inspection at intersecting stiffeners and bearings.

Sometimes a pin-and-hanger assembly is retrofitted by using a bolted field splice (see Figure 10.1.83). This is done only after a structural engineer analyzes the bridge to determine if the members can support continuous spans instead of cantilevered spans. Inspectors should 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. Extra observation should be given to these areas.

Figure 10.1.83 Pin and Hanger Replaced with Bolted 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 10.1.84). Placing a non-metallic insert and washer prevents corrosion between the pin and hanger and allows for normal rotation.

Figure 10.1.84 Stainless Steel Pin-and-Hanger Assembly Retrofit

Primary and Secondary Members

There are many different components to a pin-and-hanger assembly as Figure 10.1.85 demonstrates.

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 10.1.86) or threaded to accept a large nut (see Figure 10.1.86). 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 10.1.87).

The secondary members of a pin-and-hanger assembly include through-bolts and the pin cap, cotter pins, and spacer washers (see Figure 10.1.85).

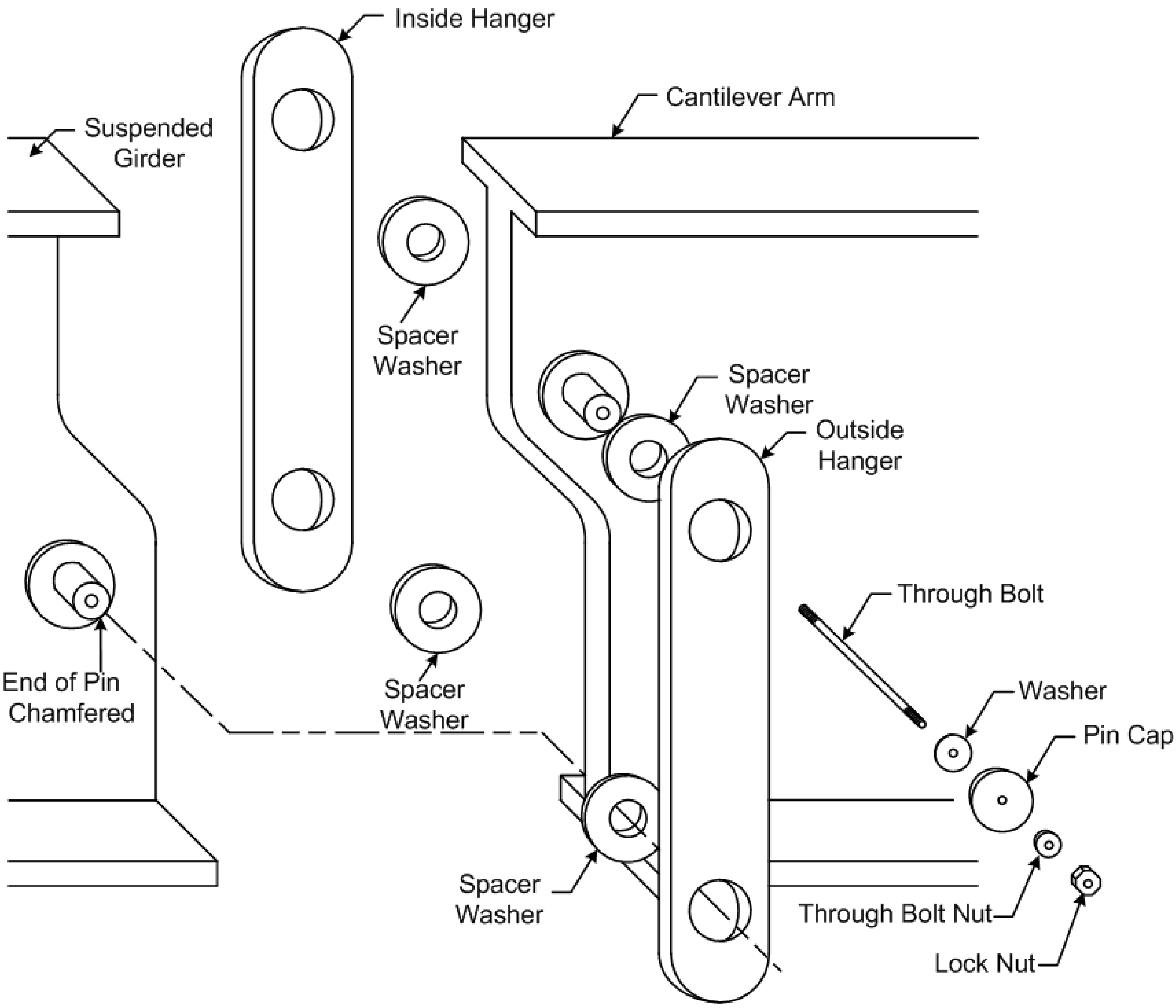


Figure 10.1.85 Pin-and-Hanger Assembly

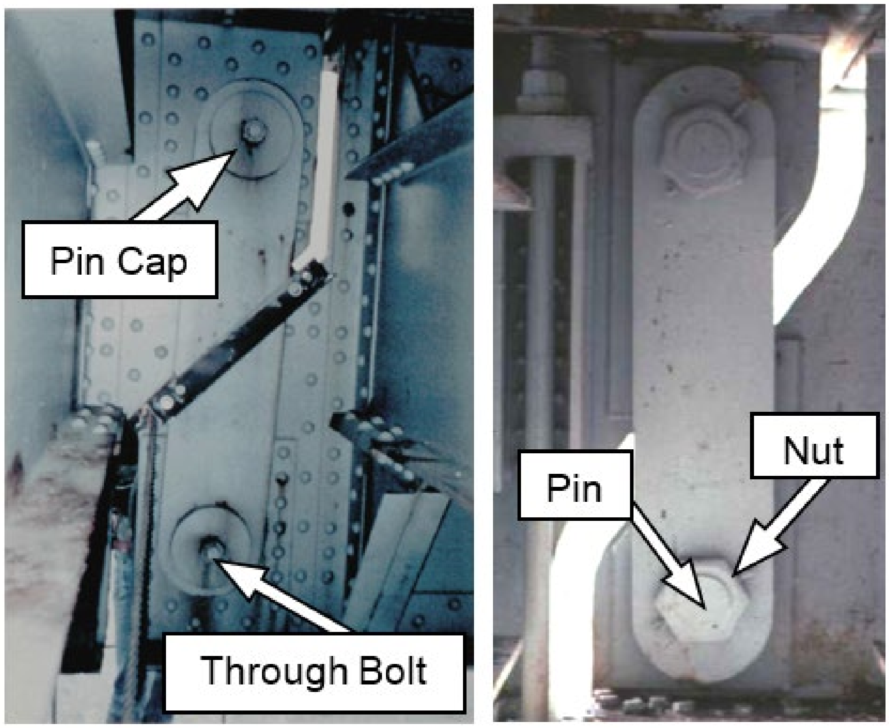


Figure 10.1.86 Pin Cap with Through Bolt and Threaded Pin with Retaining Nut

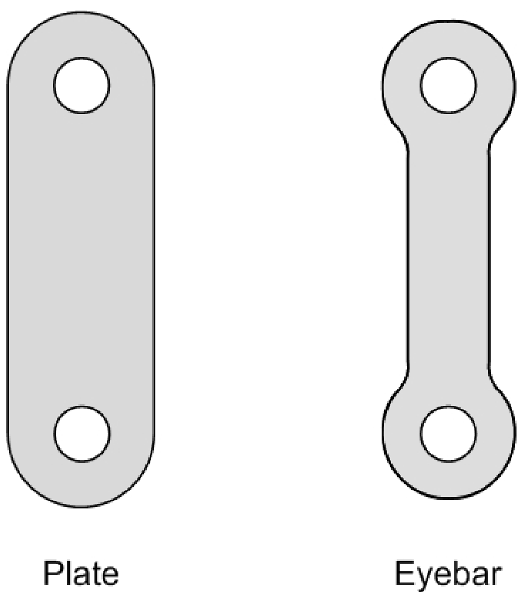


Figure 10.1.87 Plate Hanger and Eyebar Shape Hanger Link

Doubler plates may be present to reinforce the beam web and hanger link plates around the pin hole (see Figure 10.1.88).

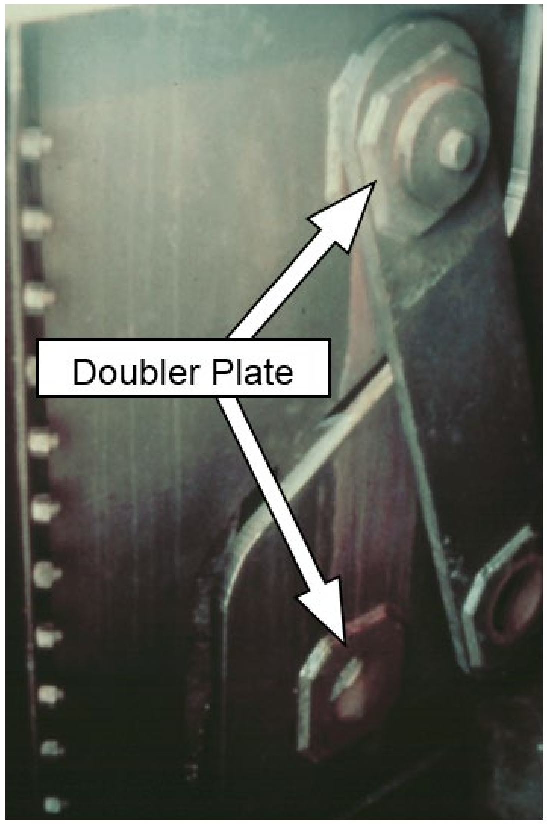


Figure 10.1.88 Hanger and Web Doubler Plates

10.1.11 Gusset Plates

Design Characteristics

Gusset plates are typically used to connect multiple superstructure members. They may connect primary load-carrying members, namely truss and arch members (see Figure 10.1.89), 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 Figure 10.1.90 and Figure 10.1.91). The gusset plate itself is considered a NSTM when it connects one or more NSTMs. The importance of documenting and inspecting gusset plates was heightened by the I-35W Mississippi River Bridge collapse, which occurred due to a gusset plate issue. Reference Section 10.5.3 for more information on this failure.

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.

Figure 10.1.89 Steel Truss Superstructure with Gusset Plates

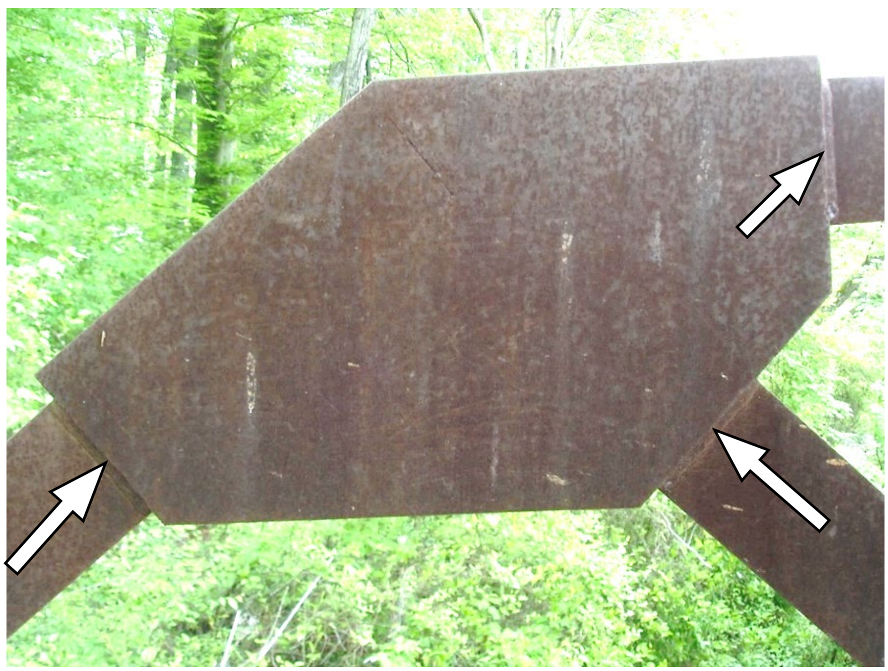


Figure 10.1.90 Steel Gusset Plate with Welded Connections on a Pony Truss

Figure 10.1.91 Steel Gusset Plate with Riveted and Bolted Connections on a Deck Truss

Primary Member Connections

Gusset plates are considered primary members when they connect any two or more primary load-carrying members. They are often the principal means of connecting primary members at panel points for truss and arch superstructures. Gusset plates used for these applications may connect two to more members (see Figure 10.1.92); though connections between three to five members are most common (see Figure 10.1.93). 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 vertical members (hangers or columns).

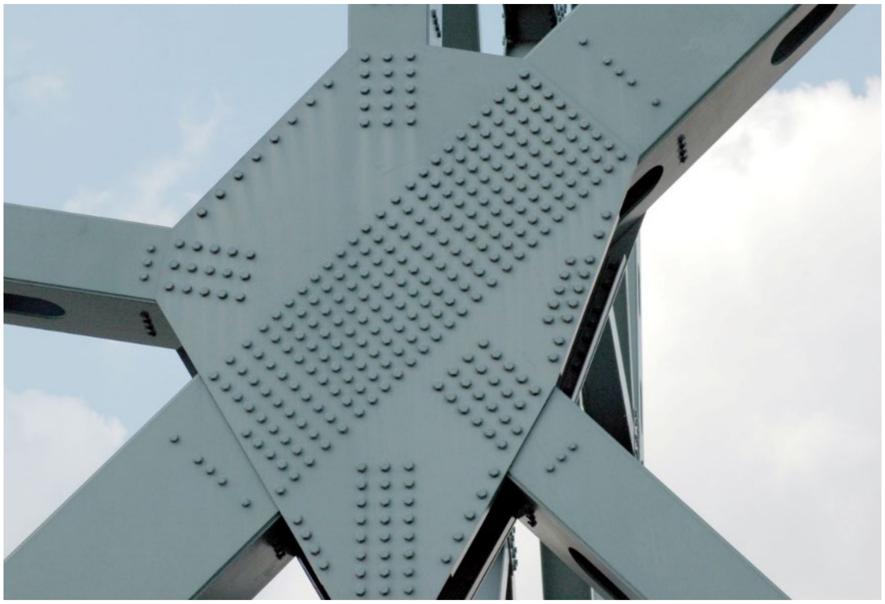


Figure 10.1.92 Odd-Shaped Gusset Plate Connecting Primary Truss Members

Figure 10.1.93 Gusset Plate Connecting Primary Truss Members

Gusset plates connecting primary load-carrying members are responsible for resisting various combinations of forces and stresses. They generally transfer shear, compression, and tensile forces, but not bending moment. 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 detail-oriented procedure.

Secondary Member Connections

Gusset plates are considered secondary when they serve as a connection for secondary members only, or only connect secondary members to a primary member. Gusset plates used in this way are sometimes referred to as connection plates. The secondary members may be connected to primary members at panel points (see Figure 10.1.94) or may be connected to other secondary members (see Figure 10.1.95).

Figure 10.1.94 Gusset Plate Connecting Lateral Bracing on a Truss

Figure 10.1.95 Gusset Plate Connecting Secondary (Bracing) Members

Repairs and Retrofits

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

- Plate thickening (see Figure 10.1.96).
- Free (unbraced) edge stiffening (see Figure 10.1.96).
- Stiffening within the plate.

Welded retrofits are very fatigue-prone. Many trusses and arches older than 1970 are constructed with steel that is more brittle than modern steel. Durable and high-quality welds in the field are difficult to obtain for these more brittle steels. Toughness specifications were generally not enforced until the late 1970s (*AASHTO Guide Specifications for Fracture Critical Non-redundant Steel Bridge Members*).

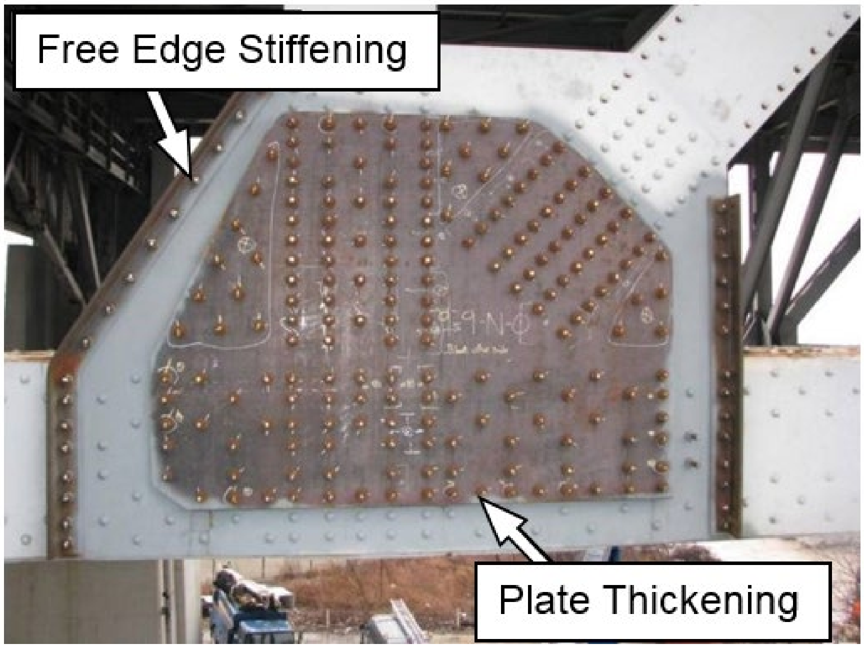


Figure 10.1.96 Plate Thickening and Free Edge Stiffening on a Gusset Plate

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.

10.1.12 Eyebars

Design Characteristics

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 Figure 10.1.97 to Figure 10.1.100).

Figure 10.1.97 Eyebar Tension Member on a Tied Arch Bridge

Figure 10.1.98 Eyebar Cantilevered Truss Bridge

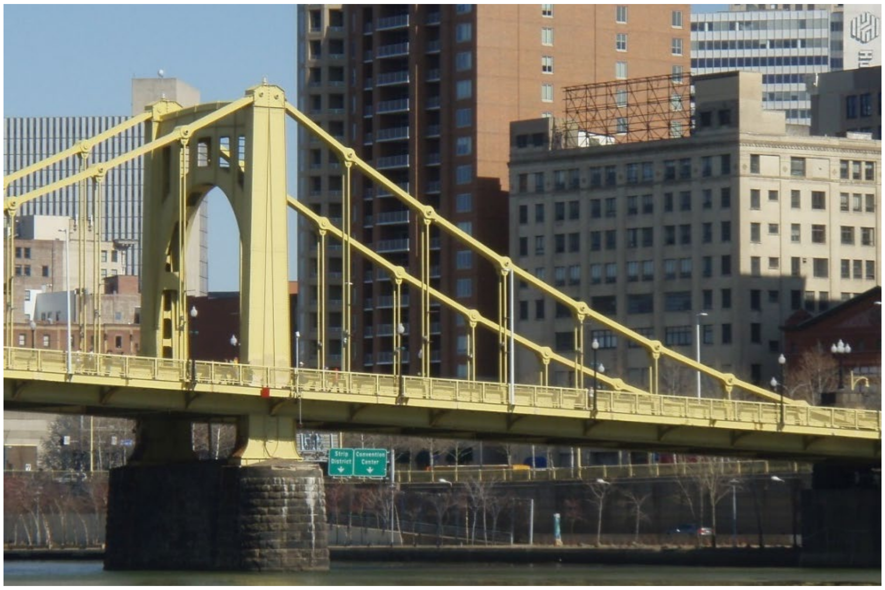


Figure 10.1.99 Eyebar Chain Suspension Bridge

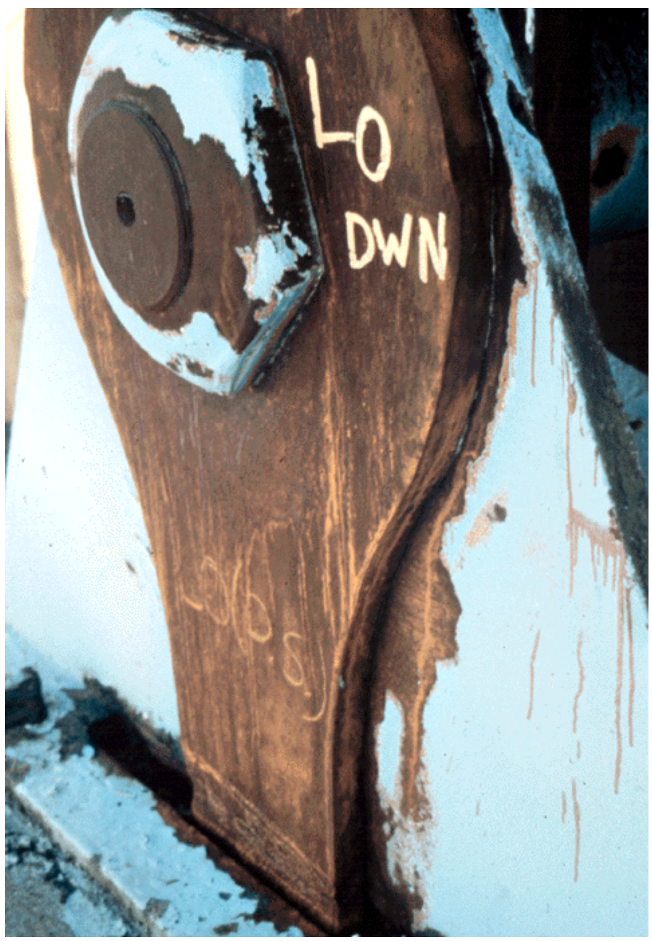


Figure 10.1.100 Anchorage Eyebar

Since the collapse of the Silver Bridge in 1967, there has been considerable concern over the safety of existing bridges, especially those containing eyebars. Reference Section 10.5.1. As a result, costly structural modifications and retrofits were made to many of these bridges to improve internal redundancy (see Figure 10.1.101). Eyebars are rarely used in new bridge designs but are present on existing bridges.

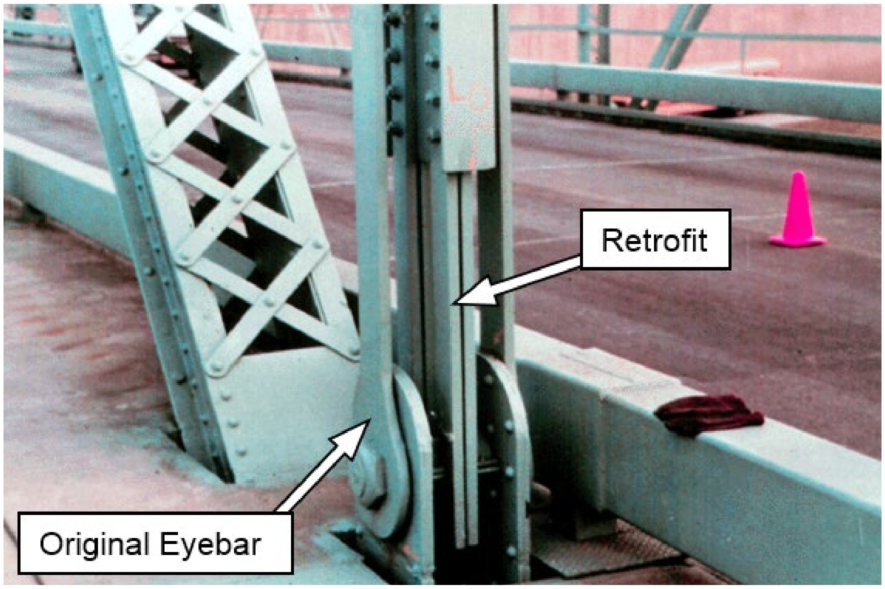


Figure 10.1.101 Retrofit of Eyebars to Add Redundancy

In the late 1800s and early 1900s 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 10.1.102).

Figure 10.1.102 Eads Bridge using Steel Eyebars

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 deficiencies.

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. 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.

Dimensions and Nomenclature

See Figure 10.1.103 for typical dimensions of eyebars used in bridge construction.

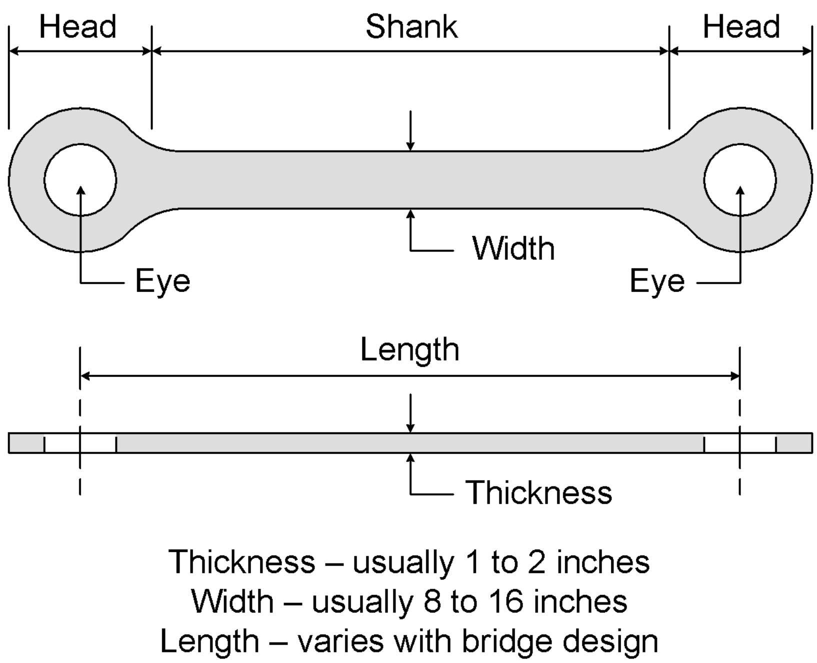


Figure 10.1.103 Eyebar Dimensions

The pin hole in the enlarged head of the eyebar is commonly formed by drilling (see Figure 10.1.104) and connected to the shank. To fabricate the hole, flame cutting is permitted to within two inches of the pin hole.

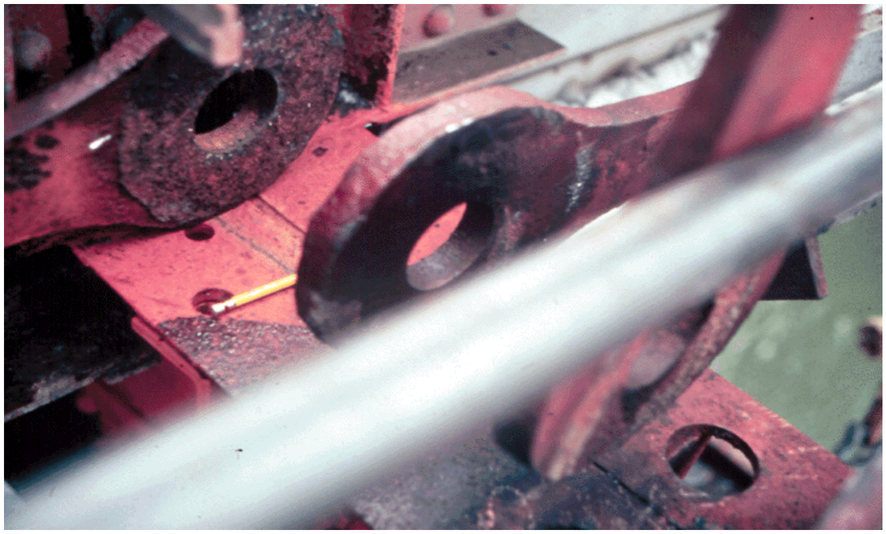


Figure 10.1.104 Eyebar Pin Hole (Disassembled Connection)

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 was the first method employed in shaping metals. An early form of the eyebar, shaped in this manner, is known as a loop rod (see Figure 10.1.105 and Figure 10.1.106). 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.

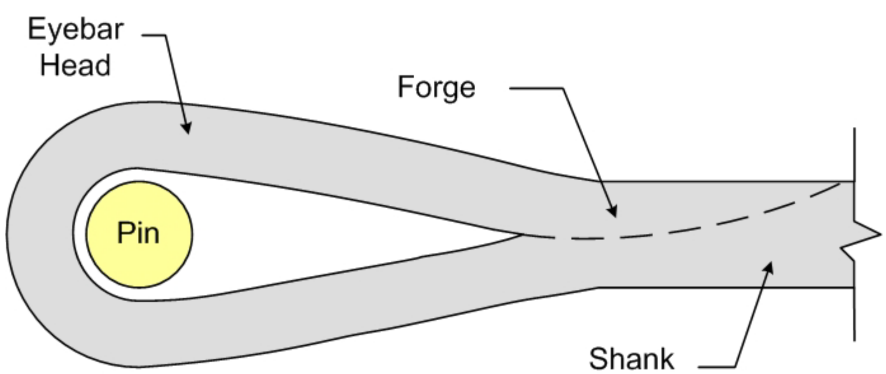


Figure 10.1.105 Forged Loop Rod

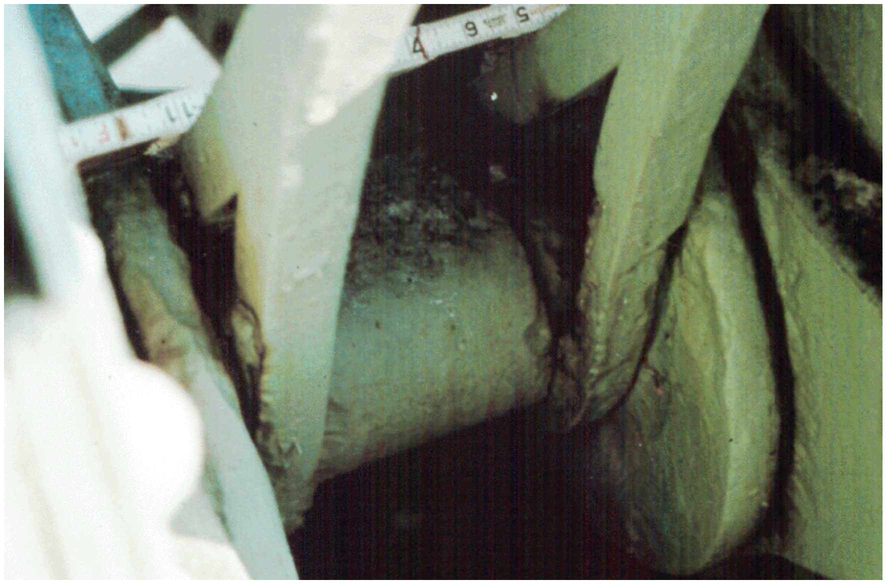


Figure 10.1.106 Close-up of the End of a Loop Rod

The eyebar consists of the two heads (formed by casting) joined to the ends of the shaft (see Figure 10.1.107).

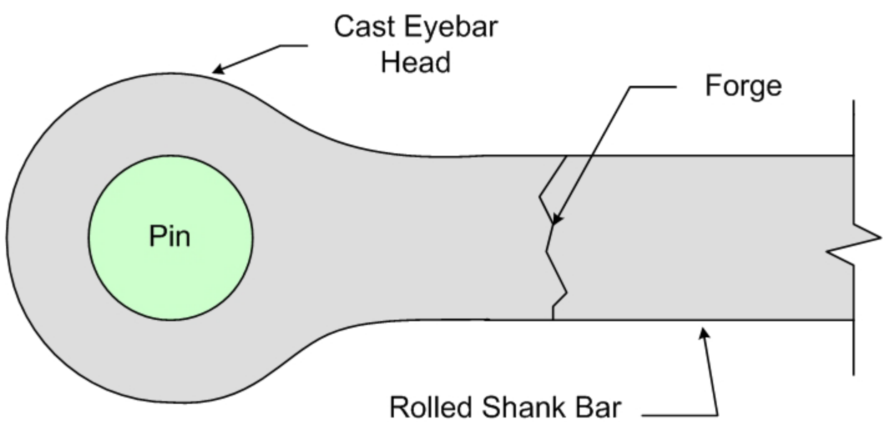


Figure 10.1.107 Forged Eyebar by Mechanical Forge Press

Packing is the term used to describe the arrangement of the eyebars at a given point. Eyebars may be spread apart or tightly packed (see Figure 10.1.108 and Figure 10.1.109). The packing is symmetrical about the center-line of the members connected.

Figure 10.1.108 Loosely Packed Eyebar Connection

Figure 10.1.109 Tightly Packed Eyebar Connection

Spacers or steel filling rings are often wrapped around the pin to prevent lateral movement within the eyebar pack (see Figure 10.1.110).

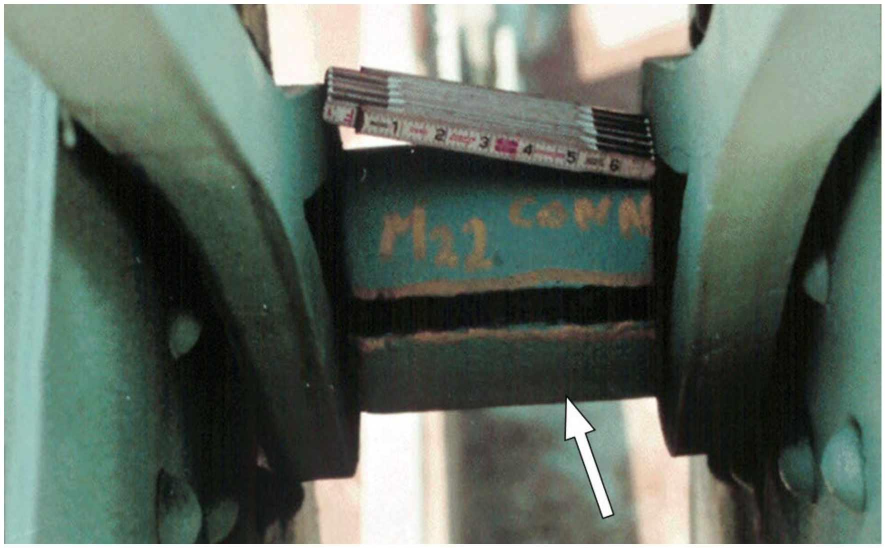


Figure 10.1.110 Steel Pin Spacer or Filling Ring

10.1.13 Converted Railroad Car Bridges

Design Characteristics

Railroad flat car bridges are mostly found in rural areas. It is a low-cost alternative in which old railroad cars are converted into bridge superstructures (see Figure 10.1.111). The bridges have unknown material properties and the history of the cars is undocumented, which raises concerns with this type of structure for public agencies. They are unsuitable for routes with a high traffic volume or exposure to cyclical loading, as the materials do not perform well with regards to fatigue. Railroad cars converted into bridges may need in-depth analysis to comply with AASHTO and State Department of Transportation standards.

Figure 10.1.111 Elevation of a Converted Railroad Car Bridge

Railroad flat car bridges are a viable option on local or private roads for a few reasons. They are inexpensive and easy to install. They also generally need little maintenance. Generally, span lengths range from 20 to 80 ft, depending on the type of car used.

A flat car generally consists of one main deep box girder in the center and two shallower exterior girders, which are usually channel beams. Transverse cantilevers are attached to the box girder on both sides that act like floorbeams. Some flat cars also have stringers on top of the transverse members (see Figure 10.1.112). Since each car is only the width of a typical train, many times multiple railroad flat cars are constructed next to each other to build a bridge wide enough for two lanes of traffic (see Figure 10.1.113). Timber decks have been the most common deck type used on railroad car bridges in the past, but steel or concrete decks are more common in current practice.

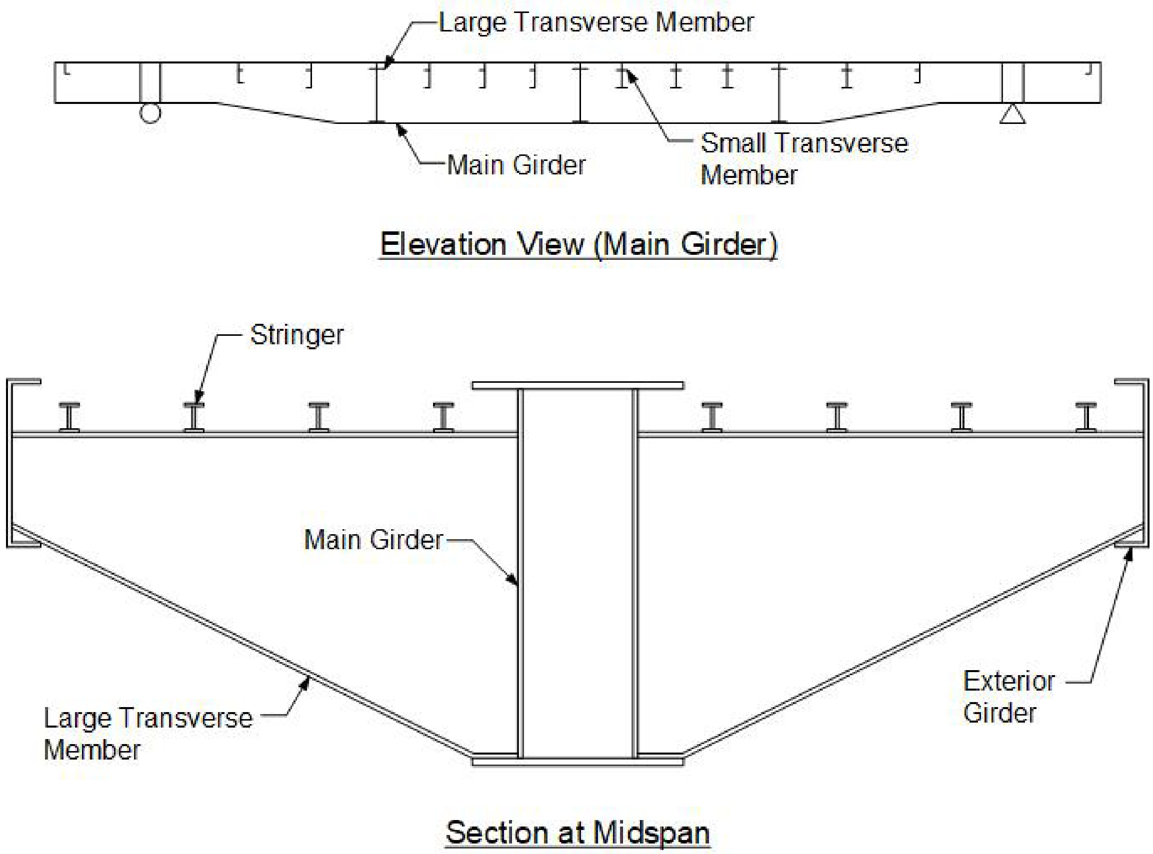


Figure 10.1.112 Elevation and Cross Section of a Railroad Flat Car Bridge

Figure 10.1.113 Underside of a Double Converted Railroad Car Bridge

Some railroad flat car bridges have no bridge railing or approach guardrail, as it is very difficult to attach an adequate bridge railing to the repurposed railroad cars (see Figure 10.1.114).

Figure 10.1.114 Railroad Car Converted Bridge without Bridge Railing

10.1.14 Secondary Members

Although secondary members do not resist primary loads, such as vehicular live load, they are important features for construction and the global stability of the bridge. Secondary members also distribute secondary live load forces such as wind (refer to Chapter 6). For all steel superstructure types, there are a few common member types that are considered secondary.

Diaphragms and Cross-Frames

Diaphragms and cross-frames are provided between beams, girders, or stringers for stabilization during construction and to help distribute secondary live load. They are also used to support the free edges of decks at the ends of spans. Diaphragms can be rolled shapes (e.g., I-beams and channels, see Figure 10.1.115) or transverse bracing can be cross-frames constructed from angles, tee shapes, and plates (see Figure 10.1.116). They are usually attached to transverse web stiffeners that are typically referred to as connection plates. Due to complex load transfer, as described previously, diaphragms or cross-frames can be considered primary members if a steel superstructure is curved.

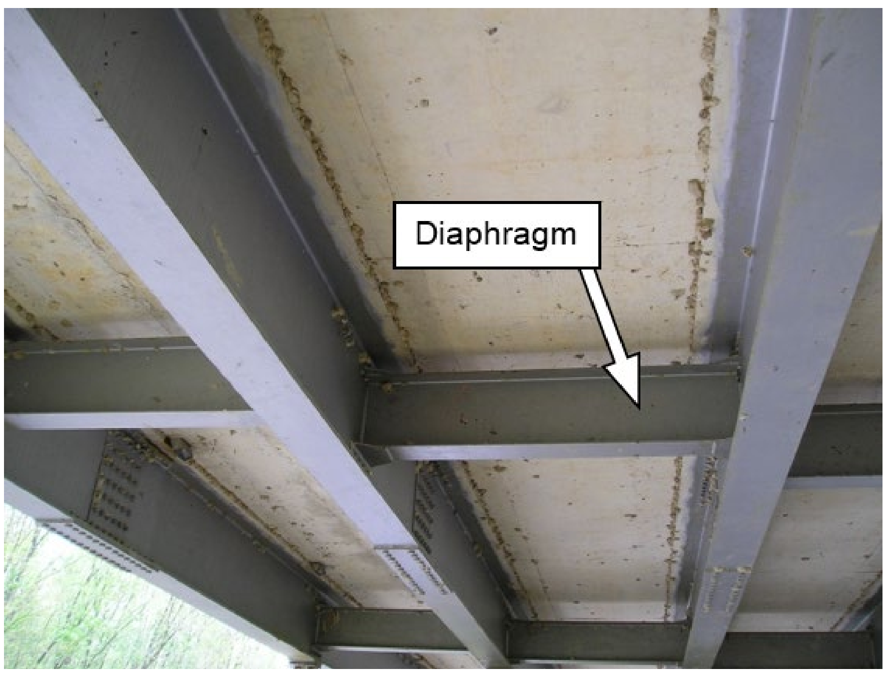


Figure 10.1.115 Diaphragm

Figure 10.1.116 Cross-Frame

Lateral Bracing

Superstructure types such as trusses, arches, and frames, need lateral bracing for global stability. Upper and lower lateral bracing is in a horizontal plane and functions to keep the two trusses longitudinally in line with each other. Most trusses have upper and lower chord lateral bracing, except for 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 or floorbeams at each panel point using gusset plates (see Figure 10.1.117 and Figure 10.1.118). Lateral bracing is subjected to tensile stresses caused by longitudinal or transverse loadings.

Figure 10.1.117 Upper Lateral Bracing on a Truss

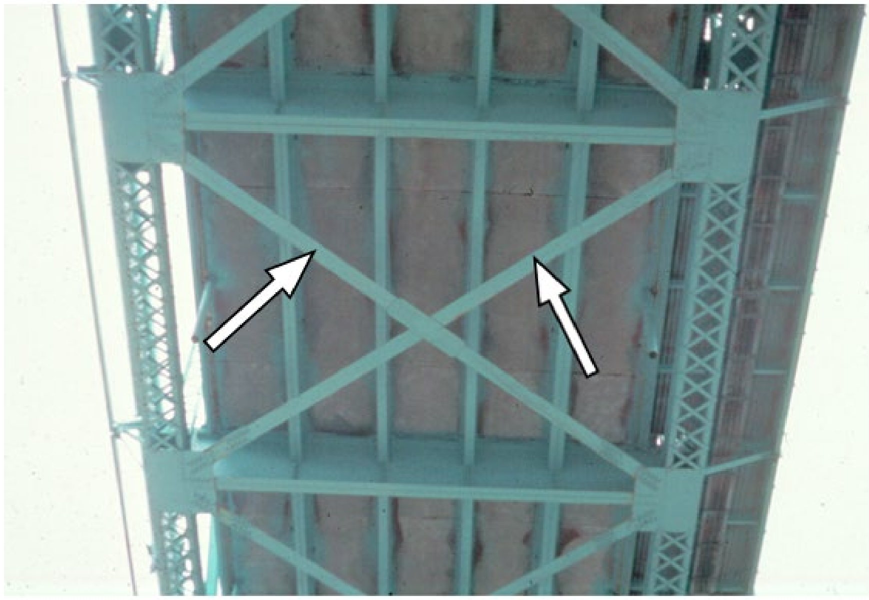


Figure 10.1.118 Lower Lateral Bracing

Except for the top flanges of box girders, lateral bracing is used less frequently than it was in the past for girder bridges due to cost and appearance. The girders, diaphragms, and cross-frames are designed to resist horizontal loads and out-of-plane bending without the need for lateral bracing.

Sway and Portal Bracing

Sway bracing is in a vertical plane and functions to keep the two trusses, arches, or frames parallel. The bracing is typically constructed from built-up or rolled shapes. The sway bracing at each end is called portal bracing and is much heavier than the other sway bracing. Sway and portal bracing are subjected to stresses caused by transverse, horizontal loads, such as wind. They provide restraint against translation and rotation of the structure and individual bracing members carry a combination of tension and compression forces. The bracing also reduces the unbraced length of the truss members and therefore helps resist buckling of due to axial compression.

Sway bracing on 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 Figure 10.1.119 and Figure 10.1.120). The legs of rigid frames utilize cross bracing as sway bracing (see Figure 10.1.121).

Figure 10.1.119 Truss Bridge Sway Bracing with Collision Damage

Figure 10.1.120 Pony Truss Sway Bracing

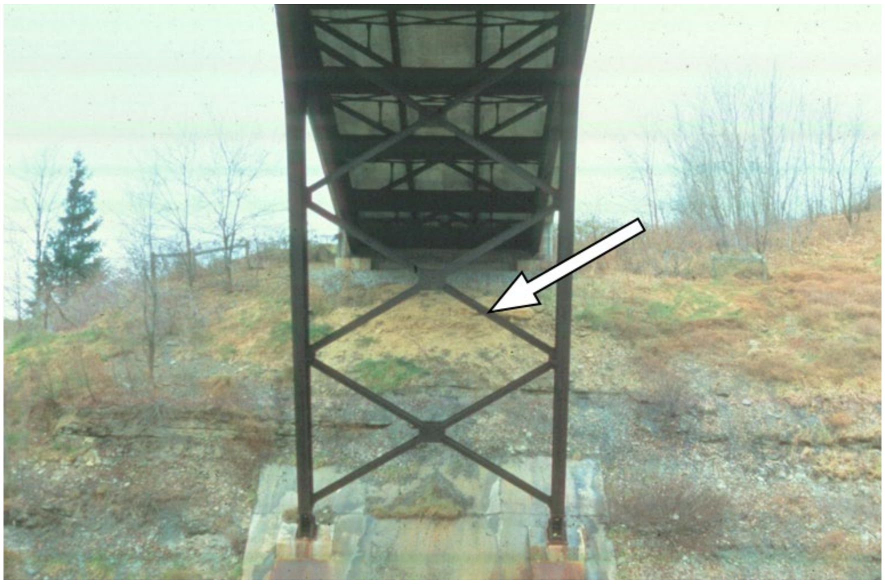


Figure 10.1.121 Rigid Frame Sway Bracing

Portal bracing performs the same function as sway bracing. It is situated at the first and last upper panel points and helps distribute all sway bracing loads to the end chords and eventually into the bearings (see Figure 10.1.122).

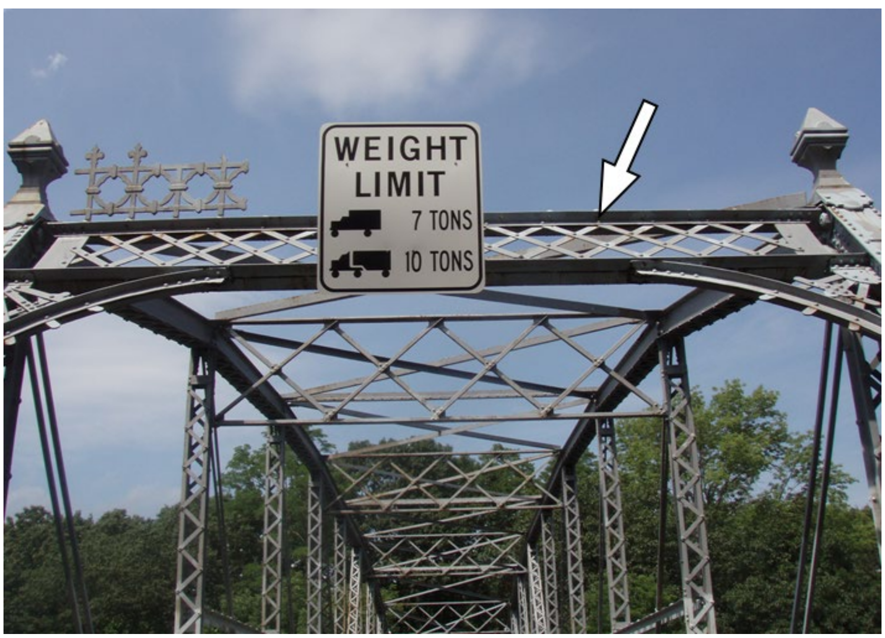


Figure 10.1.122 Portal Bracing with Attached Load Posting Sign

Portal and sway bracing, bottom flange lateral bracing, and bracing members supporting arch ribs all provide lateral support and bracing; however, these members are often especially important to prevent collapse of the main arch, truss, or girder members (see Figure 10.1.123).

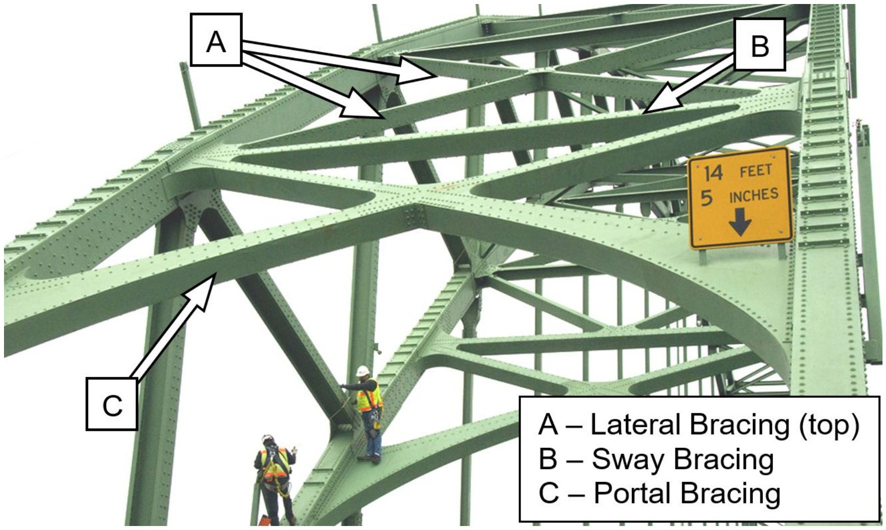


Figure 10.1.123 Tied Arch Secondary Members

For AASHTO fracture control plans (FCP), Table 6.6.2.1-1 of the *AASHTO LRFD Bridge Design Specifications, 9th Edition* lists these all as primary members.

Section 10.2 Overview of Common Deficiencies

10.2.1 Steel Deficiencies

Common potential deficiencies that may occur on steel bridges include:

- Corrosion.
- Fatigue cracking.
- Overload damage.
- Collision damage.
- Heat damage.
- Coating failures.
- Loose, missing, or deteriorated fasteners.
- Out-of-plane distortion (including buckling).

Refer to Chapter 7 for a detailed explanation of the properties of steel, types, and causes of steel deficiencies.

Fatigue-prone details may be associated with nonredundant steel tension members (NSTMs). An NSTM is a primary steel member fully or partially in tension, and without load path redundancy, system redundancy, or internal redundancy, whose failure would probably cause a portion of or the entire bridge to collapse (23 CFR 650.305).

Many steel superstructures have NSTMs. Common NSTMs include truss chords, verticals, diagonals, frames, tie girders, gusset plates, and pin and hanger assemblies, that experience tension and do not have load path redundancy. Refer to Chapter 7 for more information regarding fatigue and fracture in steel bridges.

Section 10.3 Inspection Methods

10.3.1 Visual and Physical Inspection

The inspection of steel for surface deficiencies, sagging, and distortion is primarily a visual activity. Inspection mirrors or flashlights can be used to see areas not visible by normal visual inspection light.

The most common method for inspecting steel is to use a wire brush, hand grinder, or hammer to remove loose paint or rust flakes to access bare steel to check section loss, distortion, or cracking. Care should be exercised when removing paint or rust in cracked areas. Excessive cleaning may close the crack width on the surface or gouge the material which may act as a stress riser for fatigue crack initiation. To that end, a wire wheel brush is preferred over grinding discs. The actual remaining steel should be measured when section loss occurs. Inspectors should not attempt to estimate the section loss. Distortion should be measured from the original portion; crack lengths and widths should be measured carefully. During the passage of traffic, the inspection team should listen for abnormal noises caused by moving members and loose connections.

Once the presence of a deficiency has been verified, the inspection team should examine all other similar locations and details.

10.3.2 Advanced Inspection

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

There are several advanced methods for steel inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Acoustic Emission (AE).
- Crack Propagation Gage (CPG).
- Dye Penetrant Testing (PT).
- Eddy Current Testing (ET).
- Magnetic Particle Testing (MT).
- Magnetic Flux Leakage (MFL).
- Ultrasonic Testing (UT).
- Phased Array Ultrasonic Testing (PAUT).

Other methods that may be considered destructive testing methods include the following.

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

All advanced methods are described in detail in Chapter 17.

Section 10.4 Inspection Areas and Deficiencies

For Routine inspections, all areas of the bridge and all locations of each member are to be observed, at times requiring specialized access equipment. Before inspecting specific areas below the bridge, inspectors should examine the steel superstructure for general global alignment. The curb line, top of railing, and the bottom of the superstructure should be viewed and examined for vertical or horizontal misalignment. Both sides of expansion joints should be examined to check if they line up properly. Vertical misalignment, horizontal misalignment or sagging may be an indication of specific issues noted in this section. Visual, physical, or advanced inspection methods may be necessary to evaluate the following inspection areas and deficiencies. This section describes common problems related to specific areas and details to attentively inspect.

10.4.1 Bearing Areas

The web area over a support and at frame knee areas should be examined for cracks, section loss, or buckling. If bearing stiffeners, jacking stiffeners, and diaphragms are present at the supports, they should be inspected for cracks, section loss, and buckling as well. In rigid frames, the bottom of the frame legs should also be inspected for corrosion or buckling (see Figure 10.4.1).

Figure 10.4.1 Bearing Area of a Rigid Frame Bridge

The bearings at each support should be checked for corrosion and section loss. The alignment of each bearing should be inspected and any movement should be noted. Inspectors should report any buildup of debris surrounding the bearings that may limit the bearing from functioning properly. Inspectors should check for any bearings that are frozen due to heavy corrosion. Refer to Chapter 13 for a detailed description on the inspection of bridge bearings.

10.4.2 Shear Zones

The web areas near the supports should be examined for any section loss or buckling (see Figure 10.4.2 through Figure 10.4.4). Shear stresses are greatest near the supports; therefore, the condition of the web is more critical near the supports than at mid-span. If girders are haunched, the weld between the web and the insert plate should be checked.

Figure 10.4.2 Corroded Shear Zone on a Rolled Beam Bridge

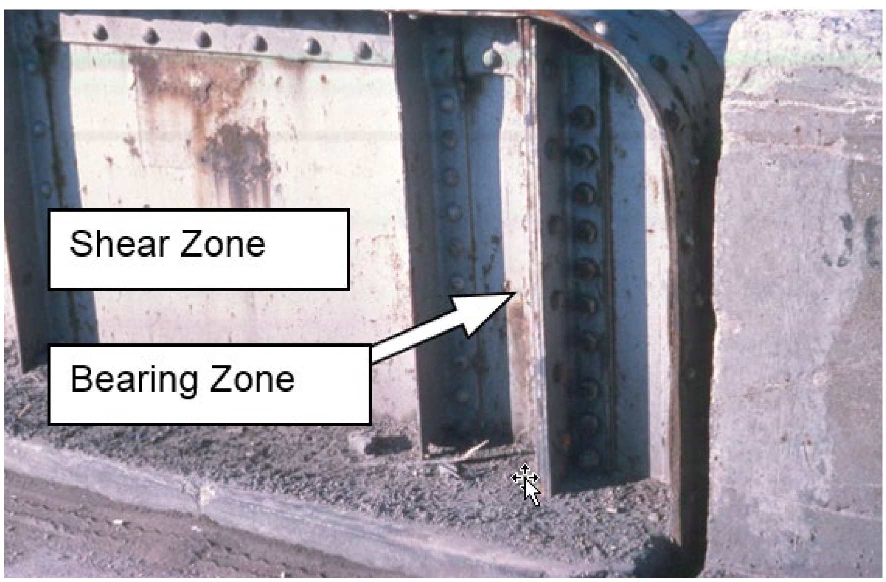


Figure 10.4.3 Web Area Near Support on a Through Girder Bridge

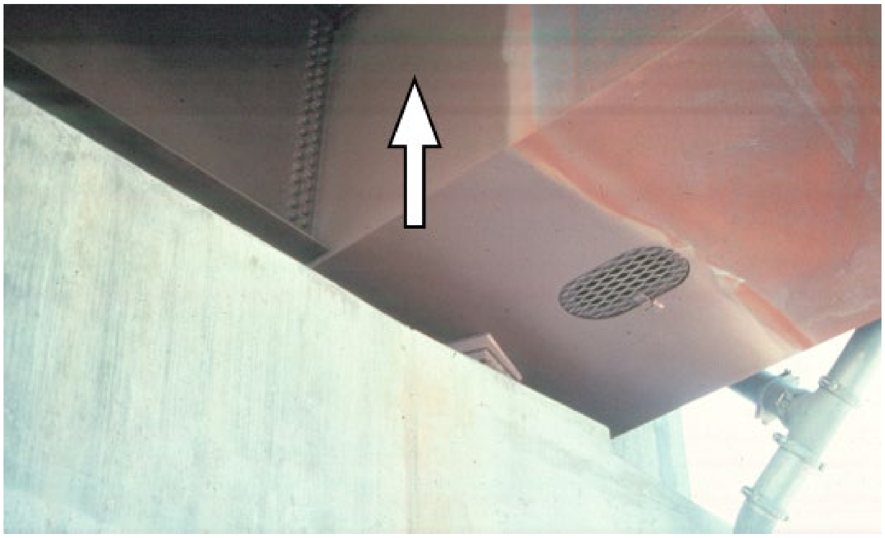


Figure 10.4.4 Box Girder Shear Zone

10.4.3 Flexure Zones

The flexure zone of each beam/girder includes the entire length between the supports, but the maximum positive moment is generally considered to fall within the middle third of simple spans (see Figure 10.4.5). Continuous spans have both positive and negative moments (see Figure 10.4.6). Field splices (if present) are close to points of counter-inflection (transition between positive and negative moments in continuous spans).

Figure 10.4.5 Maximum Moment Region on a Two-Span Simple Rolled Beam Bridge

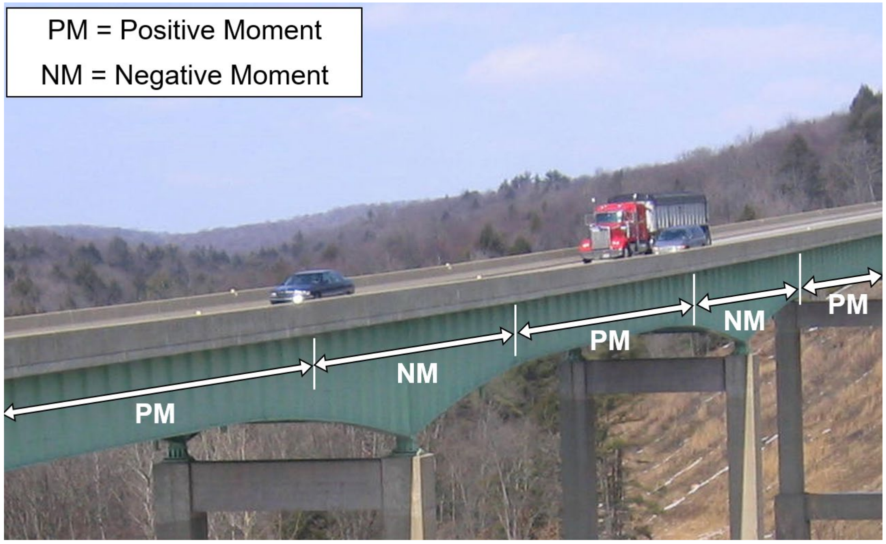


Figure 10.4.6 Flexural Zones on a Continuous Span Plate Girder Bridge

The tension and compression flanges and webs should be investigated for distortion, corrosion, loss of section, cracks, dings, and gouges (see Figure 10.4.7). The inspection team should check the flanges in high stress areas for bending or flexure-related damage. The compression flange should be examined for local buckling and, although it is uncommon, for elongation or fracture of the tension flange. If welded cover plates are present, the inspection team should carefully check at the ends of the cover plates for cracks due to fatigue. Flange splice welds and longitudinal stiffener splice welds should be checked in tension areas (see Figure 10.4.8).

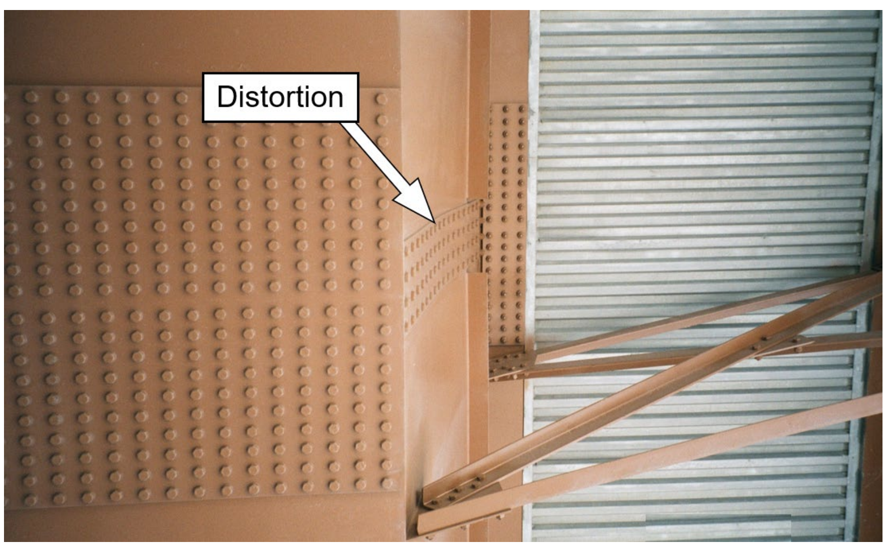


Figure 10.4.7 Example of Distortion on Box Girder

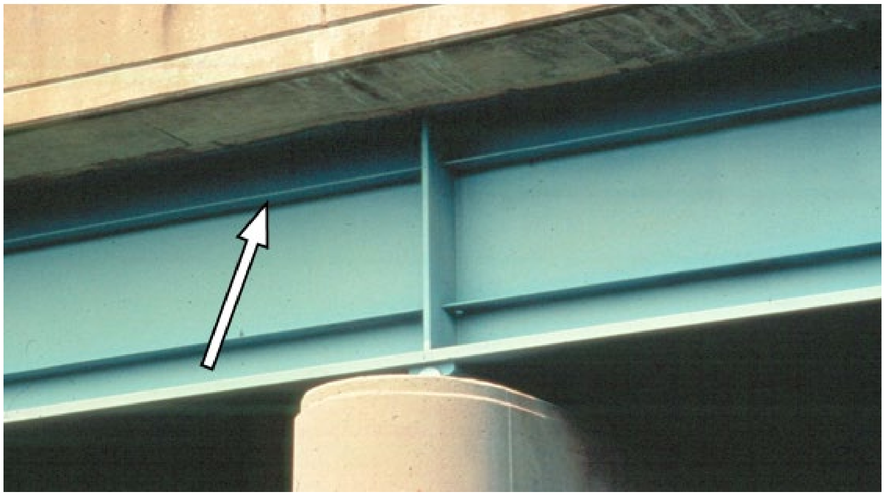


Figure 10.4.8 Longitudinal Stiffener in Tension Zone on a Plate Girder Bridge

In steel deck arch bridges, inspectors should examine the end connections of spandrel bents, spandrel columns, and spandrel (fascia) girders for section loss, cracks, and loose fasteners. The girders, caps, and columns should be checked for flexure, section loss, and distortion (see Figure 10.4.9). The connections between the lateral bracing and the spandrel members should also be examined.

Figure 10.4.9 Braced Rib Deck Arch Showing Spandrel Columns

The flexural zone for steel rigid frames should be investigated. Special observation should be given 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 10.4.10).

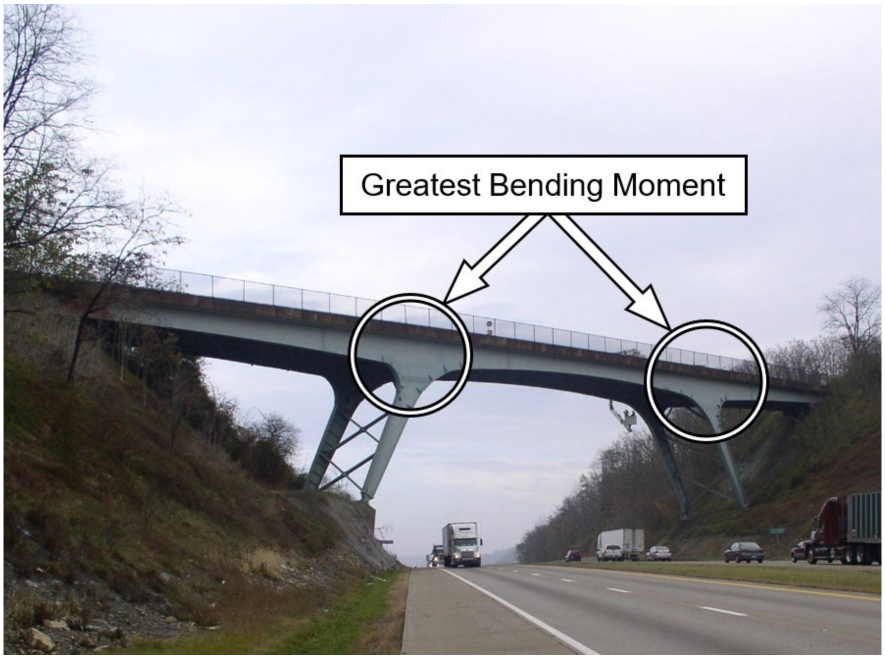


Figure 10.4.10 Flexural Zone on a Rigid Frame

10.4.4 Axial Tension Members

For steel superstructures subjected to axial tension loads, special observation should be given to the following locations:

- Inspectors should check for section loss (corrosion) and cracks (see Figure 10.4.11).
- For box-shaped chord members, the inside should be checked for debris and corrosion, cracks, or section loss (see Figure 10.4.12).
- Where multiple eyebars make one member, inspectors should 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 10.4.13).
- The alignment of the members should be checked, making sure they are straight and not bowed or distorted, as this could be a sign of pier movement, collision damage or unintentional force reversal (see Figure 10.4.14).
- Inspectors should look for repairs, especially welded repairs, if they have been applied to steel tension members. Base metal cracks can develop at these locations.
- The counters should be checked for excessive wear and abnormal rubbing where the counters cross.
- The tension in threaded members should be checked. The inspector should pull transversely (by hand) to check the relative tension. Proper tension allows the counter to move slightly. If improper tension is found, the inspector should not adjust the turnbuckle. Instead, the designated point of contact for the bridge owner should be promptly notified.

Figure 10.4.11 Corrosion and Section Loss on a Truss Bottom Chord

Figure 10.4.12 Inside of Box Chord Member

Figure 10.4.13 Bottom Chord with Eyebars

Figure 10.4.14 Bowed Bottom Chord Eyebar Member

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. The pins at such connections should be carefully inspected for corrosion, section loss, distortion, and fixity.

In through and tied arch bridges, hangers are designed to resist axial tensile loads and consist of either eyebars or cables. The connections at both ends of the hangers should be checked (see Figure 10.4.15). Inspectors should look for corrosion, cracks, and broken or misaligned wire strands. The alignment of the hangers should be examined; the hangers may be near traffic, so inspectors should check for collision or fire damage (see Figure 10.4.16). The hangers should be inspected for any welded attachment; inspectors should examine the welds between the attachment and the hanger for cracks.

Figure 10.4.15 Hanger Connection on a Through Arch

Figure 10.4.16 Hardness Test on Fire Damaged Arch Cables

10.4.5 Axial Compression Members

For superstructure members subjected to compressive loads, special observation should be given 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.
- Inspectors should check for local buckling, an indication of overstress (see Figure 10.4.17).
- Wrinkles or waves in the flanges, webs or cover plate are common forms of buckling.

Figure 10.4.17 Buckled End Post

The arches in arch bridges are designed to primarily resist axial compression (see Figure 10.4.18). The alignment of the arch should be inspected for signs of buckling and crippling in the arch ribs. Buckling can be determined using a string line to help locate distortion. Inspectors should check for general corrosion and deterioration. Any pins should be examined for corrosion and wear. The arch rib splice plates and the connections at the hangers or spandrel bents should be checked as well.

Figure 10.4.18 Through Truss Arch Compressive Members

10.4.6 Areas Exposed to Drainage

Inspectors should check horizontal surfaces that can trap debris and moisture. These areas 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. Specific areas to inspect include:

- Areas exposed to drainage runoff/at the curb line.
    - Along girder webs of through girder systems (see Figure 10.4.19).
    - Truss members adjacent to the roadway.
- Along the bottom flanges of chord members and floor system members.
- Pockets created by diaphragm or floor system connections.
- Inspectors should ensure water is not gaining access to the interior of steel box girder bridges.
- Any drainage holes should be inspected for blockage and corrosion.
- Area between eyebars, especially if closely spaced.
- Tightly packed truss panel points.
- Lateral bracing gusset plates.

Figure 10.4.19 Corrosion on Top of Floorbeam Bottom Flange

10.4.7 Areas Exposed to Traffic

Inspectors should check underneath the bridge for collision damage to the main girders and bracing if the bridge crosses over a highway, railway, or navigable channel. Any scrapes, cracks, and distortion around cross frames should be checked, as the damage may be in the bays that are not adjacent to the impacted girder. Also, any misalignment of the bearings due to possible lateral movement should be examined. This is of particular concern if the structure is non-composite. Inspectors should investigate any main members adjacent to the roadway along the curb lines and at the ends for collision damage. Any cracks, section loss, misalignment, sag, or distortion found should be documented (see Figure 10.4.20 through Figure 10.4.23).

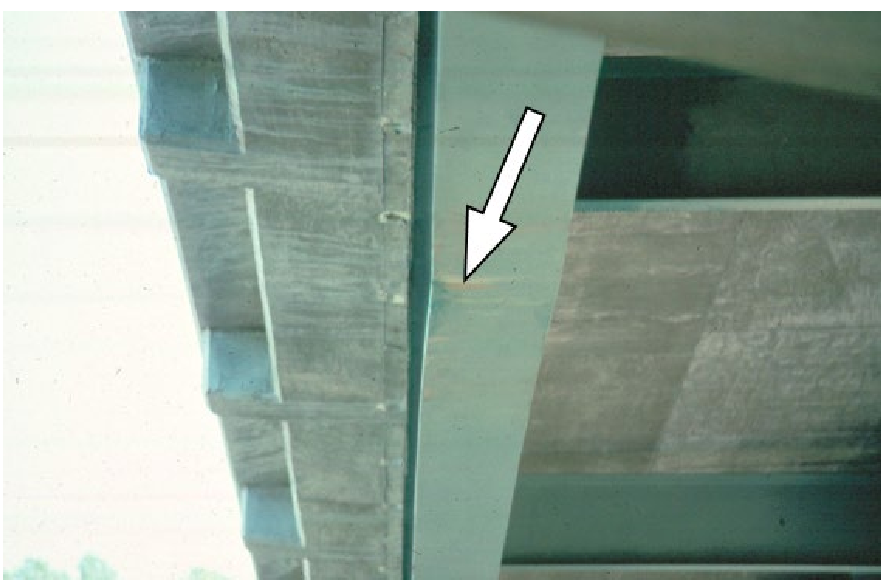


Figure 10.4.20 Collision Damage on a Deck Girder Bridge

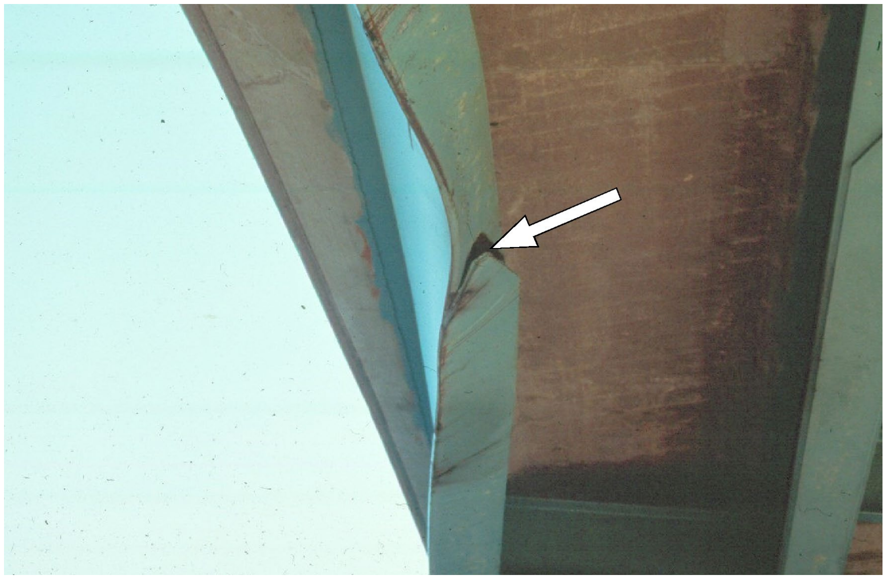


Figure 10.4.21 Collision Damage on a Rolled Beam Bridge

Figure 10.4.22 Collision Damage to Flange on a Through Girder Bridge

Figure 10.4.23 Collision Damage to Truss Members Due to Over-height Vehicle

10.4.8 Areas Previously Repaired

Any repairs that have been made previously should be thoroughly examined. Inspectors should determine if the repaired areas are sound and functioning properly. Typical repair details for steel bridges include welded or bolted plate retrofits. Inspectors should check for pack rust in between plates and steel members. Cracking can develop from bolt holes or welds; therefore, it is important to be aware of these areas.

10.4.9 Thermal Effects

Depending on the geographical location of the bridge, shifts in temperature can have a significant effect on a steel structure. If thermal expansion and contraction are limited or restricted completely, it can cause the distortion of bridge members. For example, the superstructure members can experience a build-up of internal stress if the bearings are locked up or frozen. The substructure can also be affected if the bearings are not functioning properly. The bearings should be inspected with the temperature in mind and record if the bridge is not expanding or contracting as expected.

10.4.10 Floor Systems

A typical floor system 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. Cracking can also occur in the angles connecting a stringer and floorbeam, particularly if that connection was designed as simple, but is unintentionally subjected to moment (see Figure 10.4.24).

Figure 10.4.24 Crack in Angle at Floorbeam-Stringer Connection

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

- The end connections of floorbeams should be checked for corrosion, distortion, and cracking as they are exposed to moisture and deicing chemicals from the roadway (see Figure 10.4.25).
- The floorbeams and stringers should be inspected for corrosion, particularly under open grid decks.
- Inspectors should determine if floor system member flanges and webs have corrosion or cracks (see Figure 10.4.26 and Figure 10.4.27).

Figure 10.4.25 Corroded Floorbeam End Connection with Deicing Chemical Residue

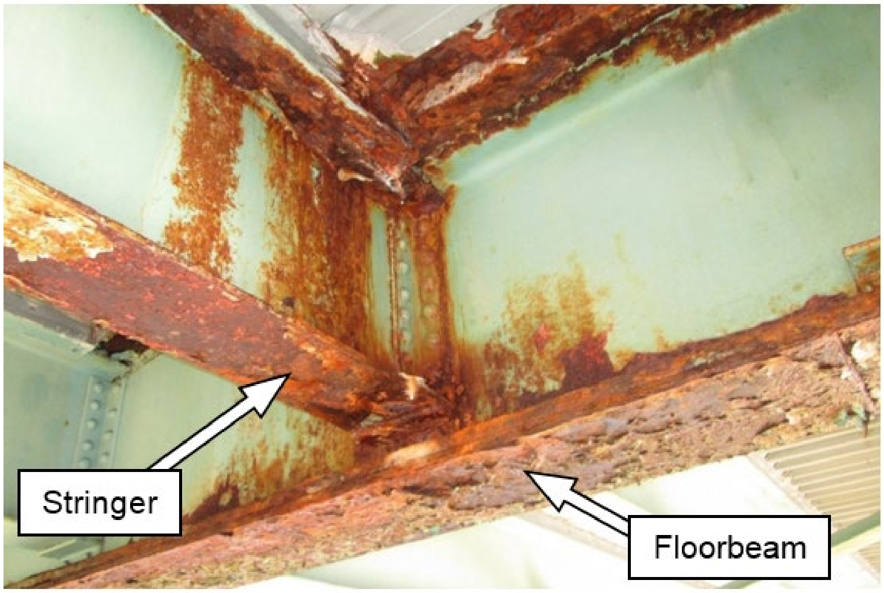


Figure 10.4.26 Corroded Stringer End and Floorbeam

Figure 10.4.27 Corroded Floorbeam

10.4.11 Pin and Hanger Assemblies

Inspectors should observe the general condition of the pin-and-hanger assembly. Alignment of the adjacent beam webs and flanges should be checked using a straight edge. If present, the wind lock should be inspected for signs of excessive transverse movement. A wind lock consists of steel or neoprene members attached to both the suspended and cantilever bottom flanges (see Figure 10.4.28). It restricts differential latitudinal movement between the cantilevered and suspended girders. It should be noted if deck drainage is entering the assembly.

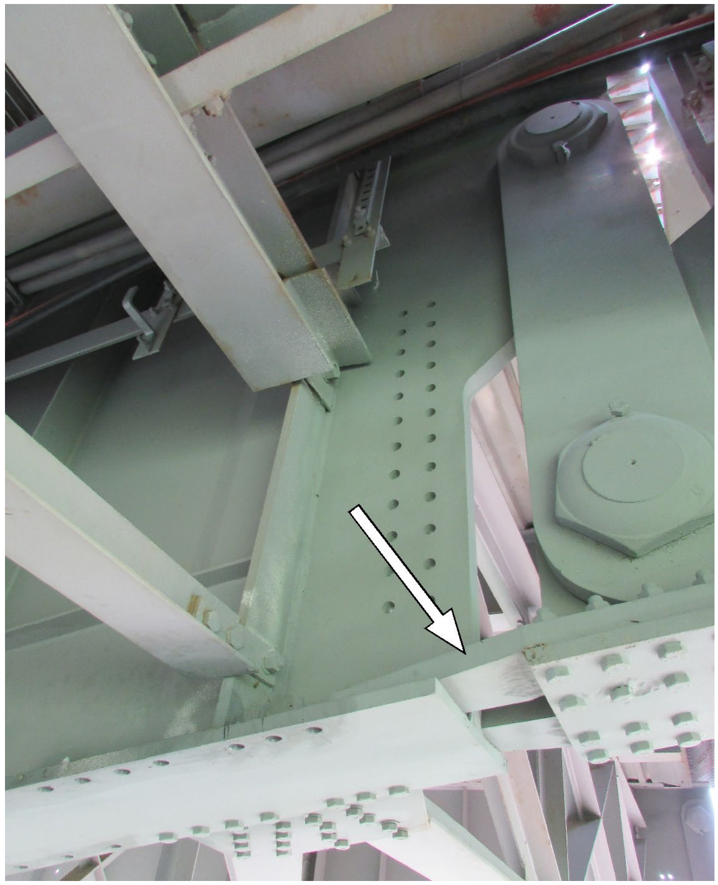


Figure 10.4.28 Pin and Hanger Wind Lock

The actual dimensions between the pins should be measured along with the distance from each pin to the end of the hanger assembly. These values should be compared to the as-built dimensions (see Figure 10.4.29). The tolerances of the dimension between the pin and hanger end should be within 1/32 inch.

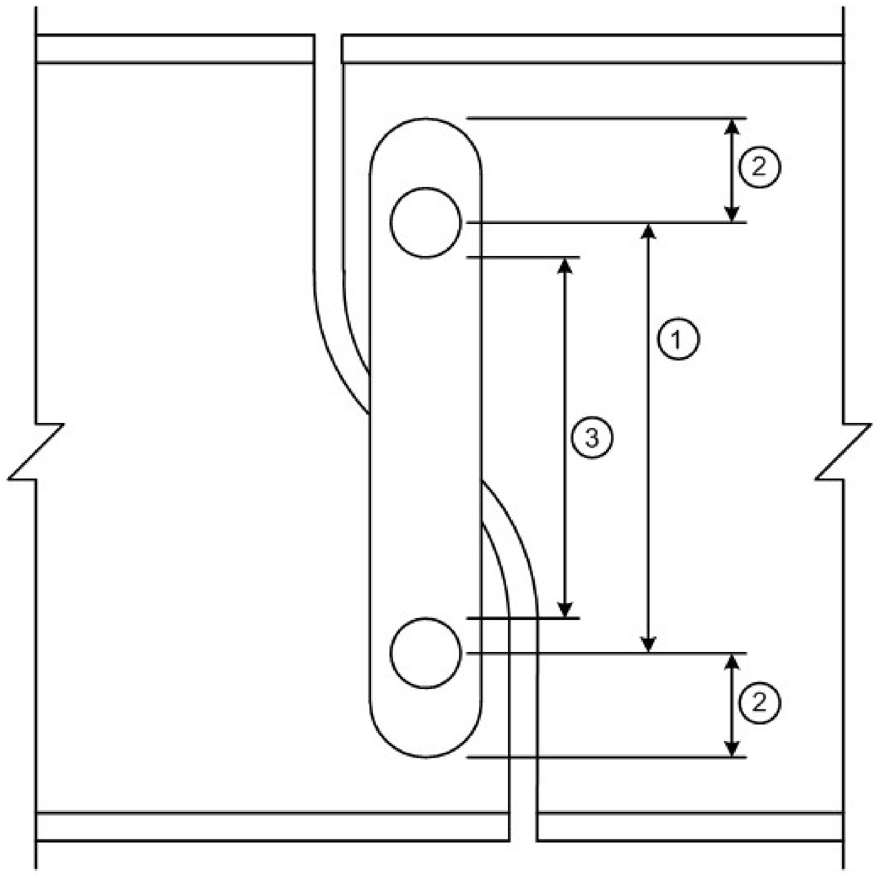


Figure 10.4.29 Elevation View of Pin and Hanger

Inspectors should try to determine if movement is taking place. Corrosion can cause fixity at pin-and-hanger connections (see Figure 10.4.30). This changes the structural behavior of the connection and is a source of cracking. Powdery red or black rust where surfaces rub indicates movement. It may or may not indicate appreciable section loss. Some minor rust staining may be an indication of normal movement or rotation. Where there is relative movement expected, an unbroken paint film across a surface indicates the pin is frozen.

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. When trucks pass over the structure, inspectors should listen for unexpected noises due to substantial movement.

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

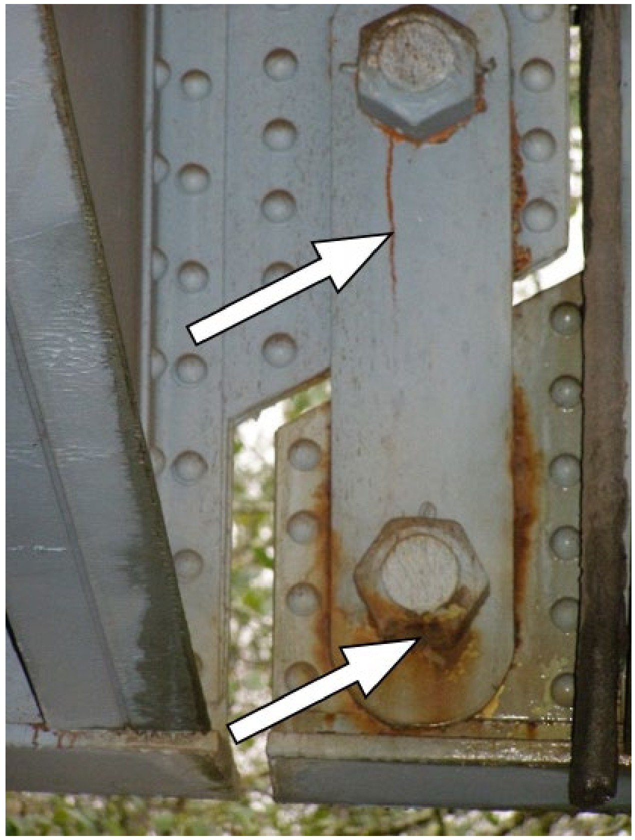


Figure 10.4.30 Rust Stains from Pin Corrosion

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 many portions of the assembly are inaccessible, they are not typically 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. An inspection team should try to determine whether the hanger-pin connection is frozen, or not moving, as this can induce large bending moments in the hanger plates.

Accessible surfaces and edges should be examined closely for cracks (see Figure 10.4.31). The most critical areas are the ends of the eyebar starting at the pin centerlines and the juncture between the heads and shanks of eyebars. Surface condition and section loss should be noted.

Figure 10.4.31 Cracked Forge Zone on an Eyebar

Both sides of the plate should be examined for cracks due to bending of the plate from a frozen pin connection. The amount of corrosion buildup between the webs of the girders and the back faces of the plates should be observed.

The hanger plate should be inspected for bowing or out-of-plane distortion from the webs of the girders (see Figure 10.4.32). Any welds should be investigated for cracks. If the plate is bowed, inspectors should carefully check around the point of maximum bow for cracks that might be indicated by a broken paint film and corrosion. The distance between the back of the hanger and the face of the web should be measured at several locations. These measurements should be compared 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.

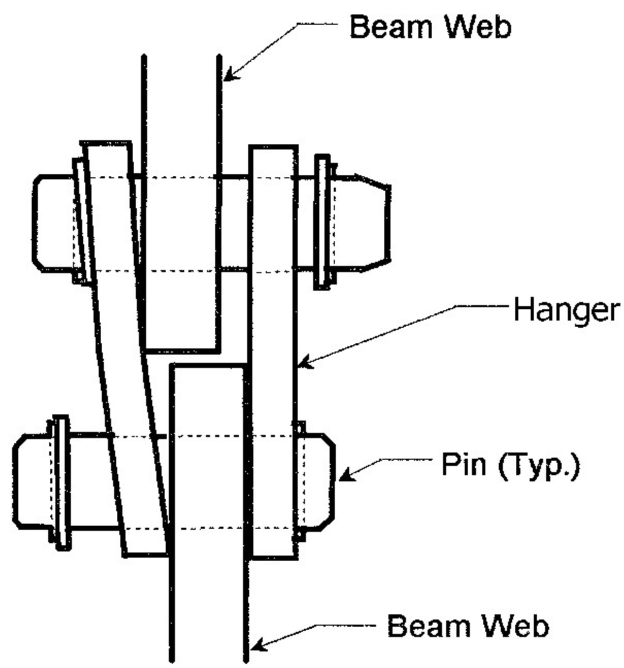


Figure 10.4.32 Bowing Due to Out of Plane Distortion of Hanger

Pins

Inspectors should measure the pin diameter and note if more than 1/8-inch net section loss of the diameter has occurred (see Figure 10.4.33). 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 lateral slippage may be indicated by misalignment of the deck expansion joints or surface over the hanger connection.

Figure 10.4.33 Corroded Pin-and-Hanger Assembly

In a fixed pin and hanger, wear will generally be on the top surface of the pin due to rotation from live load deflection and additional 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. Inspectors should compare this distance to the plan dimensions to determine the decrease in the pin diameter. Due to the typical configuration of a pin and hanger, the majority of the pin cannot be accessed, so NDE methods such as UT should be done to check the actual condition of the pin.

If the assembly includes a pin cap, the pin cap should be checked for flatness with a straight edge.

Critical Areas

The critical areas most likely to develop cracks are outlined below and shown in Figure 10.4.34:

- At welds used to connect hanger plates.
- At welds used to connect web doubler plates.
- In the base metal at the net section of eyebar head.
- In the base metal at the juncture between heads and shanks of eyebars.

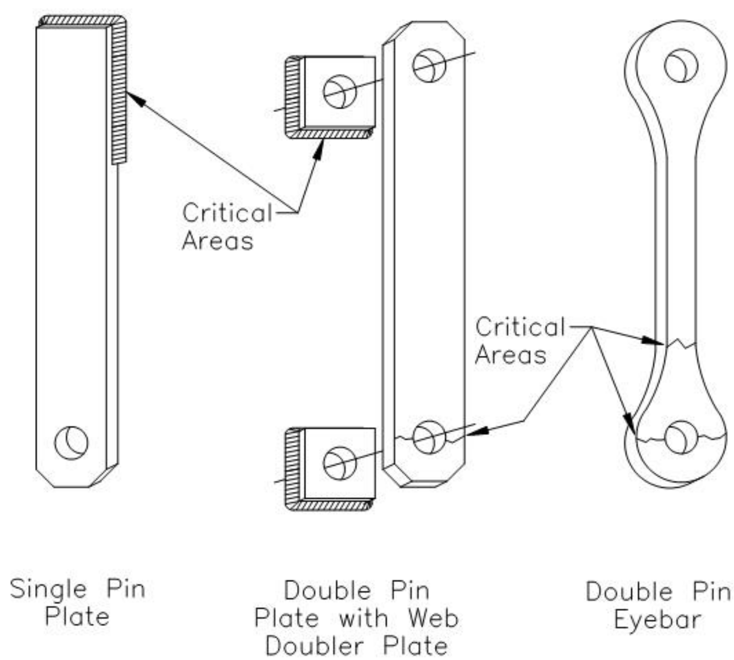


Figure 10.4.34 Critical Areas: Potential Fatigue Crack Locations in Pin-and-Hanger

10.4.12 Gusset Plates

Deficiencies that are recorded include misaligned connections/dimensions, corrosion (section loss), fatigue cracking, tack welds, paint failures, fastener condition, presence of repairs or retrofits, and out-of-plane distortions. Current dimensions should be checked against original design or as-built conditions (see Figure 10.4.35).

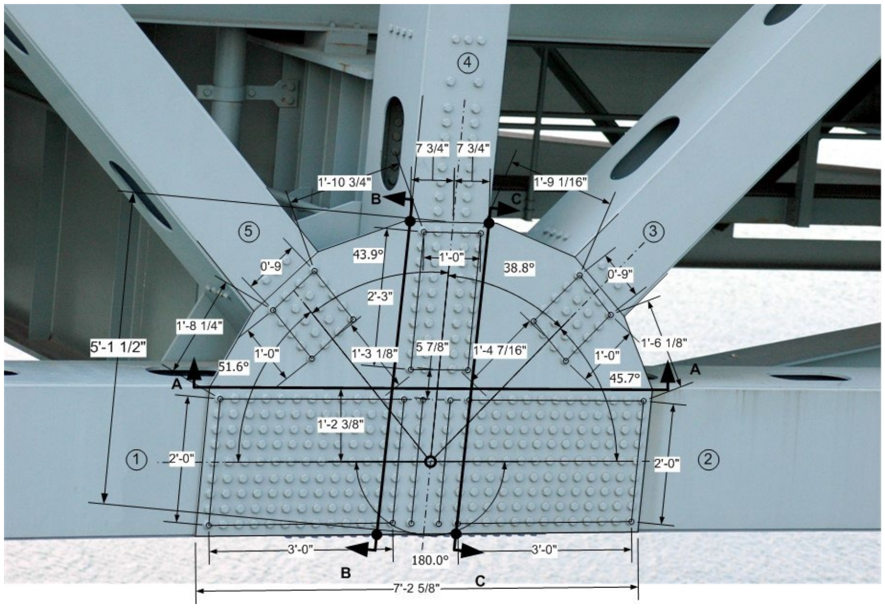


Figure 10.4.35 Gusset Plate Field Measurements

Areas with Corrosion

Surface corrosion may occur on gusset plates and can lead to section loss (see Figure 10.4.36). 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 (see Figure 10.4.37).

Figure 10.4.36 General Corrosion of Gusset Plates

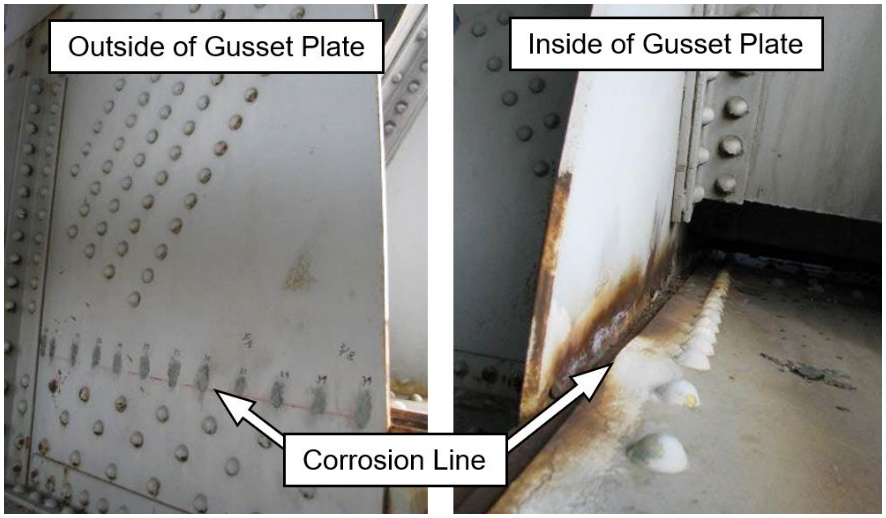


Figure 10.4.37 Corrosion Line Viewed from Inside and Outside of Gusset Plate

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 remaining section (see Figure 10.4.38). A straight edge is utilized in conjunction with the probe to obtain the amount of section loss.

Figure 10.4.38 Using Calipers to Measure Gusset Plate Thickness

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 10.4.39).

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 10.4.39 Using a Straightedge to Measure Gusset Plate Section Loss

In addition to the D-meter, caliper or depth probe, and tape measure, or a visual weld acceptance criteria (V-WAC) gage may also be used to determine section loss. The V-WAC is used to measure section loss and then subtracting from the total thickness to determine the thickness of the plate that is left (see Figure 10.4.40). It can only measure up to one-quarter inch section loss.

Figure 10.4.40 Using V-WAC in the Field to Measure Gusset Plate Section Loss

Advanced inspection systems may be used to document cracks, flaws, corrosion, and internal anomalies in steel gusset plates. Refer to Chapter 17.

Areas Susceptible to Fatigue Cracking

Common locations for fatigue cracks to develop include bolt holes and rivet holes. The rough edges of fastener holes 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 (see Figure 10.4.41).

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 more susceptible to cracking then high strength bolts.

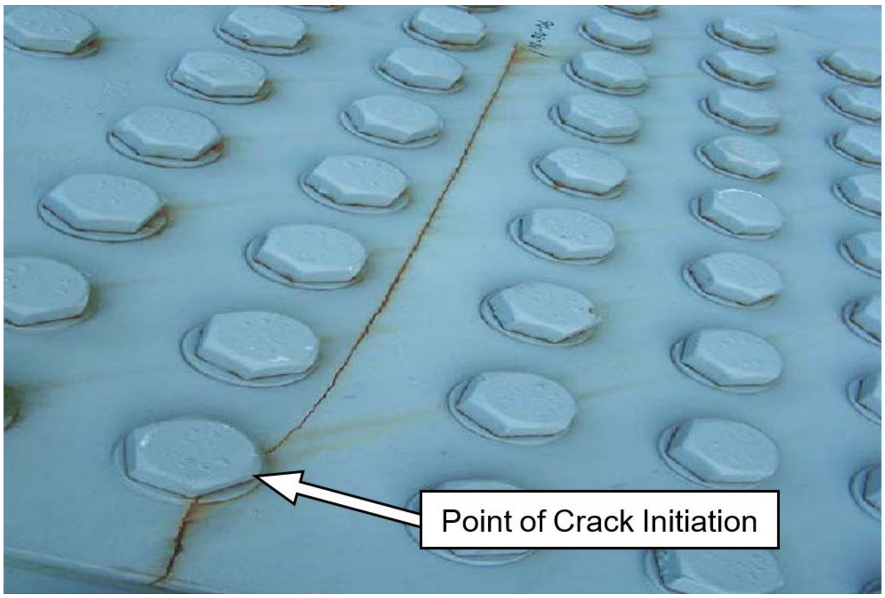


Figure 10.4.41 Cracked Gusset Plate and Point of Crack Initiation

Areas with Tack Welds

A tack weld on a tension member is considered a fatigue-prone detail because when or if the tack weld cracks, the potential for the crack to propagate into the base metal of the tension member exists (see Figure 10.4.42). Tack welds were historically utilized during construction as a way to hold plates in place before riveting. They were not eradicated, as it was not known that they would cause issues in the future. Tack welds are no longer common practice unless they are to be included in the final design weld.

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 nonredundant steel tension members, such as gusset plates, has the potential to cause partial or total bridge collapse. These cracks can also propagate into other primary tension members.

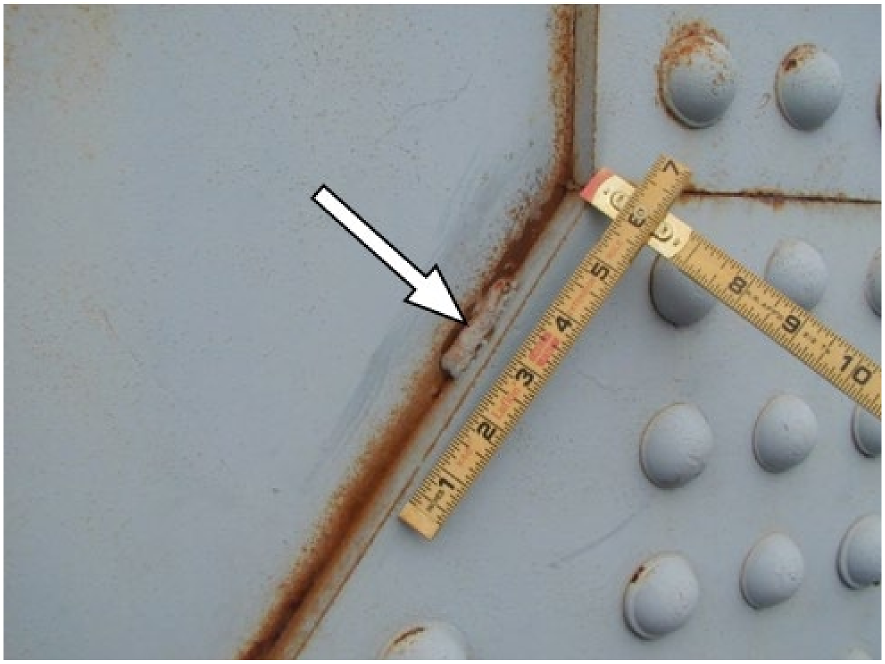


Figure 10.4.42 Partial Length Tack Weld

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) (see Figure 10.4.43). If section loss is present, gusset plates will be more susceptible to overstress due to a reduced capacity. The capacity is reduced because less material is available to distribute the tension, shear, or compression loads.

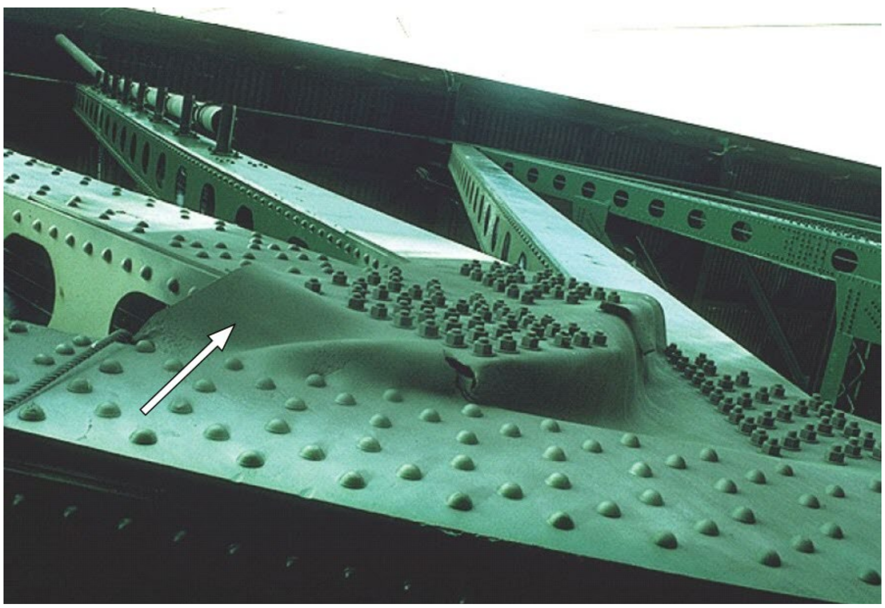


Figure 10.4.43 Gusset Plate Buckling Failure due to Significant Section Loss

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, fabrication error, erection error, or increased dead load.

A straight edge or string line is used to evaluate and quantify any distortion of the unbraced gusset plate edges between members (see Figure 10.4.44). If gusset plates exist on both sides of a given truss or arch member, check both gusset plates for out-of-plane distortion.

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.

Figure 10.4.44 Gusset Plate Out-of-Plane Distortion

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 10.4.45). If the head is still intact, this overstress can be visually observed as out-of-plane rotation of the rivet head.

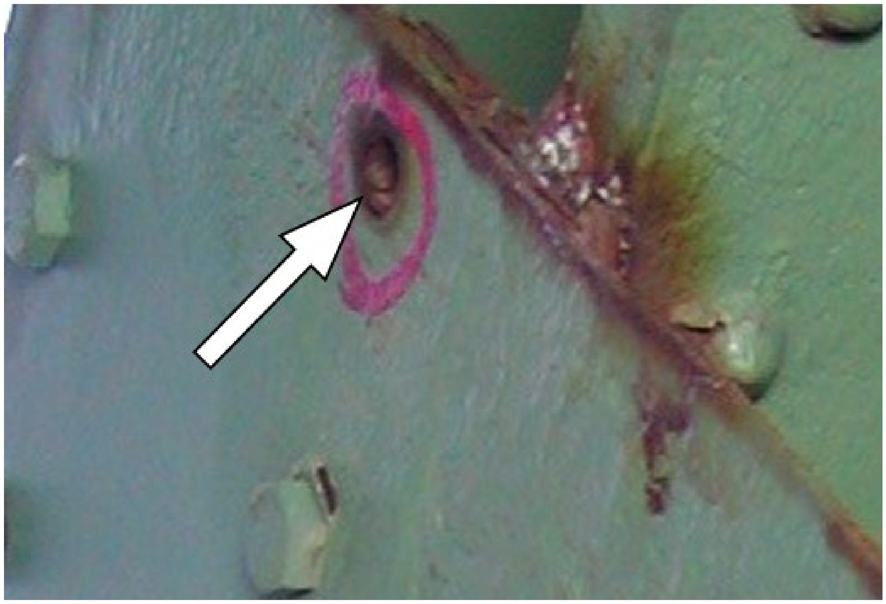


Figure 10.4.45 Missing Bolts on a Gusset Plate

The riveted or bolted connection should be inspected 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. Inspectors should check to assure the fastener number and pattern is consistent with the as-built or construction plans.

For welded gusset plate retrofits, the toe of the weld and base metal should be closely examined for signs of cracking.

Repair and Retrofits

All repairs and retrofits should be inspected for distortion, deterioration, pack rust and tack welds as a means to verify that the repairs and retrofits are functioning as intended.

10.4.13 Eyebars

Forge Zone

Inspectors should carefully examine the forged area around the eyebar head and the shank for cracks. The loop rods should be checked for cracks where the loop is formed (see Figure 10.4.31 and Figure 10.4.46). Most eyebar failures are likely to occur in the forge zone.

Figure 10.4.46 Cracked Forge Zone on a Loop Rod

Tension Zone

Since an eyebar carries axial tension, inspectors should 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. Reference Section 10.4.4 for detailed information on inspection areas and deficiencies of axial tension members.

Alignment and Load Distribution

The alignment of the shank along the full length of the eyebar should be checked. The eyebar should be straight since it is a tension member. A bowed eyebar indicates that a compressive force has been introduced (see Figure 10.4.47).

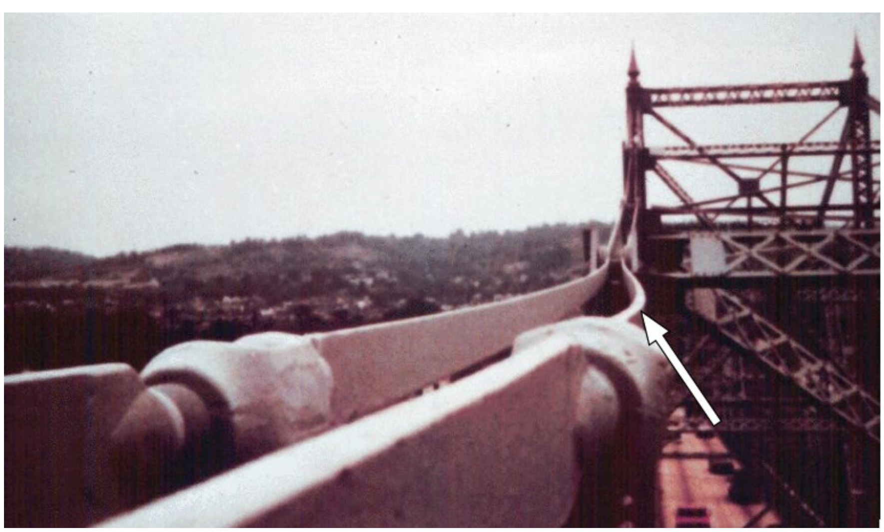


Figure 10.4.47 Bowed Eyebar Member

Misalignment due to buckling can also be caused by movement at the substructure or changes in loading during rehabilitation (see Figure 10.4.48). Eyebars of the same member are designed to be parallel and evenly loaded.

Figure 10.4.48 Buckled Eyebar due to Abutment Movement

Spacers

Inspectors should examine the spacers on the pin connections to be sure they are holding the eyebars in their proper position (see Figure 10.4.49).

Spaced eyebars should be closely examined at the pin for corrosion build-up (pack 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. Inspectors should verify that the eyebars are symmetrical about the central plane of the spacer and not twisting or distorting.

Figure 10.4.49 Spacer with Unsymmetrical Eyebars

Load Distribution

Inspectors should check to determine if any eyebars are loose (unequal load distribution) or if they are frozen at the ends - preventing free rotation. Panel point pins or eyebars should be checked for twisting (see Figure 10.4.50 and Figure 10.4.51).

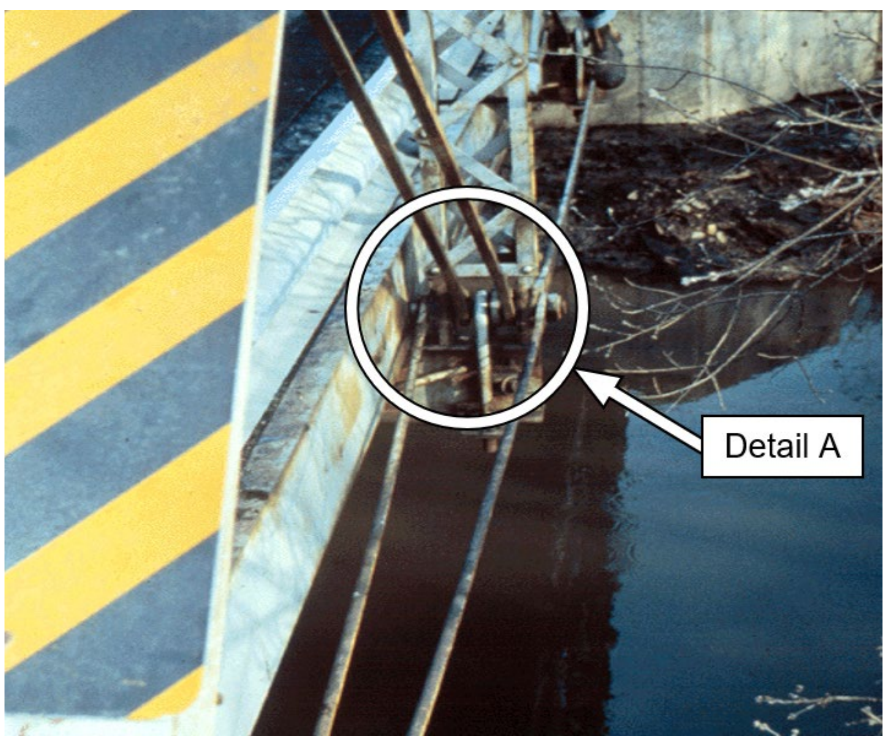


Figure 10.4.50 Eyebar Member with Unequal Load Distribution

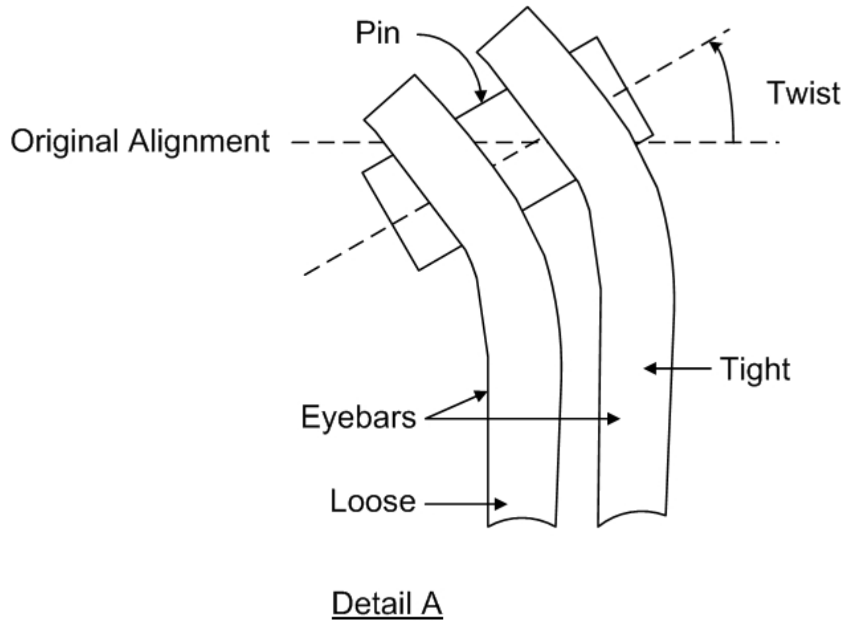


Figure 10.4.51 Detail A - Eyebar Member with Unequal Load Distribution

Weldments

The integrity of any welded retrofits or repairs to the eyebar should be evaluated (see Figure 10.4.52 and Figure 10.4.53). Any field-applied welds used in repairing or strengthening the eyebar should be checked, as well as welds for utility supports. Most of these bridges are old and constructed of steel that is considered "unweldable" by today's standards. It is difficult to obtain a high quality "field" weld. Reference American Welding Society (AWS) Standards for more information.

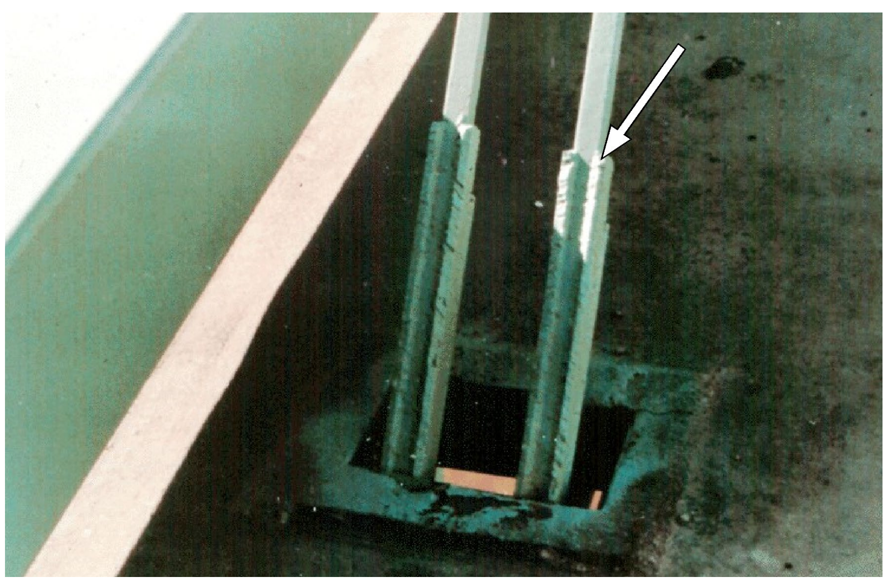


Figure 10.4.52 Weld on Loop Rods

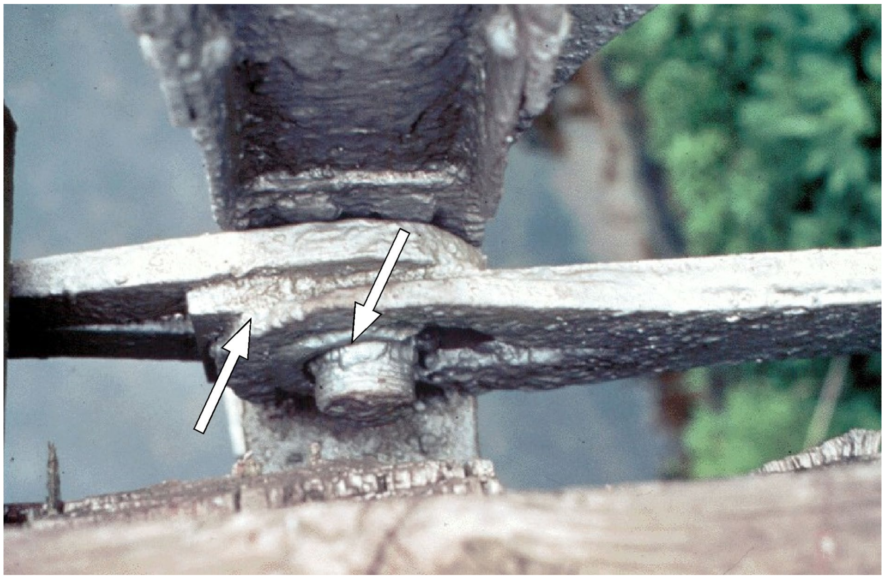


Figure 10.4.53 Welded Repair to Loop Rods

Turnbuckles

Any threaded rods in the area of the turnbuckle should be examined for corrosion, pack rust, tack welds, cracks, wear, and repairs (see Figure 10.4.54 and Figure 10.4.55). The threaded portion of the rod should be inspected for signs that the turnbuckle is loosening. Turnbuckles are often found in counter diagonals.

Figure 10.4.54 Turnbuckle on a Truss Diagonal

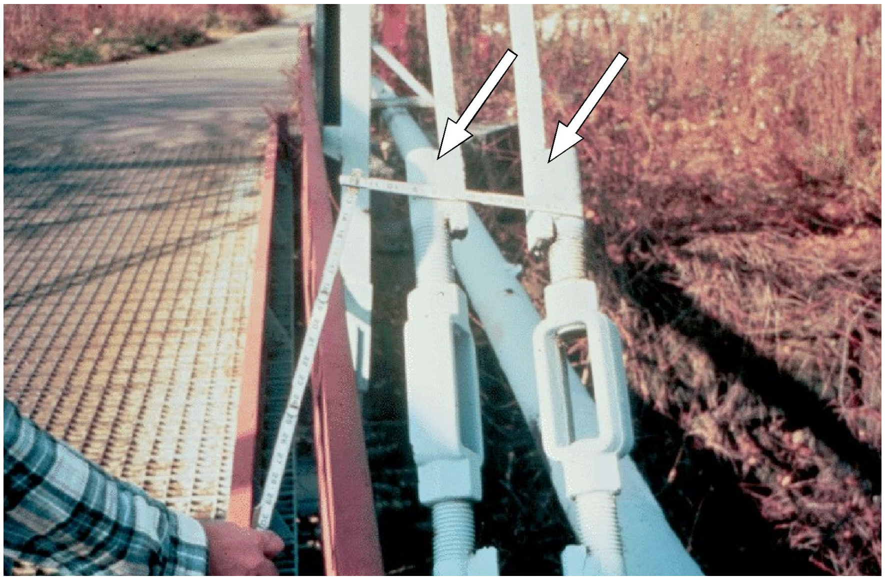


Figure 10.4.55 Welded Repairs to Turnbuckles

Pins

Pins should be inspected for signs of cracking, wear, and corrosion. Nondestructive methods, ultrasonic testing for example, are recommended since visual inspection cannot reveal internal material flaws that may exist (see Figure 10.4.56).

Figure 10.4.56 Ultrasonic Inspection of an Eyebar Pin

10.4.14 Converted Railroad Car Bridges

Repurposed railroad flat cars may have existing deficiencies before being used as a bridge structure. For instance, it is possible for a car to have members that are bent from their use in the railroad (see Figure 10.4.57). The bridge file should be reviewed for this information. Many times, the car is removed from service with the railroad because it has reached its fatigue limit, so this is particularly true with regards to fatigue-prone details.

Figure 10.4.57 Existing Distortion to Channel Beam

When multiple railroad cars are constructed next to each other, special observation should be given to the interface between cars. In some cases, the cars are attached by field welding or some other method and sometimes there is no connection between the adjacent cars. If there is no connection, the cars may deflect independently and cause cracking of the deck at the interface (see Figure 10.4.58).

Figure 10.4.58 Railroad Car Converted Bridge Deck with Cracking

Because railroad cars were not intended to be used to span the length of the car itself, the steel members were not designed by AASHTO Standards for flexure, shear, or bearing in the same way that a bridge girder would be designed. The ends of the car are typically the shallowest portions of the car (see Figure 10.4.59), which is not desirable for shear or bearing resistance. Areas above the bearing area should be inspected for web crippling due to both deterioration and undersized webs. In addition to these issues, the railroad flat cars are generally placed on a substructure unit without a bearing assembly (see Figure 10.4.60 through Figure 10.4.62). This can cause distortion of both the flat car member and the substructure member.

Figure 10.4.59 Flat Car with Shallow Ends

Figure 10.4.60 Flat Car Bearing Directly on Bent Cap

Figure 10.4.61 Flat Car Channel Beam Bearing on Bent Cap with Keeper Plate

Figure 10.4.62 Flat Car Channel Beam and Bent Cap with Distortion

10.4.15 Secondary Members

Secondary members (see Figure 10.4.63) should be examined for distortion, collision damage, corrosion, pack rust connection areas, cracked welds, fatigue cracks, and loose fasteners. Horizontal connection plates can trap debris and moisture and are therefore susceptible to a high degree of corrosion and deterioration. Distorted secondary members may be an indication the primary members may be overstressed, or the substructure may be experiencing differential settlement.

Figure 10.4.63 Sway Bracing with Pack Rust

For steel truss arch secondary members, inspectors should check for collision damage at the portals, sway bracing and at knee braces (see Figure 10.4.64).

Figure 10.4.64 Collision Damage to Portal Bracing

10.4.16 Miscellaneous Areas

Fatigue-prone 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.
- Back-up bars.
- Mechanical fasteners and tack welds.
- Built-up member connections: lacing bars, stay plates, batten plates (see Figure 10.4.65).

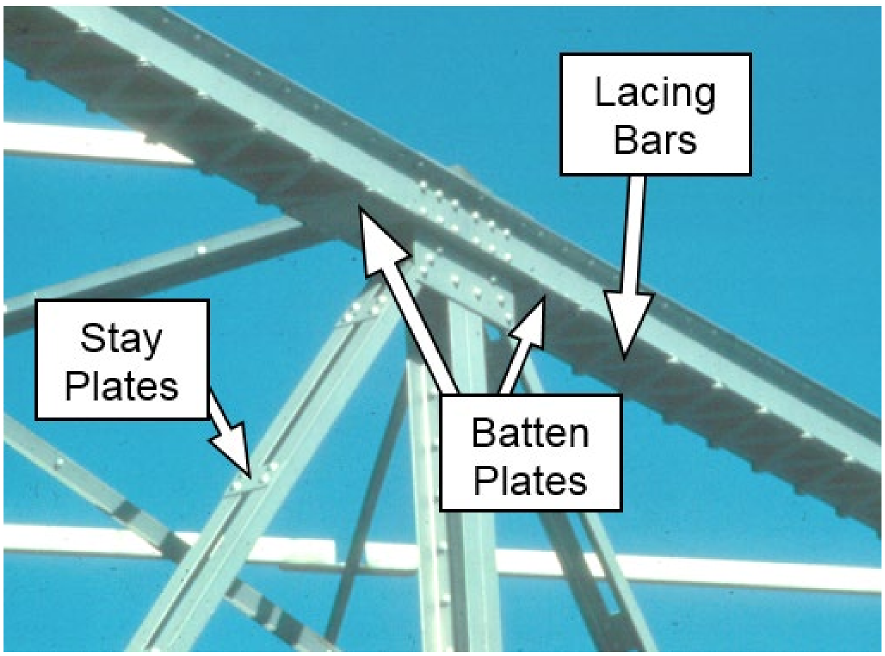


Figure 10.4.65 Other Truss Members

Note that out-of-plane bending may occur at the following areas, which are presented in additional detail further in Chapter 7:

- Girder web connections for diaphragms and floorbeams.
- Staggered floorbeams or lateral gusset plate locations (for skewed bridges).
- Gusset plates.
- Cantilevered floorbeams.

In addition to common fatigue-prone details, steel members should be inspected in the following locations:

- Stiffeners (transverse and longitudinal).
- Groove welded butt splices.
- Lateral bracing gusset plates.
- Web-to-flange welds.
- Miscellaneous connections (railing and utilities).
- Flanges that terminate before the end of the web.
- Coped flanges.
- Blocked flanges.

10.4.17 Protective Systems Failure

Steel superstructures are typically protected from corrosion by painting or using weathering steel. The failure of a coating system can eventually lead to corrosion and section loss on steel members. Refer to Chapter 7 for inspection areas and deficiencies for protective systems.

Section 10.5 Lessons Learned

10.5.1 Silver Bridge

The following information can be found at:
https://transportation.wv.gov/highways/bridge_facts/Modern-Bridges/Pages/Silver.aspx.

The Silver Bridge was an eyebar-chain suspension bridge that was built in 1928 over the Ohio River. The bridge carried traffic from Point Pleasant, West Virginia to Gallipolis, Ohio. It was in service for less than 40 years when it collapsed during rush hour on December 15, 1967 (see Figure 10.5.1). Forty-six people were killed in the collapse.

Figure 10.5.1 Silver Bridge Failure

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 typically 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 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 increased stress on the other members of the bridge. The remaining steel frame collapsed.

A series of laws, beginning with the Federal Aid Highway Act, began to put in place the United States' contemporary bridge inspection program prior to the collapse in 1967, but the failure of the Silver Bridge led to the final creation of the National Bridge Inspection Program (NBIP), which modernized the approach to bridge inspection nationally. The National Bridge Inspection Standards (NBIS) established the need for this manual, along with consistent inspection methods and inspection frequencies used throughout the United States. Prior to the development of the NBIS, states had been responsible for their own inspection programs and standards.

10.5.2 Mianus River Bridge

The following information can be found at: https://connecticuthistory.org/mianus-river-bridge-collapses-today-in-history/.

On June 28, 1983, a 100-foot-long span of the Mianus River Bridge in Greenwich, Connecticut collapsed into the river (see Figure 10.5.2 and Figure 10.5.3). The portion of the bridge that failed was a suspended span, in which each corner was attached to the girders of the cantilever arm of the adjacent anchor spans with pin and hanger assemblies.

Figure 10.5.2 Mianus River Bridge Failure - From Deck

Figure 10.5.3 Mianus River Bridge Failure - From Ground

The inside hanger in the southeast corner of the span was pushed off of the inside end of the lower pin due to extensive rusting. This action shifted the entire weight of the southeast comer of the span onto the outside hanger. The outside hanger gradually worked its way farther outward on the pin, and over a period of time, a fatigue crack developed in the pin. The shoulder of the pin fractured off and the pin and hanger assembly failed, causing the suspended span to fall into the river. An emerging national emphasis on fatigue and bridges with NSTMs was instituted due to the collapse of the Mianus River Bridge.

10.5.3 I-35W Mississippi River Bridge

The following information can be found in the National Transportation Safety Board (NTSB) report, *Highway Accident Report – Collapse of I-35W Highway* Bridge as well as the MNDOT website here: https://www.dot.state.mn.us/i35wbridge/index.html.

On 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 10.5.4 and Figure 10.5.5). As a result of this tragedy, thirteen people died, and 145 people were injured.

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 necessary thickness. 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.

Figure 10.5.4 I-35W Mississippi River Bridge Collapse

Figure 10.5.5 Collapse of the I-35W Mississippi River Bridge

The NTSB concluded that the bridge was designed with undersized gusset plates and the riveted gusset plates 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.

10.5.4 I-276 Delaware River Bridge

The following information can be found in the *Delaware River Turnpike Bridge Fracture Repair Report* by Richard Schaeffer and Frank A Corso Jr.

The Delaware River-Turnpike Toll Bridge carries four lanes of traffic across the Delaware River between New Jersey and Pennsylvania. It was previously considered part of Interstate 276 but is now a part of I-95 as of 2018. The bridge was constructed in 1956 and is comprised of a 3-span continuous arch truss with a center suspended span of 682 ft between the piers flanking the river channel. The approach spans on both sides are three and four span continuous deck truss units, as well as a number of traditional short, simple girder-floorbeam spans. The bridge was closed to traffic in January of 2017 after a bridge inspection revealed a complete fracture in the tension zone of a top chord truss member of one of the four-span continuous deck trusses (see Figure 10.5.6 and Figure 10.5.7). The member is a "jumbo" rolled beam composed of high strength manganese steel. Retrofits were made to the member and the bridge was re-opened in early March 2017.

It was later determined that the fracture was initiated at a plug weld. Plug welds were commonly used in the 1950s as a means of filling unused rivet holes. Because they are a different material than the beam itself, stress risers are created and cracking around welds is common. Undetected cracks can propagate very quickly, especially in bridges with a high ADT because the members are experiencing repeated loadings. The Delaware River-Turnpike Toll bridge incident could have been much worse if the issue had not been discovered. As a result of this event, a strong push was made to inventory, inspect, and document all plug welds in future bridge inspections.

Figure 10.5.6 Delaware River-Turnpike Toll Bridge

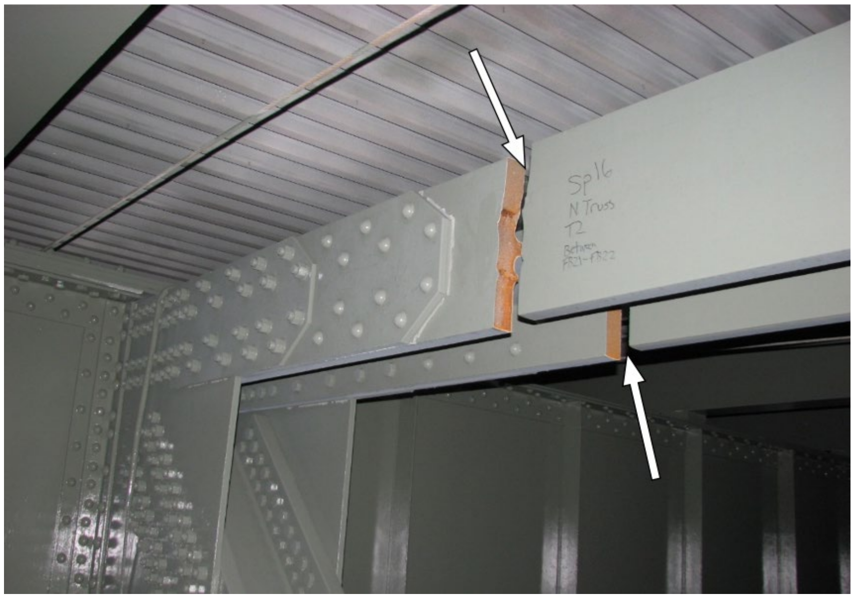


Figure 10.5.7 Fractured Top Chord Truss Member

10.5.5 Hoan Bridge

The following information can be found in the FHWA Memorandum *Hoan Bridge Failure Investigation*.

The Hoan Bridge in Milwaukee, Wisconsin is a steel superstructure that carries Interstate 794 over the Milwaukee River. In December of 2000 the bridge was closed after an approach span was observed to have dropped approximately a foot from the adjacent span. The approach span consisted of a three-girder system, in which cracks formed in all three girders, with fractures propagating completely through two girders (see Figure 10.5.8).

Figure 10.5.8 Fractured Girder

The primary cause of fracture initiation in the Hoan Bridge was determined to be related to the design details used for the welded joint assembly where the lateral bracing frames into the web. The joint was detailed with a narrow web gap that caused local high constraint, increased stiffness, and reduced the apparent fracture resistance. As detailed on original plans, the joint has only 1/8 inch separating the welds on the two plates. The fabrication tolerance resulted in reduced gaps as well as intersecting welds in many locations throughout the structure. Stress analysis showed that the intersecting welds increased the rigidity of the joint and made the constraint problem worse. This non-ductile behavior in the joint caused by a triaxial constraint and state of stress has never before been documented as being a potential problem in bridge detailing.

The failure was ultimately due to the narrow gap between the gusset plate and the transverse connection/stiffener plate that created a local triaxial constraint condition and increased the stiffness in the web gap region at the fracture initiation site. This constraint prevented yielding and redistribution of the local stress concentrations occurring in this region. As a result, the local stress state in the web gap was forced well beyond the yield strength of the material and the steel fractured.

As a result of this failure, major rehabilitation efforts were made to retrofit this type of detail, both on the Hoan Bridge and throughout the country.

10.5.6 Sherman Minton Bridge

The Sherman Minton Bridge is a double-deck tied arch truss built in 1962 that carries I-64 over the Ohio River between Kentucky and Indiana (see Figure 10.5.9). A number of members throughout the structure have been identified as NSTMs and a number of the members were fabricated using AASHTO M270 Grade 100 Steel, also known as “T-1” steel. This material has since been known to be highly susceptible to hydrogen cracking after welding processes. On September 9, 2011, the bridge was ordered to be closed after construction crews discovered cracks propagating from butt welds in both tie girders. During the subsequent investigation (see Figure 10.5.10), it was found that these cracks were present since construction but were not externally visible. It was concluded that the cracks were likely caused by hydrogen cracking during welding and fabrication processes. FHWA released a Technical Advisory *Inspection of Fracture Critical Bridges Fabricated from AASHTO M270 Grade 100 (ASTM A514/A517) Steel,* which can be found here: https://www.fhwa.dot.gov/bridge/t514032.pdf.

Figure 10.5.9 Sherman Minton Bridge

Figure 10.5.10 Magnetic Particle Testing on Sherman Minton Bridge

10.5.7 Hernando de Soto Bridge

The following information can be found in the FHWA Memorandum *Non-Destructive Testing of Fracture Critical Members Fabricated from AASHTO M244 Grade 100 (ASTM A514/A517) Steel* as well as the ArDOT After Action Report *I-40 Hernando DeSoto Bridge Emergency Repair and Inspection*.

The I-40 Hernando DeSoto bridge is a tied arch truss bridge built in 1973 that spans the Mississippi River between West Memphis, Arkansas and Memphis, Tennessee. The bridge was constructed with a number of members that have been identified as NSTMs within the arch truss, bottom chord, and floor system. Several members in the arch, along with the entire bottom chord, were fabricated using AASHTO M244 Grade 100 (T-1) Steel. Like the Sherman Minton bridge, Hernando DeSoto is a very similar type of structure fabricated with nearly identical materials. On May 11, 2021, a large fracture was discovered in the bottom chord during an NSTM inspection of the arch. Upon further inspection, the inspectors discovered that less than half the cross section remained at the location of the crack and the member exhibited out-of-plane distortion (see Figure 10.5.8). The inspection team acted quickly to notify authorities and close the bridge as well as the river below. The subsequent investigation revealed that the fracture originated from hydrogen cracking during fabrication and had gone unnoticed for at least two inspection cycles. Several phases of repairs were designed and the bridge was reopened on August 2, 2021.

Figure 10.5.11 I-40 Hernando DeSoto Bridge Fracture

FHWA released a memorandum (*Non-Destructive Testing of Fracture Critical Members Fabricated from AASHTO M244 Grade 100 (ASTM A514/A517) Steel)* with new requirements for all State transportation departments regarding T-1 steel. The State responsible for past bridge inspections also released an After Action Report detailing organizational and procedural changes to improve future inspections including more robust QC and QA processes and the use of nondestructive evaluation methods. The After Action Report can be found here: http://www.ardot.gov/wp-content/uploads/2021/11/ARDOT-After-Action-Report-I-40-MS-Rvr-Bridge.pdf.

Section 10.6 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of steel bridges. The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 11 TABLE OF CONTENTS

CHAPTER 11 LIST OF FIGURES

This page intentionally left blank.

Chapter 11 Inspection and Evaluation of Timber, Masonry, and FRP Superstructures

Section 11.1 Introduction

For more information on timber bridges, please refer to the following documents:

- U.S. Forest Service. *Timber Bridges Design, Construction, Inspection, and Maintenance*
- U.S. Forest Service. *Wood Bridges: Decay Inspection and Control*
- USDA. *Timber Bridges Design, Construction, Inspection and Maintenance*

For more information on FRP bridges, please refer to the NCHRP *Report 503: Application of Fiber Reinforced Polymer Composites to the Highway Infrastructure* as well as the NCHRP *Report 564: Field Inspection of In-Service FRP Bridge Decks*.

Though timber, masonry, and fiber reinforced polymer (FRP) are materials less commonly used in bridge construction, there are various bridges throughout the country that utilize these superstructure types. It is important for inspectors to understand the design characteristics and inspection methods for these types of bridges. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

Section 11.2 Characteristics

Sections 11.2.1 through 11.2.5 describe the characteristics of various timber and miscellaneous other superstructures. Refer to Chapter 4 for additional information on basic characteristics and standard timber structural shapes.

11.2.1 Solid Sawn Timber Superstructures

Design Characteristics

As discussed in section 7.4, a solid sawn member is a section of a single tree log cut to the desired size at a sawmill. These members can be used to create different bridge types, but solid sawn multi-beam bridges are the simplest type of timber bridge (see Figure 11.2.1).

Figure 11.2.1 Elevation View of a Solid Sawn Timber Bridge

Multi-Beam Bridges

Solid sawn multi-beam bridges consist of multiple solid sawn beams spanning between substructure units (see Figure 11.2.2). The deck is supported by the beams and is typically comprised of transversely laid timber planks or nail-laminated deck units, and longitudinally laid planks called runners may serve as a wearing surface. Sometimes a bituminous or gravel 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 20 inches, and the beams are usually spaced about 2 ft on center.

Figure 11.2.2 Underside View of a Solid Sawn Multi-Beam Bridge

This bridge type is generally used in older, shorter span bridges, spanning up to about 25 ft. Shorter spans are sometimes combined to form longer multiple span bridges and trestles. Older multiple span timber beam bridges supported on timber bents or towers are often referred to as trestles. Solid sawn timbers are rarely used for modern bridges today due to the development of high-quality glulam members.

Covered Bridges

Covered bridges are generally found along rural roads, and their name reflects the walls and roof which protect the bridge superstructure (see Figure 11.2.3 and Figure 11.2.4). Some still carry highway loads, but many are only open to pedestrians or restricted to light weight vehicles. While most covered bridges were built during the 1800s and early 1900s, there are a number of covered bridges being built today as historic reconstruction projects.

Figure 11.2.3 Elevation View of Covered Bridge

Figure 11.2.4 Inside View of Covered Bridge Showing Truss Members

The majority of covered bridges have trussed superstructures (see Figure 11.2.5). The covers on the bridges tend to prevent decay of the truss and are largely responsible for their longevity. There are many "false" covered bridges that are supplemented by support girders underneath the bridge that reduce the loading on the truss. These support girders may also be added in response to deterioration, usually decay of the floorbeam system, or to increase the live load capacity in general; the original capacity of the structure is often less than what would be considered satisfactory by current design standards. Typical truss types for covered bridges include the king post, queen post, Town, Warren, and Howe (see Figure 11.2.6).

The floor system consists of timber floorbeams and stringers (if present). The span lengths of covered bridges generally range from 50 to 100 ft, although many are well over 100 ft and some span over 200 ft.

Figure 11.2.5 Truss Covered Bridge

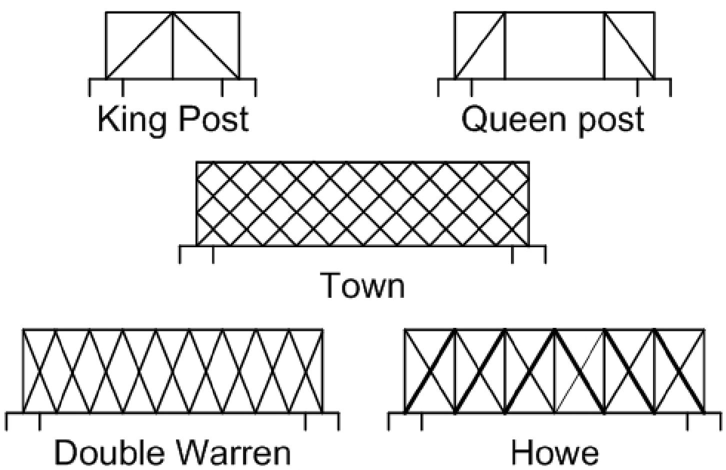


Figure 11.2.6 Common Covered Bridge Trusses

Timber arches were first used in covered bridges by Theodore Burr to strengthen the series of truss configurations typically used in covered bridges. These became known as Burr arch-trusses (see Figure 11.2.7, Figure 11.2.8, and Figure 11.2.9). The arch served as the main supporting member, and the connected king posts carry a portion of the live load, but also serve to stabilize the arch. The span lengths for Burr-arch truss bridges generally range from 50 to 175 ft. Because of their greater strength, many of these structures still exist today.

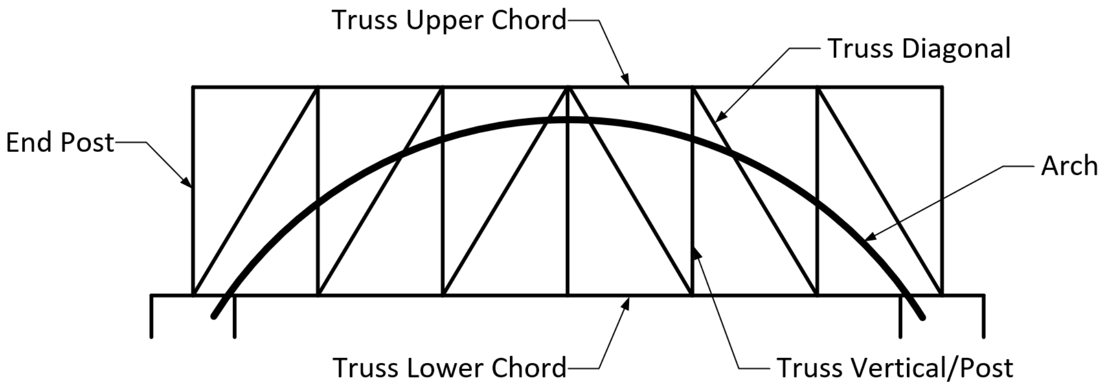


Figure 11.2.7 Schematic of Burr Arch-truss Covered Bridge

Figure 11.2.8 Burr Arch-truss Covered Bridge

Figure 11.2.9 Inside View of Covered Bridge with Burr Arch-Truss Configuration

Primary and Secondary Members

The primary members of solid sawn multi-beam bridges are the beams themselves, and the secondary members are the diaphragms or cross bracing if present (see Figure 11.2.2). These bridges typically have timber diaphragms or cross bracing between beams at several locations along the span.

The primary members in truss and arch structures are the truss members (chords, diagonals, and verticals), arch ribs, stringers, and floorbeams (see Figure 11.2.9 and Figure 11.2.10). 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 11.2.10 Town Truss Design

11.2.2 Glulam Timber Superstructures

Design Characteristics

Glue laminated (glulam) superstructures are typically not as common as solid sawn timber structures, but a significant number have been constructed in recent decades. A glulam member is fabricated by gluing strips of wood 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 properties, since a lower grade of material can be used to fabricate portions of these members.

Many strength-reducing characteristics of wood are minimized due to relatively small laminate dimensions. Any weaknesses due to knots, checks, or grain orientation are confined to a single layer, and do not extend through the full section. Another advantage is 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 Figure 11.2.11).

Figure 11.2.11 Underside View of a Glulam Timber Bridge

Slab Bridges

Glulam timber slab bridges are deck bridges with the timber placed longitudinally with respect to the direction of traffic. Glulam slabs consist of glulam deck panels, which are typically a few feet wide and span in the longitudinal direction of the bridge. The panels are subsequently connected by transverse stiffeners. Glulam slabs are generally only used in short span applications.

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 11.2.12). 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 5ft-6inches to 6ft-6inches on center.

Figure 11.2.12 Elevation View of a Glulam Multi-Beam Bridge

These more modern multi-beam bridges can typically be used in spans of up to 80 ft, although some spans as long as 150 ft have been constructed. 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 similar to steel trusses. Usually, the floor system consists of timber stringers and floorbeams, supporting the deck. The floorbeams are connected to the truss (see Figure 11.2.13). 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 ft (see Figure 11.2.14).

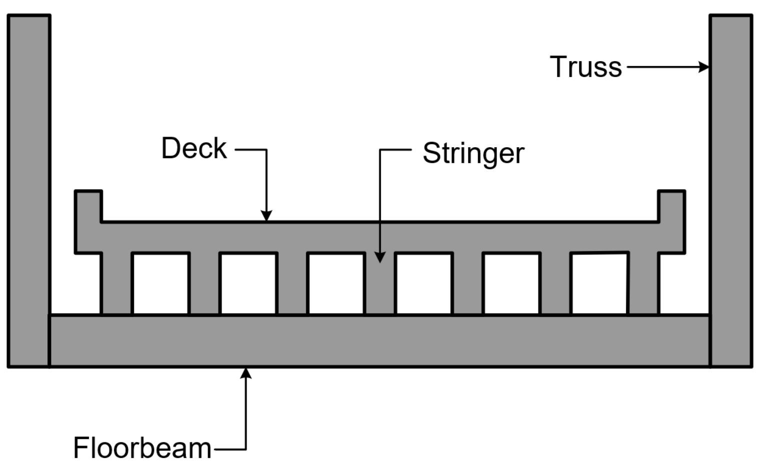


Figure 11.2.13 Timber Through Truss Typical Section

Figure 11.2.14 Timber Through Truss Bridge

Figure 11.2.15 Long Span Timber Through Truss

Arch Bridges

Glulam arch bridges usually consist of two- or three-hinged deck arches, which support a deck and floor system (see Figure 11.2.16 and Figure 11.2.17). Glulam arches are practical for spans of up to about 300 ft. Arches are generally used in locations where aesthetics is important.

Figure 11.2.16 Glulam Arch Bridge over Glulam Multi-Beam Bridge

Figure 11.2.17 Glulam Arch Bridge with Steel Tie Girder

Primary and Secondary Members

The primary members of glulam multi-beam bridges are the beams themselves, and the secondary members are the diaphragms or cross bracing (see Figure 11.2.18). Due to the larger depth of the glulam beams, diaphragms or cross bracing are typically 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 11.2.18 Glulam Diaphragms

11.2.3 Stress-Laminated Timber Superstructures

Stress-laminated timber bridges are rare, especially box beam, tee beam, and k-frame superstructure types. Like glulam superstructures, these are commonly found on local and secondary roads.

Design Characteristics

Stress-laminated timber bridges consist of multiple mechanically clamped planks using metal rods (see Figure 11.2.19). The compression-induced frictional resistance within the timber laminations creates a single structural member with all laminations sharing any loads.

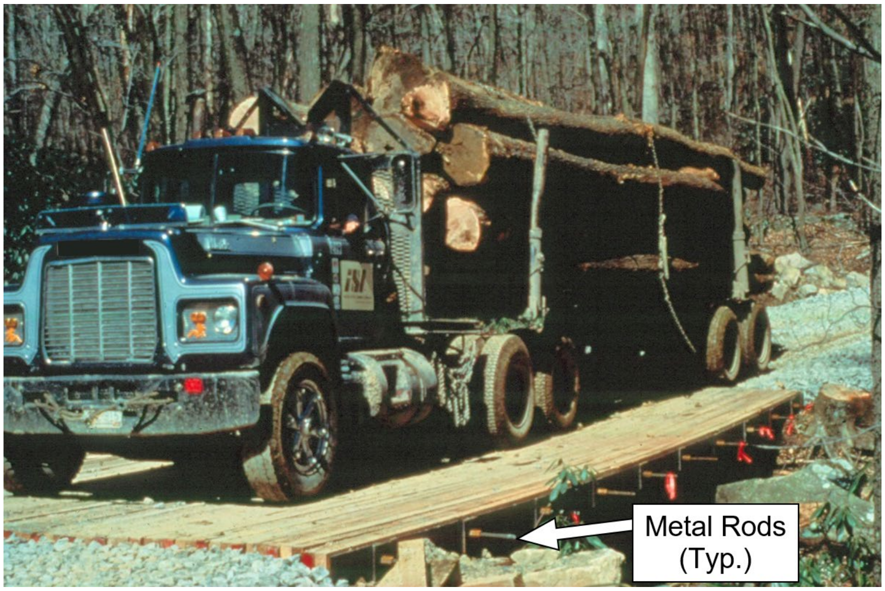


Figure 11.2.19 Stress-Laminated Timber Slab Bridge Carrying a Logging Truck

Slab Bridges

Stress-laminated timber slab bridges can be typically used for simple spans of up to 50 ft and are capable of carrying modern highway loadings (see Figure 11.2.20 and Figure 11.2.21). Stressed deck bridges have also been constructed using glulam members. Combining glulam technology with stress-lamination can increase practical span lengths to 65 ft (see Figure 11.2.22).

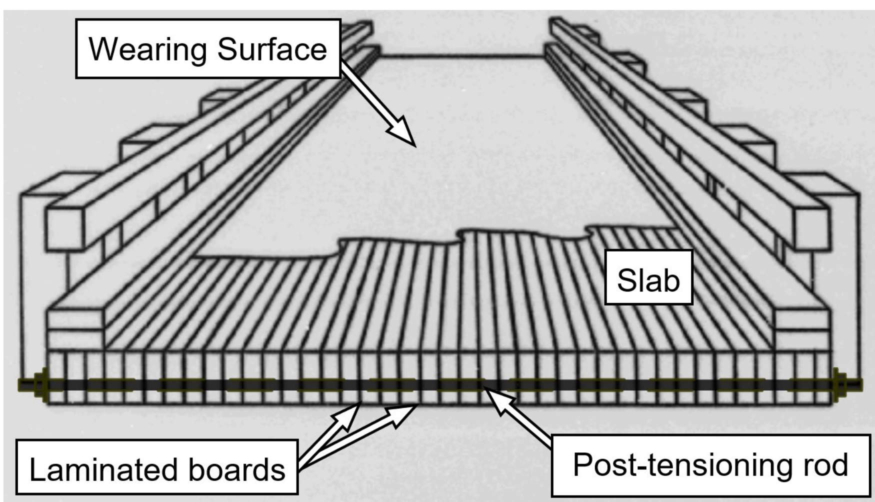


Figure 11.2.20 Typical Section of a Stress-Laminated Timber Slab Bridge

Figure 11.2.21 Stress-Laminated Timber Slab Bridge

Figure 11.2.22 Glulam Stress-Laminated Timber Slab Bridge

Tee Beam Bridges

Tee beam bridges consist of a stress-laminated top flange and glulam beams (see Figure 11.2.23). High strength steel rods are generally used to join the stress-laminated top flange and glulam beams together to form stress-laminated timber tee beams. The average span length for stress-laminated tee beam bridges ranges between 25 and 85 ft (see Figure 11.2.24).

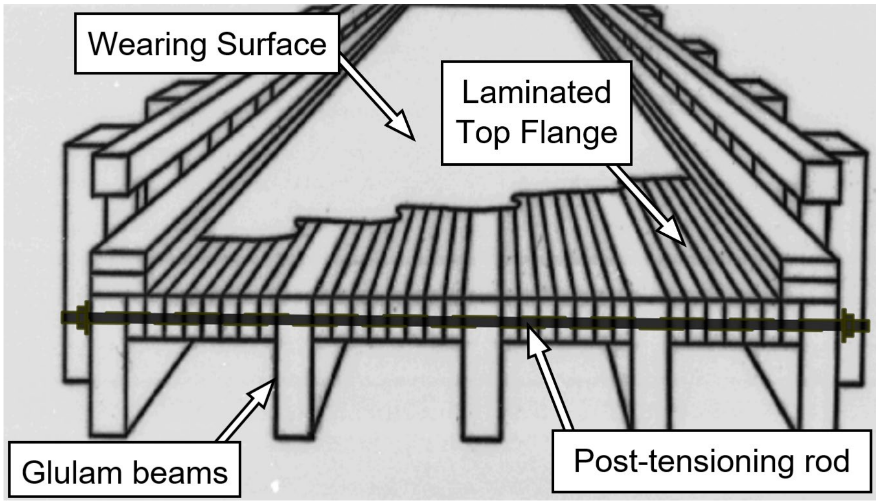


Figure 11.2.23 Typical Section of a Stress-Laminated Timber Tee Beam

Figure 11.2.24 Elevation View of Stress-Laminated Timber Tee Beam Bridge

Box Beam Bridges

Box beam bridges consist of adjacent box beam panels individually comprised of stress-laminated flanges and glulam beam webs (see Figure 11.2.25). This bridge type is also known as a cellular stressed deck. Average span lengths typically range between 35 and 65 ft, but longer span lengths are possible with different configurations (see Figure 11.2.26).

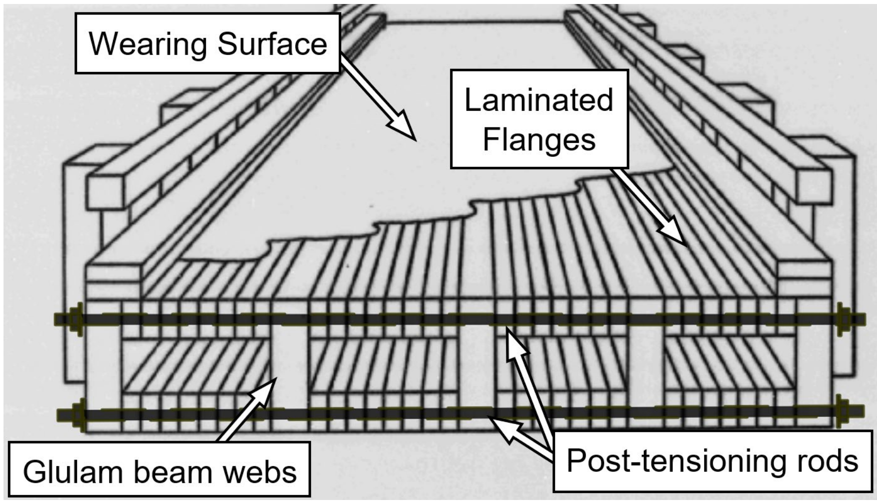


Figure 11.2.25 Typical Section of a Stress-Laminated Timber Box Beam

Figure 11.2.26 Stress-Laminated Timber Box Beam Bridge

K-frame Bridges

Stressed K-frame bridges consist of three spans in which the stressed deck is supported at two intermediate points by stressed laminated timber struts (see Figure 11.2.27). This bridge type has been used for a bridge with a total length of excess of 100 ft, and it has a potential for center span lengths over 50 ft.

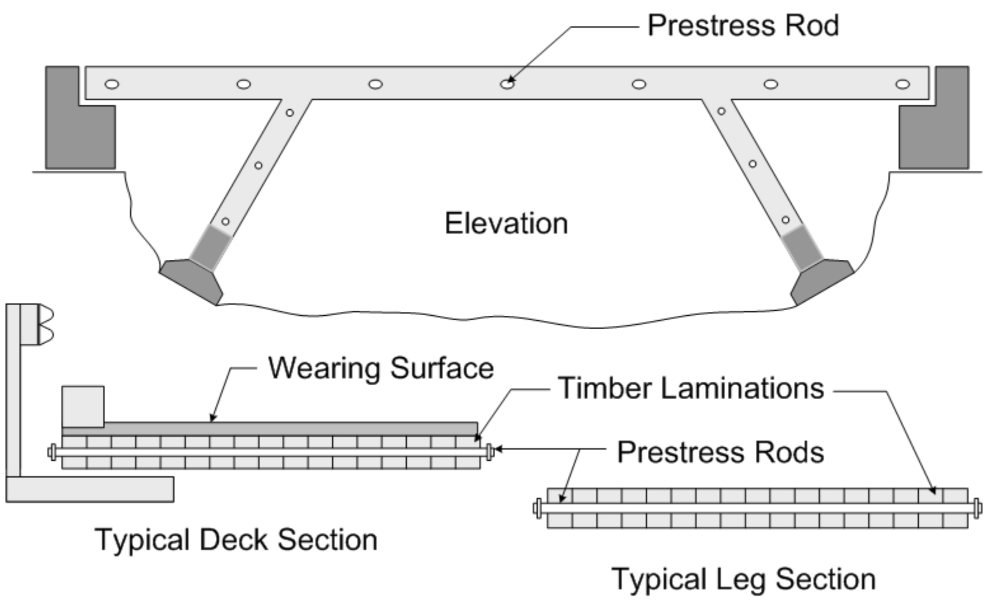


Figure 11.2.27 Stress-Laminated Timber K-frame Bridge

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.

11.2.4 Masonry Superstructures

Masonry is not a common superstructure material, but there are some bridges that are constructed primarily of masonry.

Masonry superstructures include closed spandrel arch structures in which the spandrel wall extends above the roadway surface. Closed spandrel arches have fill material between the roadway and the arch. The fill material is contained by vertical panels, often giving the appearance of a "closed" arch-deck unit (see Figure 11.2.28). Closed spandrel arches may be concrete or, as discussed here, masonry (see Figure 11.2.29).

The design characteristics are similar to masonry arch culverts. Refer to Chapter 4 and Chapter 15 for detailed information on culverts.

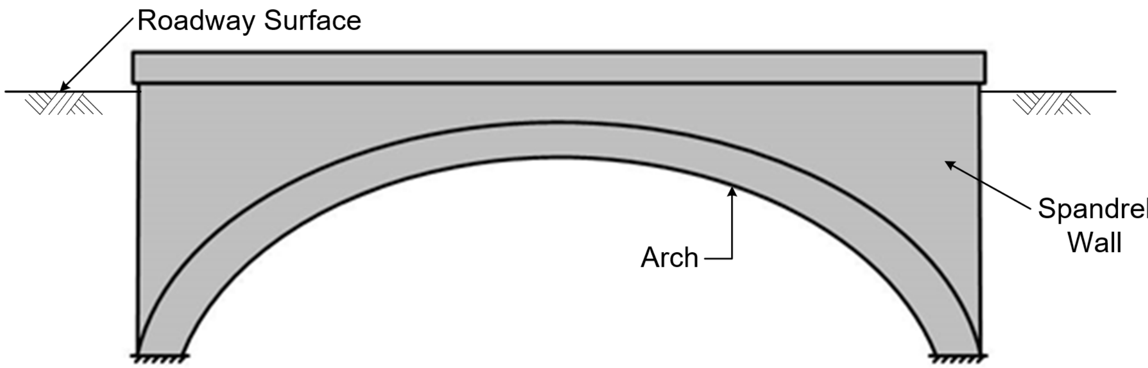


Figure 11.2.28 Elevation View – Closed Spandrel Deck Arch

Figure 11.2.29 Masonry Closed Spandrel Arch Bridge

11.2.5 FRP Superstructures

The majority of FRP used in bridge construction is for decks or for strengthening steel or concrete members. FRP shapes and composite sections are commercially available; however, new bridge construction with FRP primary members has been limited. FRP suspension and stay cables were also being considered due to a significant reduction in weight over their steel counterparts but are also rare. Several experimental bridges have been constructed using FRP superstructure members.

Section 11.3 Overview of Common Deficiencies

Refer to Chapter 7 for a detailed explanation of the properties of Timber, Masonry, and FRP, as well as the types and causes of deficiencies.

11.3.1 Timber Deficiencies

Common deficiencies that occur on timber bridges include:

- Inherent deficiencies - Checks, splits, shakes, and knots.
- Decay by fungi.
- Damage by insects and borers.
- Delamination.
- Loose connections.
- Surface depressions.
- Fire/Heat damage.
- Impact damage.
- Damage from wear, abrasion, and mechanical wear.
- Damage from overstress.
- Damage from weathering/warping.
- Failure of protective system.
- Chemical attack.

11.3.2 Masonry Deficiencies

Common potential deficiencies that may occur on masonry bridges include:

- Missing stones or mortar.
- Weathering.
- Spalling.
- Splitting.

11.3.3 FRP Deficiencies

Common potential deficiencies that may occur on FRP bridges include:

- Blistering.
- Voids and delaminations.
- Discoloration.
- Wrinkling.
- Fiber exposure.
- Scratches.
- Cracking.

Section 11.4 Inspection Methods

11.4.1 Visual and Physical Inspection

The inspection of timber for inherent deficiencies such as checks, splits, cracks, shakes, fungus decay, deflections, crushing, delaminations, and loose connections is primarily a visual activity.

The physical examination of a timber member can be conducted with a hammer or pick. The pick hammer is used to sound the members to detect hollow areas or internal decay. Picks can be used to determine the condition at or near the surface by probing into deterioration (see Figure 11.4.1).

Figure 11.4.1 Inspector using Pick Hammer to Inspect Timber Substructure

The location of any cracks in a timber member should be documented. The inspection team should measure the length and record the information for future monitoring.

Obvious surface deficiencies may be identified via a thorough visual assessment of stone masonry surfaces and mortar joints. The overall shape of a masonry bridge should be examined for misalignment that indicate overstress or differential settlement. A string line or straight edge can be used to determine arch misalignment. A plumb bob can be used to check vertical alignment of the spandrel wall. The inspection team should check for any separation or movement of stones.

Some stone masonry and mortar deficiencies may necessitate physical inspection methods in addition to visual inspection. Stone masonry and mortar should be physically examined using a measuring tape, crack comparator card, inspection hammer, rotary percussion tool, or straight edge. Suspect areas can be sounded using an inspection hammer. A delaminated area will most likely have a distinctive hollow “clacking” sound when tapped with a hammer or rotary percussion tool.

For FRP surfaces, a thorough visual assessment should be performed to identify obvious surface deficiencies. FRP members can be physically examined using a measuring tape, crack comparator card, inspection hammer, chain drag, feeler gage, taper gage, or rotary percussion tool. Any suspect or delaminated areas should be sounded using an inspection hammer or rotary percussion tool. The inspection team should document the length and width of cracks found during the physical inspection methods. A feeler or taper gage can be used to measure the spacing between any delaminated layers.

11.4.2 Advanced Inspection

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

There are several advanced methods for timber inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Stress wave velocity.
- Ultrasonic testing.
- Vibration.

Drilling and probing, considered destructive testing methods, are fairly common inspection methods, accompanying the physical methods listed previously, to detect soft or hollow areas caused by decay and insect damage. Another destructive testing method that may be utilized for timber is a moisture meter.

There are several advanced methods for masonry inspection that may be performed in the field. The methods that can be completed on site by a bridge inspector or certified NDE technician include the following.

- Flat Jack testing.
- Impact echo testing.
- Infrared thermography.
- Rebound and penetration methods.
- Ultrasonic Pulse Velocity (UPV).
- Ultrasonic Pulse Echo (UPE).
- Ultrasonic Surface Waves (USW).

GPR has also been used to determine the size/extent/depth of the substructure or arch when plans are not available.

There are several advanced methods for FRP inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Acoustic emission testing.
- Ultrasonic testing.
- Laser-based ultrasound testing.
- Tap testing.
- Thermal testing.

Advanced methods are described in detail in Chapter 17.

Section 11.5 Inspection Areas and Deficiencies

For Routine inspections, all areas of the bridge and all locations of each member are to be observed, at times needing specialized access equipment. Before inspecting specific areas below the bridge, the superstructure should be inspected for general global alignment. The inspection team should look along the curb line, top of railing or the bottom of the superstructure and look for vertical or horizontal misalignment. The inspection team should also look at both sides of expansion joints to check if they line up properly. Vertical misalignment, horizontal misalignment or sagging may be an indication of specific issues noted in this section, such as crushing of timber members or missing stones in masonry structures. Visual, physical, or advanced inspection methods may be beneficial in evaluating the following inspection areas and deficiencies. This section describes common problems related to specific areas and details to attentively inspect.

11.5.1 Timber

Bearing Areas

The inspection team should check the bearing areas for plumbness and crushing of the beams near the bearing seat (see Figure 11.5.1). Decay and insect damage should be investigated by visual inspection and sounding and/or probing at the ends of the beams where dirt, debris, and moisture tend to accumulate.

Figure 11.5.1 Bearing Area of Typical Solid Sawn Beam

Shear Zones

Maximum shear occurs near supports. Horizontal shear within the beam accompanies the vertical shear. Because of timber's orthotropic cell structure, it has excellent resistance against vertical shear (perpendicular to the grain) but low resistance against horizontal shear (along the grain). Failure due to overload often begins with a short horizontal shear split. The member capacity is progressively diminished as the split propagates through the member. A horizontal shear crack is a split along the length of the beam that extends from one vertical face of the member to the other.

Investigate the area near the supports for the presence of horizontal shear cracking. The presence of horizontal cracks on the sides of the girders indicates the onset of shear failure. These cracks can propagate quickly toward mid-span and can significantly decrease the capacity of the beam (see Figure 11.5.2). This is a prevalent problem with timber beams.

Figure 11.5.2 Horizontal Shear Crack in a Timber Beam

Glulam members should be examined for horizontal shear cracks and delaminations near the ends of the beam. Delaminations (i.e., separation of the laminations) can occur due to failure of the glue or failure at the bond between the glue and the lamination (see Figure 11.5.3 and Figure 11.5.4). Delaminations that are found near the center of the cross section are more serious than those near the top or bottom of the beam, but any delamination observed should be noted. Delaminations directly through a connector are also undesirable. Be aware that delaminations and horizontal shear cracking can look similar.

Figure 11.5.3 Glulam Floorbeam Showing with Delamination

Figure 11.5.4 End of a Laminated Timber Beam Showing Delaminations

Flexure Zones

The zones of maximum bending should be examined for signs of structural distress. Section loss due to decay or fire, especially near mid-span, should be investigated. Beams should be inspected for excessive deflection or sagging.

Solid sawn beams with sloping grain that intersects the surface in the maximum flexure zone are particularly susceptible to cracking because the tensile stress and horizontal shear stress combine to split the grain apart. Tension cracks in timber break the cell structure perpendicular to the grain and may be preceded by the appearance of horizontal shear cracks.

Axial Tension Members

Timber members may experience axial tension when utilized in a truss bridge. Failure occurs parallel to the direction of the grain when the axial tensile capacity of a timber member is exceeded.

Axial Compression Members

Truss or arch bridges also contain members that are exposed to axial compression. The inspection team should investigate these members for signs of cracking of the timber due to buckling. There can also be signs of crushing at the ends or connection points of the member. Deformations of the member or portion of the member can lead to eccentricities in other areas of the truss due to load redistribution.

Areas Exposed to Drainage

Timber beams, 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 all cases, regardless of deck type, any joint areas should be checked for damage due to drainage. The curb line areas should also be examined. Standing water soaks into the beams and can corrode any stressing rods, if present.

The inspection team should investigate for signs of decay along the full length of the beam but especially where the beam is subjected to continual wetness or prolonged exposure to moisture (see Figure 11.5.5). 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.

Figure 11.5.5 Areas of Decay in a Timber Beam

Decay and chemical attack may be evidenced by discolored wood, brown and white rot (see Figure 11.5.6), 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. Any drill hole areas should be examined carefully for proper preservation treatment and dowel plug installations.

Figure 11.5.6 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; termites can often be readily seen. Another indication of insect infestation can be hollow sounding wood.

Areas Exposed to Traffic

For overhead and through structures, the inspection team should check for collision damage from vehicles passing below or adjacent to structural members.

Areas Previously Repaired

Any repairs that have been previously made should be thoroughly examined. The inspection team should determine if repaired areas are sound and functioning properly.

Secondary Members

Solid sawn or glulam diaphragms should be inspected for decay, fire damage, and insect damage (see Figure 11.5.7). Connections of bracing to beams should be examined for tightness, cracked or split members, and corroded, loose, or missing fasteners (see Figure 11.5.8). Deteriorated secondary members may indicate potential problems in the primary members.

Figure 11.5.7 Typical Timber End Diaphragm

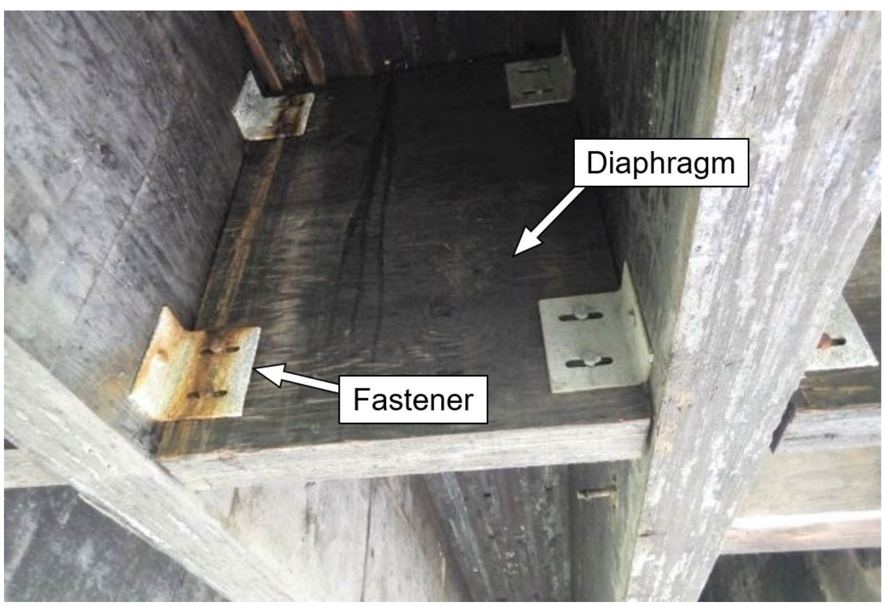


Figure 11.5.8 Timber Diaphragm with Corroded Fasteners

Steel cross bracing should be examined for corrosion, bowing, or buckling (see Figure 11.5.10).

Stressing Rods

The condition of the steel stressing rods should be examined and inspected for crushed areas and splits in the fascia members. Note any broken or missing rods (see Figure 11.5.9). Loss of prestress in the rods, indicated by shifted planks in the stress-laminated timber and excessive deflection or loose rods, should be checked. This may be observed when the bridge is subject to a moving live load.

Figure 11.5.9 Broken Stressing Rods

Fasteners and Connectors

The fasteners (e.g., nails, screws, bolts, and deck clips) should be checked for corrosion. The inspection team should also look for loose or missing fasteners. Moisture and decay around the holes should be checked (see Figure 11.5.10). For stressing rod hardware, the inspection team should check the condition of the plates and nuts on the threaded rods.

Figure 11.5.10 Steel Diaphragms Bolted to Glulam Beams

Miscellaneous Areas

The inspection team should check any notched members for signs of cracking or splitting. Stress concentrations are created at square notches and small deficiencies can propagate into larger issues.

11.5.2 Masonry

This section provides information regarding masonry superstructures. Refer to Chapter 15 for more information on specific inspection areas and deficiencies for masonry culverts.

Bearing Areas

For arches, the arch/footing interface has the greatest bearing load magnitude. Loss of stones or cracked or missing mortar should be inspected. The arch should be examined for separation of or missing masonry stones, which may indicate an overstress condition.

The arch/spandrel wall interface also carries a large bearing load. Missing, shifted, or crushed stones at the bottom of the wall or in the arch rib should be examined. These may indicate bending in the wall, which is caused by overloads and differential arch rib deflection.

The arch ring may be examined for missing or moving stones. The inspection team should look for efflorescence, missing mortar, or wear on the stones themselves (see Figure 11.5.11).

Figure 11.5.11 Masonry Stone Arch with Deterioration

Shear Zones

For arches, high shear zones are close to the bearing areas at the arch/footing interface or between the individual stones in the arch (see Figure 11.5.11). The deficiencies in these areas are similar to those in bearing areas. The inspection team should examine these areas for signs of cracked or missing stones and mortar.

Flexure Zones

The spandrel wall may be forced outward due to transverse forces caused by dead load of the fill material and live load surcharge. Note the position of the wall so that future movement can be tracked in future inspections. Any indication of misalignment in the normal curvature of the arch could indicate a serious problem related to spandrel wall movement.

Movement of the wall can cause cracks in the mortar and stones to become loose over time (see Figure 11.5.12).

Compression Zones

The entire arch portion of the masonry structure is primarily in compression. Any visible distortion or bulging indicates that there is a change in the geometry of the arch shape over a substantial area of the structure. This could indicate a serious condition and a possible formation of a hinge which could lead to a hinge collapse mechanism.

The condition should be documented with measurements and descriptions of the extent, location, and magnitude of bulge (see Figure 11.5.13).

Figure 11.5.12 Masonry Arch Spandrel Wall with Widespread Mortar Deterioration

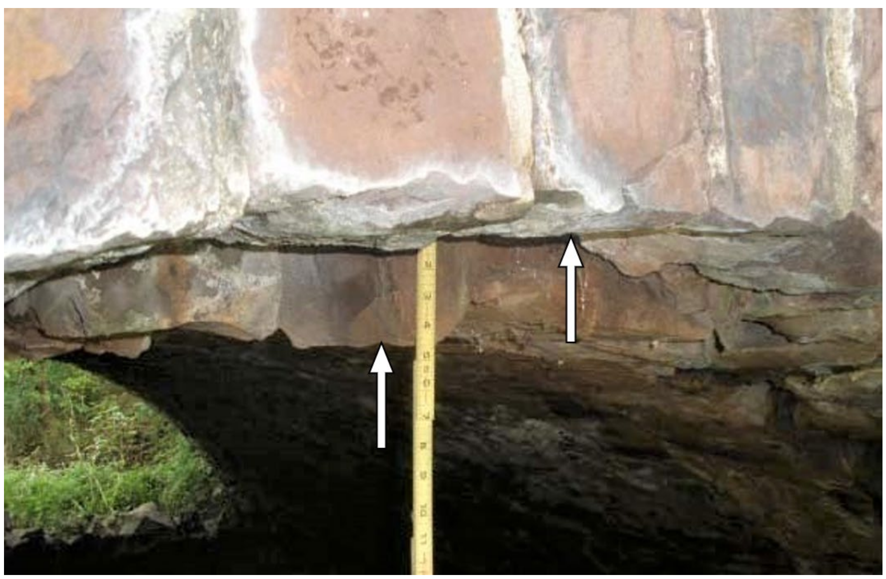


Figure 11.5.13 Downward Deflection of Masonry Arch

Areas Exposed to Drainage

Water seepage through the structure may be caused by poor drainage. Since closed spandrel arches have fill material between the roadway and the arch, both walls and the arch are subjected to infiltration from drainage water. Occasionally, utility lines may be buried in the fill between the spandrel walls. Leaking utilities can saturate the arch fill as well.

This can ultimately result in stone and mortar deterioration as well as loosening of the stones when combined with freeze-thaw conditions.

Areas Exposed to Traffic

Any locations exposed to traffic should be inspected for signs of collision damage. The top of the spandrel wall may be along the roadway, even if it is not directly adjacent to the shoulder. The inspection team should check for cracking in the mortar or missing, crushed, or cracked stones (see Figure 11.5.14).

The arch portion of the structure may also be exposed to collision damage if the bridge is over a roadway. Any damage to the underside of the structure should be documented.

Figure 11.5.14 Top of Spandrel Wall Exposed to Traffic

Areas Previously Repaired

Any repairs that have been previously made should be thoroughly examined. The inspection team should determine if repaired areas are stable and functioning properly. In particular, previously repointed masonry should be checked to ensure that the mortar is sealing the joint between stones and is not deteriorating.

Joints

Minor mortar loss is relatively shallow which does not indicate the onset of any stones becoming loose. Complete mortar loss can result in loose stones or the loss of stones altogether. Mortar loss can also lead to problems caused by drainage. If not addressed, this issue can lead to loss of fill material or a break in the load path.

Miscellaneous Areas

An opening in the spandrel wall can cause partial or complete collapse of the wall due to loss of the fill material supporting the roadway.

11.5.3 FRP

Special observation 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 - Look for and measure 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.

Refer to Chapter 7 for more information on inspection areas and deficiencies for FRP bridges.

Section 11.6 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of timber, masonry, and FRP bridges. The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 12 TABLE OF CONTENTS

CHAPTER 12 LIST OF FIGURES

This page intentionally left blank.

Chapter 12 Inspection and Evaluation of Bridges with Complex Features

Section 12.1 Introduction

This chapter describes the inspection and evaluation of bridges with specific members and systems that are complex features, and not covered in Chapters 7 through 11. Refer to Chapter 5 for characteristics of bridges with complex features.

Per the National Bridge Inspection Standards (NBIS), a complex feature is defined as bridge component(s) or member(s) with advanced or unique structural members or operational characteristics, construction methods, and/or requiring specific inspection procedures. This includes mechanical and electrical elements of moveable spans and cable- related members of suspension and cable-stayed superstructures (23 CFR 650.305).

The following sections cover common types of bridges that often have complex features, including information on the operation, maintenance and inspection of the structure and the movement mechanisms. The inspection of these specialized features demands a diverse team collectively capable of inspecting the structure as well as the mechanical, electrical, and pneumatic or other movement mechanisms. The duties of the bridge team 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 of maintenance and operation personnel. Regardless of additional specialized inspection personnel, an NBIS team leader should be on site for all types of inspection, including structural, mechanical, and electrical. The team leader should coordinate with the owner to determine everyone's role in the inspection of these highly specialized structures. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

Section 12.2 Cable-Supported Bridges

12.2.1 Overview of Common Deficiencies

Commonly-identified deficiencies in steel cable members include:

- Breaches in corrosion protection systems.
- Loosening of cable bands at transitions.
- Breakdowns in dehumidification systems.
- Malfunctions in health monitoring systems.
- Plugged drains.
- Grout voids.
- Damage to dampers or cross-ties.
- Failure of hose clamps or keeper rings.
- Fatigue in wires or cable casing.
- Fretting in wires over saddles.

Refer to Chapter 7 for a more detailed presentation of the properties of steel, types and causes of steel deficiencies, and the examination of steel.

12.2.2 Inspection Methods

The inspection methods presented in this chapter are not exhaustive but are unique to cable-supported bridge members. Therefore, include both the procedures presented in this chapter as well as the general procedures described in Chapter 7 during the inspection of bridges with complex features.

Visual and Physical Inspection

These bridges are considered to have complex features according to the NBIS regulation (23 CFR 650.305). Additional inspector training and experience is likely necessary to inspect these bridges with complex features. Inspection of these features is called a complex feature inspection and any specific procedures for unique and complex structural features must be developed for each bridge and contained in the bridge file (23 CFR 650.313(g)).

It is possible to inspect the interior wires of a parallel wire suspension cable by using wooden or plastic wedges to separate portions of the cable (see Figure 12.2.1). This process is labor-intensive and is limited by the size of the cable, the size of the wedges, and access. It does give valuable information, however, that can be extrapolated to other areas of the cable that exhibit similar exterior conditions.

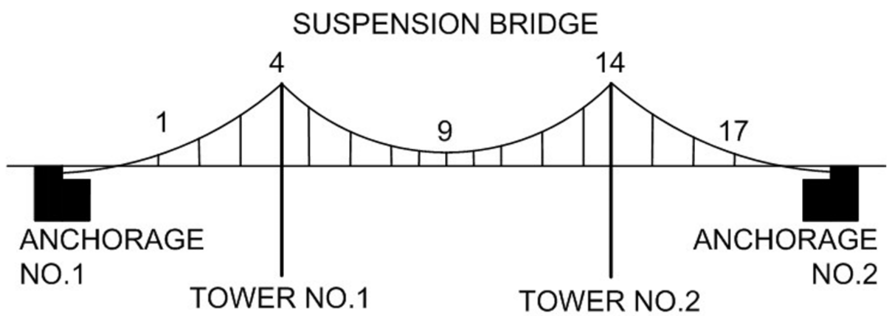
Figure 12.2.1 Inspection of Cable with Wedges

Reference the *National Cooperative Highway Research Program (NCHRP) Synthesis 353 "Inspection and Maintenance of Bridge Cable Systems", 2005* for a detailed description of inspection areas and procedures for cable-stayed bridge cable element systems.

Reference *NCHRP Synthesis 534 "Guidelines for Inspection and Strength Evaluation of Suspension Bridge Parallel Wire Cables", 2005* for detailed guidance for inspecting and determining remaining service life for suspension bridge main cables.

Due to the specialized nature of these bridges, and because no two cable-supported bridges are identical, the inspection is led by someone very familiar with the bridge. Many major bridges, such as cable-supported bridges, should 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 inspectors to report the findings in a rigorous and systematic manner. Refer to Chapter 3 and Chapter 18 for further detail on inspection reporting.

A set of customized, preprinted forms should be prepared for documenting deficiencies encountered in the cable system of a suspension bridge. Sample forms are presented in Figure 12.2.2 and Figure 12.2.3. Separate forms should be used for each main suspension cable or plane of cables. Designations used to identify the suspender ropes or cables and the panels provide a methodology for locating the problems in the structure. Inspectors should note and describe vibrations, whether local or global, when performing inspections of cable-supported structures.



CONDITION OF EAST CATINARY CABLES

1.	7.	13.
2.	8.	14. N/A
3.	9.	15.
4. N/A	10.	16.
5.	11.	17.
6.	12.	

Figure 12.2.2 Sample Form for Recording Deficiencies in Suspension Bridge Cables

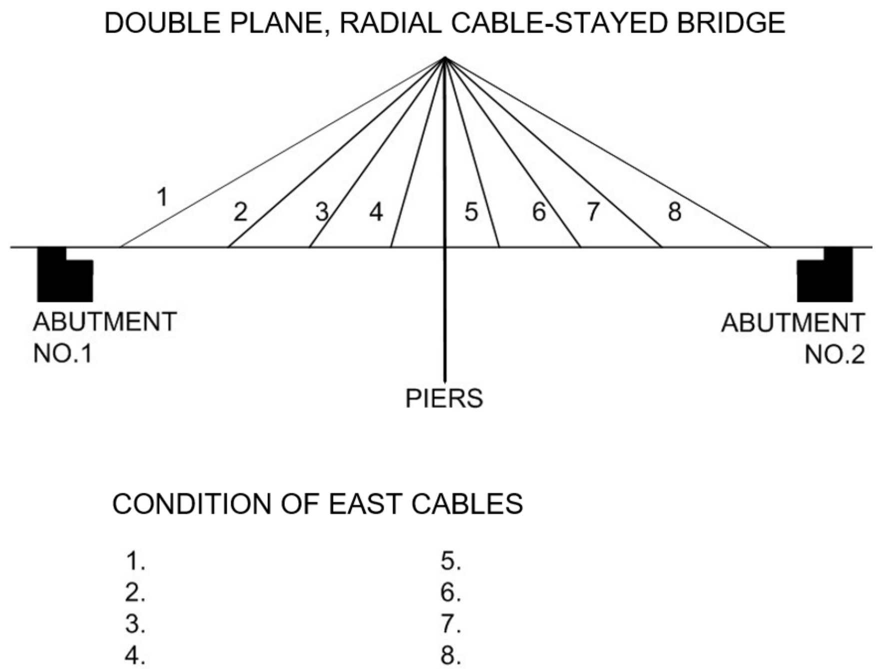


Figure 12.2.3 Sample Form for Recording Deficiencies in Cable-Stayed Bridge Cables

Advanced Inspection

In bridge cables, whether a suspension bridge or cable-stayed bridge, the greatest problems generally occur due to corrosion and SI of individual wires. Visual inspection of unwrapped cables is limited to the outer wires. Visual inspection of wrapped cables is limited to the protective sheathing. Advanced inspection methods are typically used to achieve a more rigorous and thorough inspection of steel cable-supported bridges, including:

- Acoustic emissions monitoring.
- Corrosion sensors.
- Smart coatings.
- Dye penetrant testing.
- Magnetic particle testing.
- Radiography testing.
- Computed tomography.
- Robotic access and testing.
- Ultrasonic testing.
- Eddy current.
- Electrical Fatigue Sensor (EFS).
- Magnetic flux leakage.
- Laser vibrometer.
- Impulse radar.
- Infrared thermography.

Advanced methods are described in detail in Chapter 17.

Review of Bridge File

The owner of cable-supported bridges generally keeps a complete file available for the engineer who is responsible for the inspection and maintenance of the bridge. Inspectors should review the bridge file with preventative maintenance measures in mind. Refer to Chapter 2 for information on planning the inspection. Refer to Chapter 3 and Chapter 18 for general record keeping and documentation. The file commonly includes (if applicable), but is not limited to, the following:

- Inspection procedures for cables.
- Complete set of design plans and special provisions.
- As-built plans for the bridge.
- Copies of inspection reports.
- Copies of maintenance reports.
- Copies of repair plans.
- Previous NDT results or load test documentation for any cables.
- Health and safety plan.
- Personnel safety plan.
- Public safety plan.
- Bridge-specific safety plan.

12.2.3 Inspection Areas and Deficiencies

During the inspection of any type of cable-supported structure, any deficiencies that are detrimental to the bridge members should be noted. Most of the bridge structure material deficiencies that are listed in Chapter 7 as potential problems also apply to cable-supported bridges. Inspection areas are similar to those listed in Chapters 8-10, 13, and 14 for other non-cable portions of the bridge.

Cables

Cable members of suspension and cable-stayed superstructures are considered complex features, and should be inspected according to specific inspection procedures developed by the owner. Inspectors should check for cable alignment irregularities including waviness or excessive sag. Cable sag can be estimated or measured using optical devices or through video or photo image processing. Cable angle can be measured with an inclinometer at specific points.

Cables should be inspected for indications of corroded wires. Inspectors should examine 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. Inspectors should also check for cracking or damage to guide pipes and vandal protection pipes or evidence of the impact of cable components on guide pipes.

Common wrapping methods for corrosion protection of finished cables include spirally wound soft steel galvanized wire, neoprene, or plastic wrap type tape (see Figure 12.2.4). The wrappings should be checked for corrosion and cracking of soft galvanized wire, staining and dark spots indicating possible corrosion of the cables, and loose wrapping wires or tape. White oxidation is indicative of the zinc layer oxidizing and breaking down instead of the steel wire. This may continue until the zinc coating is locally exhausted and then corrosion of the underlying steel will most likely begin. Bulging or deforming of wrapping material may indicate possible corrosion or broken wires (see Figure 12.2.5). Inspectors should check for evidence of water seepage at the cable bands, saddles, and castings (see Figure 12.2.6).

Figure 12.2.4 Cable-Wrapping Placement

Figure 12.2.5 Deformed Cable Wrapping – Possible Broken or Corroded Wires

Figure 12.2.6 Deteriorated Cable Wrapping Allowing Water Penetration into Sheath

Cable sheathing assemblies are also closely monitored. The most common types of assemblies are steel sheathing and polyethylene sheathing. These may be pre-assembled in sections and may be coated.

If steel sheathing is used, the system should be inspected for corrosion (see Figure 12.2.7), condition of protective coatings, and any welds that may be present. Cracking may be caused by water infiltration and corrosive action. Cracking in steel sheathing may be caused by fatigue (see Figure 12.2.8).

Figure 12.2.7 Corrosion of Steel Sheathing

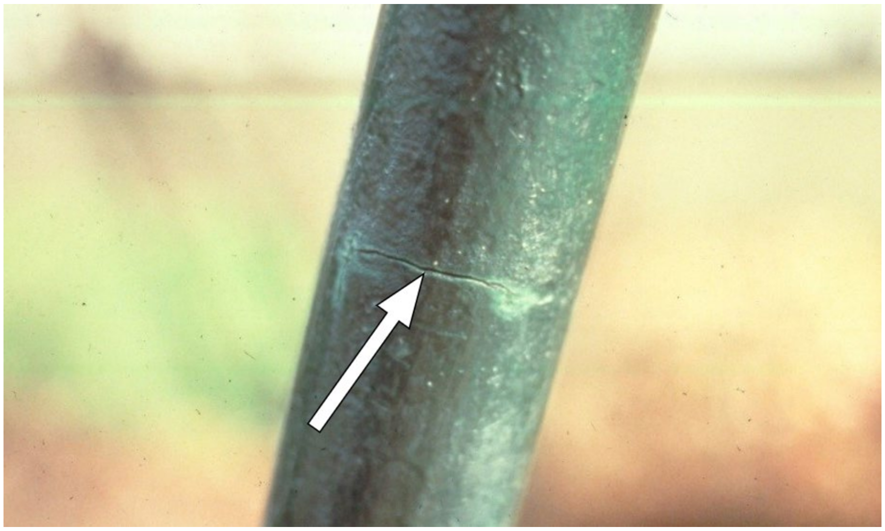


Figure 12.2.8 Cracking of Cable Sheathing

If polyethylene sheathing is used, the system should be inspected for nicks, cuts, and abrasions. Inspectors should check for cracks and separations in caulking and in fusion welds. Bulging may indicate corrosion or broken wires (see Figure 12.2.9). Previous inspection reports should be reviewed for any history of bulging, as grouting during construction can also cause this issue. Splitting is sometimes caused by temperature fluctuations (see Figure 12.2.10). The coefficient of the thermal expansion for polyethylene materials is approximately three times greater than that of steel or concrete.

Figure 12.2.9 Bulging of Cable Sheathing

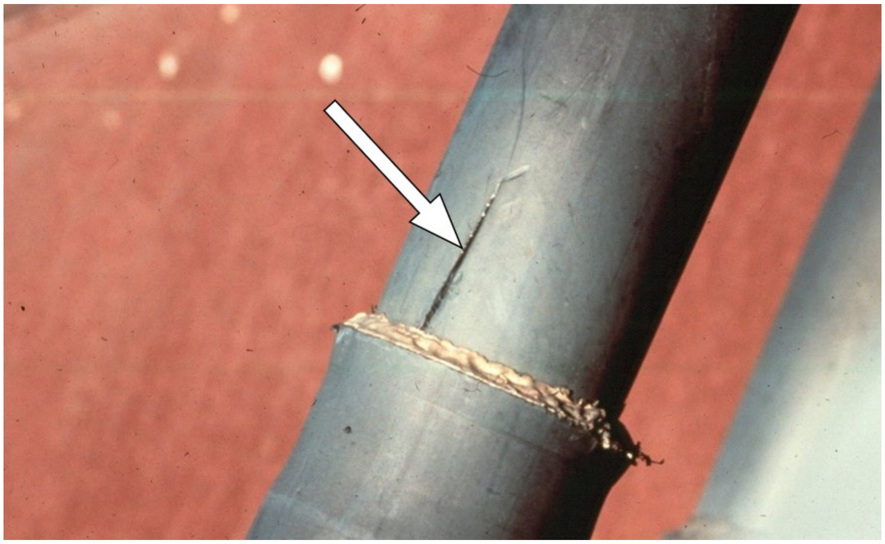


Figure 12.2.10 Splitting of Cable Sheathing

Anchorage and Connections

The anchorage system at the ends of the main suspension cable may consist of the following parts that need inspection:

- Splay saddles.
- Bridge wires.
- Strand shoes or sockets.
- Anchor bars.
- Chain gallery/anchor vault.

Inspectors should check the splay saddles for missing or loose bolts and the presence of cracks in the casting itself. There is a possibility of movement of the cable away from the splay. Signs of this movement may be the appearance of unpainted strands on the bottom or "bunched up" wrapping on the top.

In parallel wire type suspension bridges, the unwrapped wires between the strand shoes and the splay saddle should be inspected. Inspectors should carefully insert a large screwdriver between the wires and apply leverage. This should help reveal broken wires. The wires should be examined for abrasion damage, corrosion, and movement.

At the anchorages of parallel wire type suspension bridges, inspectors should check 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, the strand sockets should be examined for signs of movement, slackness or sag, corrosion, and broken sockets. Unpainted or rusty threads at the face of the sockets may indicate possible "backing off" of the nuts.

The anchor bars or rods should be inspected for corrosion (section loss), deficiencies, or spalls and cracks at the face of their concrete embedment. Corrosion or other signs of distress should be checked over the entire visible portion.

The chain gallery (anchor vault), or interior of the anchorage block, should be inspected for corrosion and deficiencies of any steel hardware, and cracks and spalls in the concrete anchor (see Figure 12.2.11). Deficiencies within the anchor vault itself should be noted, including spalls, cracks, and infiltrating water. Inspectors should note the proper function of a sump pump and sump pit, if present. Inspectors should look for signs of inadequate ventilation or vermin infestation or nesting blocking the ventilation openings.

Figure 12.2.11 Chain Gallery (Anchor Vault) Interior

The transition area between the steel anchor pipe and cable should be inspected for water tightness of neoprene boots at the upper ends of the steel guide pipes (see Figure 12.2.12). Drainage between the guide pipe and transition pipe, and deteriorations, such as splits and tears, in the neoprene boots should be checked (see Figure 12.2.13). Inspectors should check for sufficient clearance between the anchor pipe and cable, noting rub marks, and kinks. Any keeper rings or neoprene rings should be examined for damage or dislocation. Any gaps between the neoprene rings and the sheathing should be identified.

Figure 12.2.12 Neoprene Boot at Steel Anchor Pipe Near Anchor

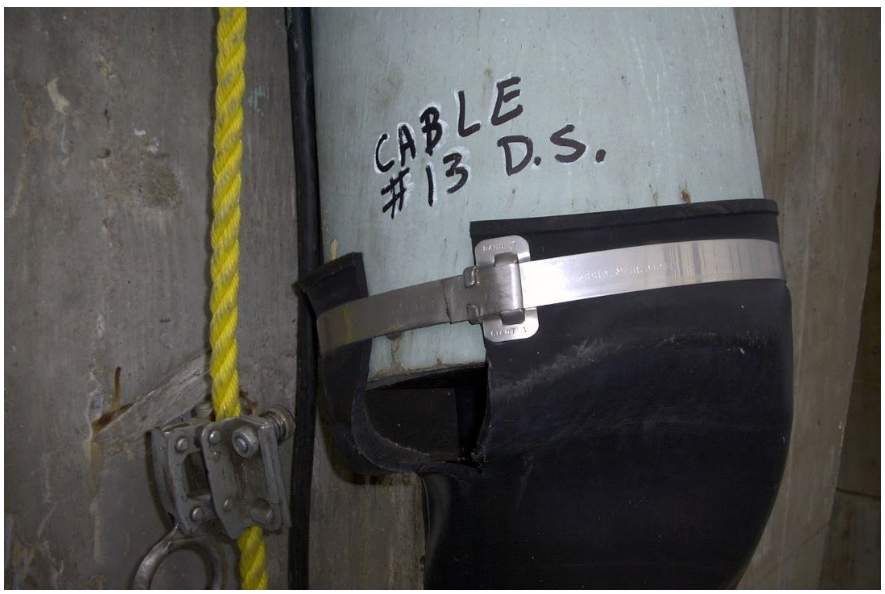


Figure 12.2.13 Split Neoprene Boot

Inspectors should examine the cable anchorages for corrosion of the anchor system. Cracks and nut rotation at the socket and bearing plate should be inspected. Evidence of moisture and seepage of grease or other fillers from the anchorage should be checked. If there is an access port at the end cap, it can be opened and examined for moisture or moisture-contaminated grease. In some cases, removal of the end caps on the sockets allows for visual inspection of the anchorage plate and anchorage devices. The presence of moisture can also be investigated.

Dampers

A variety of damper types may have been installed on cable-supported bridges, including:

- Shock absorber systems.
- Tie-type dampers.
- Tuned mass dampers.

If shock absorber-type dampers are utilized, the system should be inspected for corrosion, oil leakage in the shock absorbers, and deformations in the bushings (see Figure 12.2.14 and Figure 12.2.15). Inspectors should check for tightness in the connection to the cable pipe and ensure bolts are tight. Inspectors should also check the shock absorber connections and support rails if present.

Figure 12.2.14 Shock Absorber Damper System

Figure 12.2.15 Small Shock Absorber Damper

Tie-type dampers (see Figure 12.2.16) should be inspected for corrosion and deformations in the bushings. Inspectors should check for tightness in the connection to the cable pipe and ensure bolts are tight. The cross-tie cables should be inspected for sagging, as they may need to be retensioned. Any damage or cracking on any portion of the cross-tie cables should be noted. Inspectors should check for evidence of fretting and fatigue, especially at the connections.

Figure 12.2.16 Cable Tie Type Damper System

Tuned mass dampers (see Figure 12.2.17) should be inspected for corrosion and deformations in the bushings. Tightness in the connection to the cable pipe bolts should be checked.

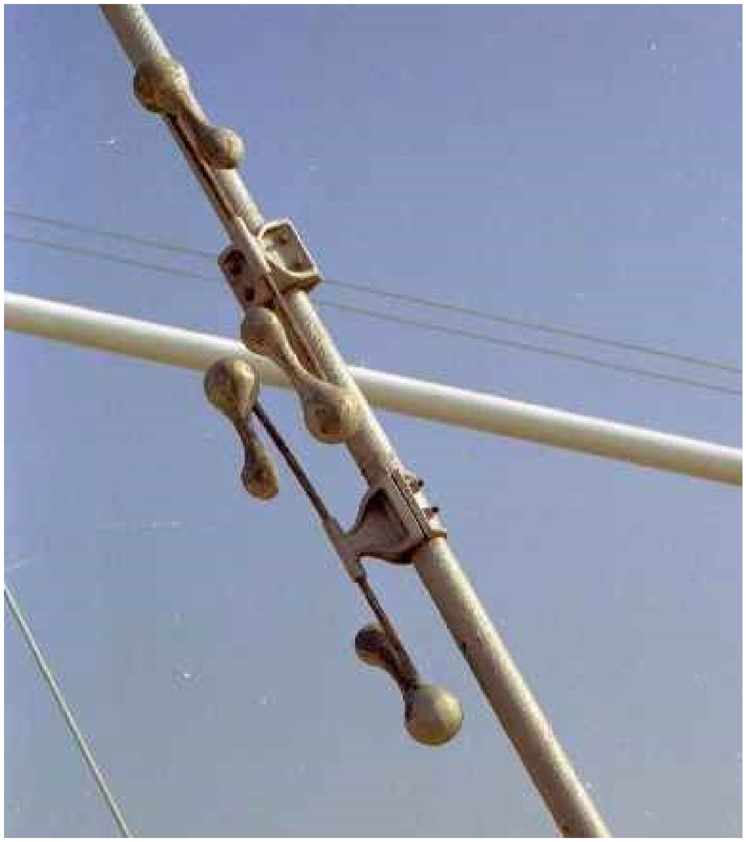


Figure 12.2.17 Tuned Mass Damper System

Miscellaneous

The hand ropes and connections along the main cables should be inspected for loose connections of stanchions (hand rope supports) to cable bands or loose connections at anchorages or towers. Inspectors should also check for corroded or deteriorated ropes or stanchions, bent or twisted stanchions, and too much slack in the rope. Deficiencies of other inspection access mechanisms should be noted, including access doors, hatches, ladders, walkways, and their connections for signs of damage or vandalism.

The saddles should be inspected for missing or loose bolts, and corrosion or cracks in the casting. Inspectors should check for proper connection to top of tower or supporting member and possible slippage of the main cable.

The suspender cables should be inspected for corrosion or deficiencies, broken wires, and kinks or slack. Abrasion or wear at sockets, saddles, clamps, and spreaders should be checked. Excessive vibrations should be noted.

The suspender rope sockets should be inspected for corrosion, cracks, or other deficiencies. Inspectors should check for abrasion at connection to bridge superstructure. Any unanticipated movement should be noted.

The cable bands should be inspected 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. Inspectors should check for the presence of cracks in the band itself, corrosion or deficiencies of the band, and loose wrapping wires at the band.

Anchor pipe clearances, flange joints, and polyethylene expansion joints should be included within the inspection of the cable system. The load cell forces in the cables should be recorded, if available. Excessive vibrations should also be recorded including amplitude and type of vibration along with wind speed and direction, or other forces including vibrations such as traffic. Cable and tower lighting systems should also be evaluated.

Inspectors should check weather data collection system that may be mounted on the bridge and check that it is operational. Any structural health monitoring systems should be inspected for soundness and operational status, including potential issues with the instruments, wiring, communications, or power. Soundness of any installed survey markers should be checked.

12.2.4 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of cable-supported bridges. The two major rating systems currently in use are:

- *FHWA's Specification for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

Section 12.3 Movable Bridges

This section discusses mechanical and electrical systems of movable bridges, which need inspection and testing by qualified individuals, and subsequent documentations of the qualifications of the inspection team per the NBIS (23 CFR 650.313). Thorough inspection of such systems is beyond the scope of a Routine inspection. Inspection procedures for all aspects are to be developed to include any additional qualifications of specialized inspectors and testing personnel necessary for a complex feature inspection. Routine inspection recommendations will be presented in subsequent sections. However, inspectors should be aware of these basic operations of movable bridges to be able to identify any obvious defects and malfunctions of these systems during Routine inspections, as well as complex feature inspections.

12.3.1 Overview of Common Deficiencies

Common deficiencies that can occur to steel members of movable bridges include:

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

Refer to Chapter 7 for a detailed explanation of the properties of steel, types and causes of steel deficiencies, and an overview of fatigue and fracture in steel bridges.

Common deficiencies that occur to concrete members of movable bridges include:

- Cracking (structural).
- Cracking (non-structural).
- Scaling.
- Delamination.
- Spalling.
- Chloride contamination.
- Freeze-thaw damage.
- Efflorescence.
- Alkali silica reactivity (ASR).
- Ettringite formation.
- Honeycombs.
- Pop-outs.
- Wear.
- Collision damage.
- Abrasion.
- Overload damage.
- Reinforcing steel corrosion.
- Loss of prestress.
- Carbonation.

Refer to Chapter 7 for a detailed explanation of the properties of concrete and types and causes of concrete deficiencies.

12.3.2 Inspection Methods

Visual and Physical

The general condition of the machinery and its performance during operation should be observed. Inspectors should check for smoothness of operation and note any abnormal performance. Any noise or vibration should be noted, and inspectors should determine the source. Any unsafe or detrimental methods followed by the operator should be documented to prevent injury to the public, operating personnel, or operating equipment. Also, the condition of the paint system should be noted.

Movable bridges are considered to have complex features according to the NBIS, which requires identification of specialized inspection methods, and additional inspector training and experience necessary to inspect these bridges with complex features (23 CFR 650.313(g)). The bridges are then to be inspected according to these methods.

Safety

Normal safety considerations are accounted for when inspecting movable bridges. Refer to Chapter 2 for safety fundamentals for bridge inspection. However, it is imperative that a movable bridge inspection team coordinates their work with the bridge operator and emphasize the need for advanced warning of a bridge opening. The bridge operator cannot operate the bridge until being notified by the inspectors that they are ready for an opening. Established lockout-tagout procedures should be used on the control panel and contact should be maintained with the inspectors via radio or cell phone.

Public safety considerations include good visibility of roadway and sidewalk for the bridge operator (see Figure 12.3.1), 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 deactivate or indicate “green” unless the gates and barriers are fully retracted. It should be ensured that inspectors and any passers-by are not within the proximity of the bridge and its mechanical parts during opening or closing operations, as there could be both electrical and crushing hazards associated with the process.

The location of the bridge opening should be observed in relation to the gates (see Figure 12.3.2), traffic signals and bells, and it should be determined whether approaching motorists can easily see them. Their operation and physical condition should be checked to determine if they are functioning and well maintained. Inspectors should 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 provided on the gate arms. High-speed roadways and curved approaches to a movable bridge preferably have advanced warning lights, signals, and/or signage.

Inspectors should use caution when performing any activity related to climbing or specialized access. There are additional hazards associated with movable bridges that may not exist for a typical structure. There may be stairs or access equipment in dimly lit areas. It should be ensured that any necessary access equipment is clear of moving sections of the bridge.

Figure 12.3.1 Operator's House with Clear View of Traffic Signals and Lane Gates

Figure 12.3.2 Movable Bridge Traffic Control Gate

Navigational safety considerations include compliance with minimum channel width and vertical clearance when the span is open for navigation. Minimum underclearances designated on the permit drawing are to be provided. Underclearance gauges for closed bridges should be inspected for accuracy, visibility, and legibility.

It should be noticed that navigation lights have a relay for backup light, and red span lights do not change to green until bridge is in fully open position. Navigation lights should be checked for broken lenses, deteriorated insulation of wiring and cable, and dry and clean interior (see Figure 12.3.3).

Inspectors should check that the marine radio communication equipment is functional (see Figure 12.3.3). It should be verified that the operator can automatically sound the emergency signal to navigation vessels if bridge cannot be opened.

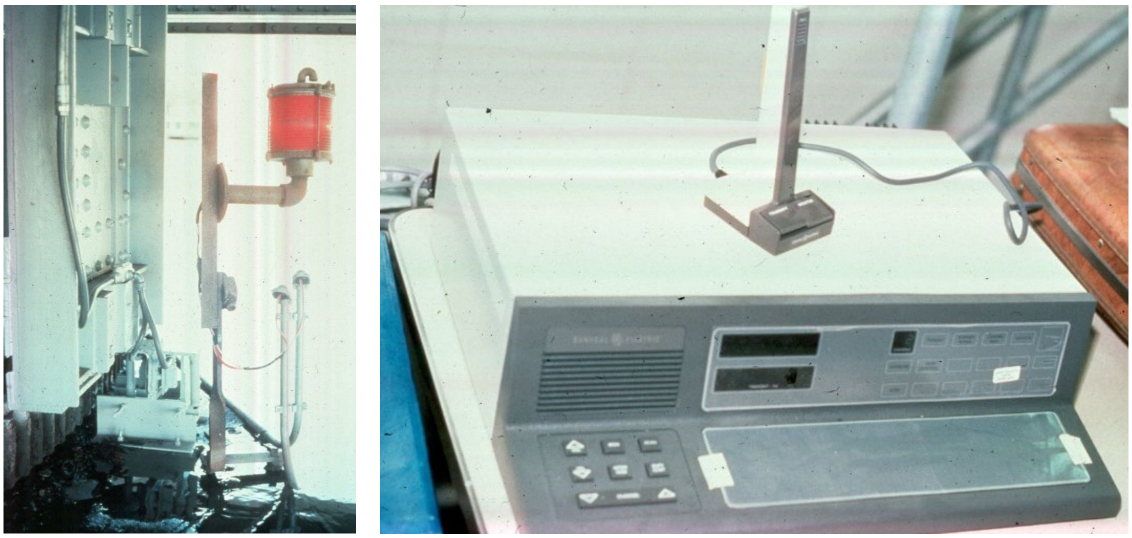


Figure 12.3.3 Navigational Light and Marine Two-Way Radio Console

Structure safety considerations include the structural ability to carry the anticipated loads. Pressure relief valves on hydraulic power units are generally used to limit hydraulic forces applied to machinery and structure. The power applied to machinery and the structure are to be kept within design limits by limiting speed.

Opening and Closing Sequences

When a movable bridge is fully open, it is supposed to provide the necessary navigational channel clearances provided on the Permit Drawing(s). It should be verified that the bridge conforms to the Permit Drawing(s). If the bridge cannot be opened to provide the necessary clearances, the U.S. Coast Guard should be notified immediately, and action should be taken to restore the clearances.

Trial openings should be conducted as necessary to ensure proper operation and that the movable span is properly balanced. Trial openings are specifically for inspection. The movable bridge should be operated in both normal and emergency modes to check all interrelated interlocks and to verify all parts are operating correctly.

During normal operation, inspectors should verify that each interlock functions properly and can be bypassed (when necessary). The controls should be verified 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 accordance with the specific operations of the bridge.

An example bridge opening sequence for a bascule bridge is as follows:

1. Activate traffic signals.
2. Lower oncoming gates and, when traffic has cleared, lower off-going gates. – 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 – 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.

An example bridge closing sequence for a bascule bridge is as follows:

1. Press “lower” button if automatic operation is provided or, if manual operation is provided, proceed as follows:
   a. Release emergency brakes.
   b. Accelerate leaves to full speed.
   c. For all types of bridges with lock bars:
      1) At advanced nearly closed position, decelerate leaves to slow speed. Leaves 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 should clear the diaphragm) by action of the bridge limit switch.
      3) Depress foot switch to 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 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".

Advanced Inspection

In addition to the advanced inspection methods for typical steel or concrete bridges, movable bridges have mechanical, hydraulic, and electrical items that are considered complex features and should be inspected thoroughly. These are described in detail in Section 12.3.3.

Advanced inspection methods for steel and concrete are described in detail in Chapter 17.

Review of Bridge File

The owner of a movable bridge generally keeps a complete file available for the engineer who is responsible for the operation and maintenance of the bridge. Refer to Chapter 2 for information on planning the inspection. Refer to Chapter 3 and Chapter 18 for general record keeping and documentation. The file commonly includes (if applicable), but is not limited to, the following:

- Copy of the latest approved permit drawing.
- Inspection procedures.
- Complete set of design plans and special provisions.
- As-built plans for the structural steel, architectural, mechanical, hydraulic, and electrical systems.
- Machinery Maintenance Manual.
- Electrical Maintenance Manual.
- Hydraulic Maintenance Manual.
- Copy of maintenance methods being followed.
- Copy of the latest Operator's Instructions being followed.
- Copies of inspection reports.
- Copies of maintenance reports.
- Copies of repair plans.
- Up-to-date running log on all spare parts that are available, on order, or out of stock.
- Health and safety plan.
- Personnel safety plan.
- Public safety plan.
- Bridge-specific safety plan.

The bridge file should be reviewed with preventative maintenance measures in mind. An example would be the reading from a wiring insulation test; especially those taken on damp rainy days when moisture could influence (reduce) the values. An anticipated 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.

12.3.3 Inspection Areas and Deficiencies

During the inspection of any type of movable structures, inspectors should note any deficiencies that are detrimental to all steel and concrete structures. Most of the bridge structure material deficiencies that are listed in Chapter 7 as potential problems also apply to movable spans. Inspection areas are similar to those listed in Chapters 8-10, 12, and 13.

Fatigue can be a problem with movable bridges due to the reversal or the fluctuation of stresses as the spans open and close (see Figure 12.3.4). Any member or connection subject to such stress variations should be inspected carefully for signs of fatigue. The main longitudinal members experience negative moment when cantilevered in the open position and positive moment when they are in the closed and locked position.

Figure 12.3.4 Stress Reversals in Members

Control House

Inspection of the control house is necessary to ensure the safety of a movable bridge. The operator is responsible for public and navigational safety during operation and, with maintenance personnel, is usually the most familiar with any known structural or operational issues. Operational and maintenance logbooks can be found in the bridge file and may also be found at the control house for reference. The resources within the control house can 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.

Inspectors should consult with the bridge operators to ascertain whether there are any changes from the normal operation of the bridge. It should be noted whether Coast Guard, Corps of Engineers, and local instructional bulletins are posted. Inspectors should check for obvious hazardous operating conditions involving the safety of the operator, maintenance personnel, pedestrians, motorists, and helmsperson.

Inspectors should note the location of the control panel in relation to roadway and waterway and whether the bridge operator has a good view of approaching boats, vehicles, and pedestrians (see Figure 12.3.5). Inspectors should check operation of all closed-circuit TV equipment and evaluate its position for safe operation. If controls are in more than one location, the inspector should note description of the other locations and include their condition as well as the information about the control house. It should be noted whether alternate warning devices such as bullhorns, lanterns, flashing lights, or flags are available and in working order.

An inspection team should note whether the structure of the control house shows cracks and determine whether it is windproof and insulated. Inspectors should check for any accumulations of debris, that may be readily combustible, or signs of vermin infestation. Controllers should be checked when the bridge is opening and closing. Inspectors should look for excess play and for sparking during operation. It should be noted whether the submarine cables are kinked, hooked, or deteriorated, especially at the exposed area above or below the water. In tidal areas, the inspection team should check for marine and plant growth. It should be noted if the ends of the cable have been protected from moisture.

Figure 12.3.5 Control Panel

Counterweights and Attachments

The counterweights should be inspected to determine if they are sound and are properly affixed to the structure. Also, inspectors should 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. It should be determined whether the counterweight pockets are properly drained. On vertical lift bridges, inspectors should be sure that the sheaves and their supports are well-drained. Every portion of the bridge should be examined where water can collect. Pockets that are exposed to rain and snow should have a removable cover. The counterweight pit should be examined for water. The condition of the sump pump should be checked along with the concrete for cracks, and the entire area for debris, including birds, animals, and insect nests.

Where steel members pass through or are embedded in concrete, inspectors should check for any section loss due to corrosion of the steel member and for rust stains on the concrete. Where lift span counterweight ropes are balanced by chains (or other means), it should be verified that the links hang freely, and these devices along with slides, housings, and storage devices should be checked for deficiencies and for adequacy of lubrication, where applicable.

Piers

Refer to Chapter 14 for general inspection areas and deficiencies of common bridge substructures. Any rocking of the piers when the leaf is lifted should be noted. This is an indicator of a serious deficiency or critical finding, and immediate follow-up action is needed. Inspectors should survey the spans including towers to check both horizontal and vertical displacements. This should help to identify any foundation movements that have occurred.

The braces, bearings, and housings should be checked for cracks, especially where stress risers would tend to occur. Concrete should be inspected for cracks in areas where machinery bearing plates or braces are attached (see Figure 12.3.6). The tightness of bolts and the tightness of other fastening devices used should be noted. Inspectors should check any pier protection system that may exist, such as dolphins or fenders (see Figure 12.3.7).

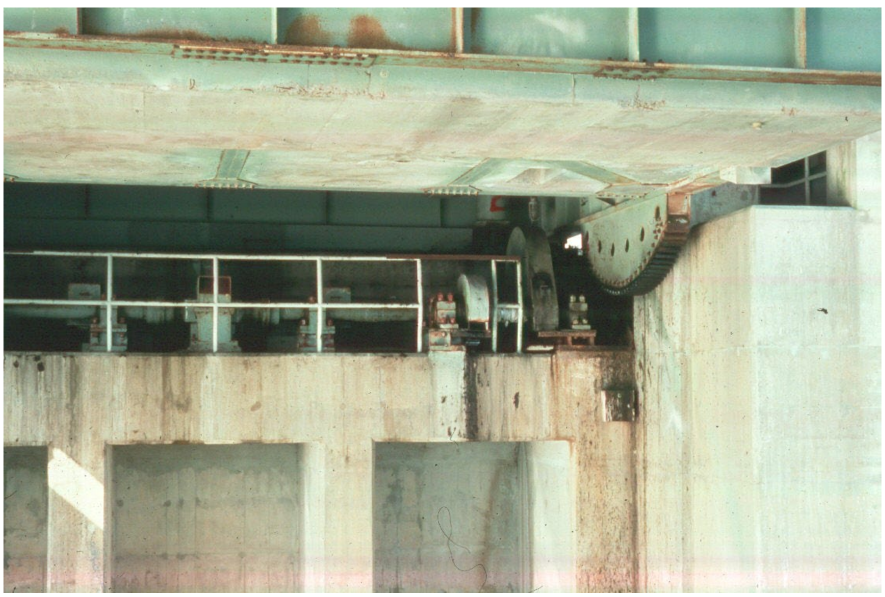


Figure 12.3.6 Concrete Bearing Area

Figure 12.3.7 Pier Protection System (Fenders Shown)

Decks

It should be verified that structural welds are sound and the amount of remaining serration on the top edges of grid decks is sufficient for skid resistance. The roadway surface should be checked 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, refer to Chapter 8.

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, the ceiling should be checked for leaks or areas that allow debris and rust to fall on the machinery. For more information of concrete decks, refer to Chapter 8.

Special Machinery for Swing Bridges

Center bearings should be checked for proper and adequate lubrication, oil leaks, and noise. The housing should be examined for cracking, pitting, fit of joints, and indications of span translation (irregular rotation) at racks and track should be noted. Inspectors should measure for proper clearance of balance wheels above track. It should be verified that the tracks and balance wheels are free of wear, pitting, and cracking. Inspectors should check for proper and adequate lubrication at all lubrication points.

An inspector should note balance characteristics as indicated by loads taken by balance wheels, and by drag on the rest pier rail.

The rim bearing should be checked for wear on tracks and rollers, particularly at rest positions where the bridge is carrying traffic. The center pivots and guide rings should be examined for proper fit, and for wear, pitting, and cracking. Inspectors should check for proper and adequate lubrication at all lubrication points.

The center (live load) wedges under the trusses or girders at the pivot pier should be examined for proper fit (no lifting) and alignment. Inspectors should check end wedges and bearings at the rest piers for alignment and amount of lift. This can be recognized by excessive vibration of the span or uplift when live load crosses the other span. The end lift jacks, shoes, and all linkages should be inspected for wear, proper bearing under load, and proper adjustment.

The condition of end latches should be noted, including any modification that adversely affects their functional design.

Special Machinery for Bascule Bridges

On rolling lift bascule bridges, inspectors should check the segmental and track castings and their respective supporting track girders (if used) for wear on the sides of track teeth due to movement of sockets on segmental castings. The trunnion assemblies should be inspected for deflection, buckling, lateral slip, and loose bolts. Trunnion bearings should be checked for lubrication of the full width of the bearing. It should be verified that extreme pressure (EP) lubrication oil of the proper grade is used. The trunnions should be examined for any signs of corrosion, pitting, or cracking, particularly at stress risers. Laser leveling may be used during the inspection of trunnions. The balance of each leaf should be checked. Inspectors should compare all wear patterns for indications of movement of the leaves. Inspectors should also 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. Rack support should be inspected for lateral movement when bridge is in motion.

On bascule bridges, inspectors should check if the front live load bearings fit snugly. Inspectors should also observe the fit of tail locks at rear arm and of supports at outer end of single-leaf bridges.

On double-leafed bascule bridges, inspectors should measure the differential vertical movement at the joint between the two leaves under heavy loads. On other types, inspectors should 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. The joint between the two leaves on double-leaf bascule bridges, or the joints between fixed and movable portions of the structure should be inspected for adequate longitudinal clearance for change in temperature (thermal expansion).

On multi-trunnion bascule bridges, the strut connecting the counterweight trunnion to the counterweight should be checked 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.

Special Machinery for Vertical Lift Bridges

Inspectors should look for flattening or fraying of the rope strands and deficiencies. This is reason for replacement. Similarly, inspectors should check the up-haul and down-haul ropes to observe if they are winding and unwinding properly on the drums. Any need for tension adjustments in up-haul and down-haul ropes should be noted. Inspectors should determine whether ropes have freedom of movement and are running properly in sheave grooves. Inspectors should also look for any obstructions to prevent movement of the ropes through the pulley system and check the supports on span drive type bridges. Rope guides should be checked for alignment, proper fit, free movement, wear, and structural integrity of the longitudinal and transverse grooved guide castings. The grooved guide castings should be inspected closely for wear in the grooves. The cable hold-downs, turnbuckles, cleats, guides, clamps, splay castings, and the travel rollers and their guides should be examined.

The balance chains should be checked that they hang freely, that span leveling devices are functioning, and that span and counterweight balance closely. Inspectors should observe if span becomes "out of level" during lifting operation. Spring tension, brackets, braces, and connectors of power cable reels should be inspected.

Inspectors should check for damage, including cracking, at drums and sheaves. The condition and alignment of span guides should be noted.

Other Structural Considerations

Other structural considerations include:

- The live load bearings and wedges under the trusses or girders at the pivot pier should be examined for proper fit alignment and amount of lift.
- The fully open bumper blocks and the attaching bolts should be inspected for cracks in the concrete bases.
- Inspectors should check 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.

Mechanical Inspection Considerations

Mechanical, hydraulic, and electrical 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 established programs of inspection, maintenance, and repair. Inspectors should coordinate with the owner or bridge operator to check if they are aware of any issues with the bridge. However, there is always the possibility that some important issue may have been overlooked. Any problems noted during the inspection should be reported to the owner.

The items covered and termed as machinery include all motors, brakes, gears, tracks, shafts, couplings, bearings, locks, linkages, over-speed controls, and any other integral part that transmits the necessary mechanical power to operate the movable portion of the bridge. Machinery should be inspected, not only for its current condition, but also for operational and maintenance methods and analysis of the characteristics of operation. The items described below, and similar items should be inspected and analyzed by a machinery or movable bridge specialist.

Inspectors should perform an evaluation of maintenance methods. Application methods and frequency of lubrication in the maintenance logbook should be checked, if available. General appearance of existing applied lubricant should be noted.

Reference the *AASHTO Movable Bridge Inspection, Evaluation and Maintenance Manual, 2nd Edition, 2017* for further information on inspecting these items.

Open Gearing

Inspectors should check open gearing for tooth condition and alignment including over-engagement and under-engagement. It should be verified that the pitch lines match. Any excessive or abnormal wear should be noted. The teeth, spokes, and hub should be inspected for cracks. Inspectors should 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, inspectors should recommend the contaminated lubricant to be removed and a new lubricant be applied. Inspectors should also ask for a copy of the work order to verify the maintenance has been completed. The teeth of all gears should be checked for wear, cleanliness, corrosion, and for proper alignment.

Speed Reducers, Including Differentials

The exterior of the housing and mountings of speed reducers should be examined for cracks and deficiencies (see Figure 12.3.8 and Figure 12.3.9). Bolts should be checked for tightness and any corrosion should be noted. The interior of the housing should be inspected for condensation and corrosion. The condition of gears should be checked and noted. Inspectors should watch for abnormal shaft movement during operation, indicating bearing and seal wear. Oil levels and condition of lubricant should be periodically checked. Inspectors should check that circulating pumps and lubricating lines are properly operating. Any abnormal noise should be documented. Leaking oil may indicate the presence of a crack, loose seals, or damaged bearings.

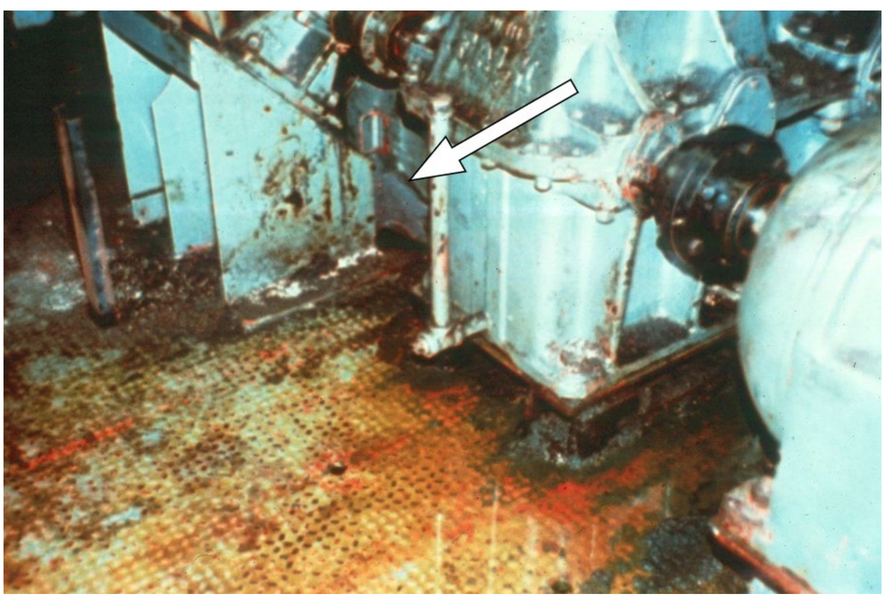


Figure 12.3.8 Cracked Speed Reducer Housing

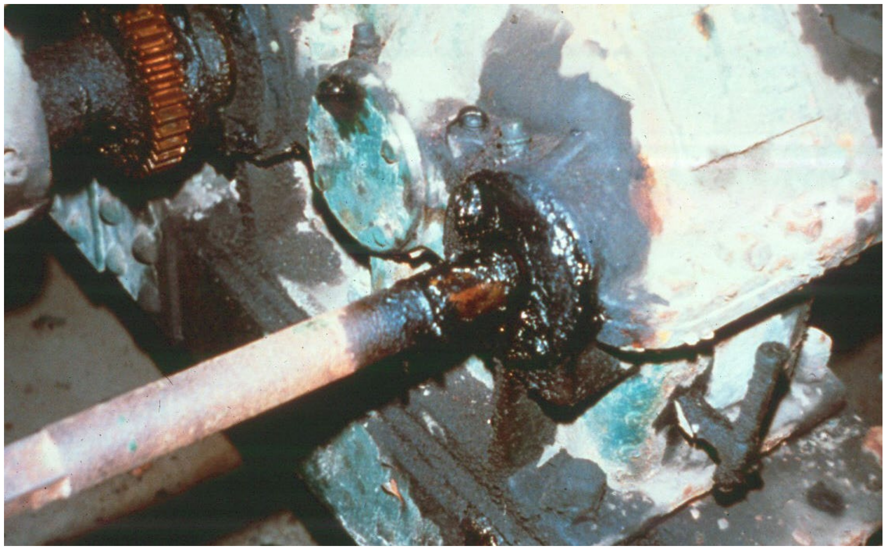


Figure 12.3.9 Leaking Speed Reducer

Shafts and Couplings

Inspectors should examine shafts for damage, twisting, and strain. Cracks, if suspected, may be detected using nondestructive evaluations such as magnetic particle or dye penetrant (see Figure 12.3.10). Various advanced inspection methods for steel members are presented in Chapter 17. Cracks in mechanical parts may be determined to be a critical finding. Misalignment with other parts of the machinery system should be noted. Cracks in shafts and their exact locations should be documented. Other shafts in the same locations should be examined, as they are likely the same material and fabricated to the same details. They have also likely been exposed to the same magnitude and frequency of loading. Inspectors should check coupling hubs, housings, and bolts for condition. Seals and gaskets should be inspected for leaks. Internal inspection of couplings is warranted if problems are suspected and can be used to determine tooth wear in gear couplings.

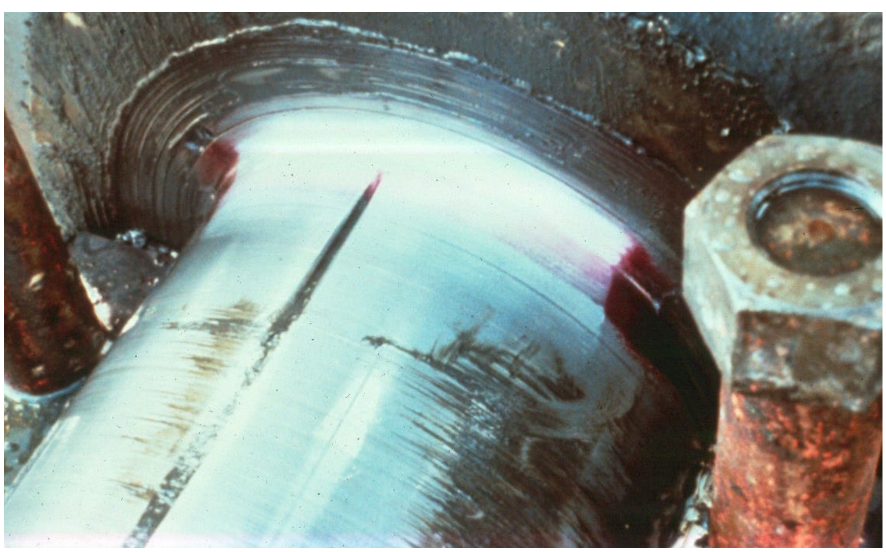


Figure 12.3.10 Hairline Crack Revealed on Shaft from Dye Penetrant Test

Bearings

Bearing housings, pedestals, and supports should be examined for external condition, noting any cracks. Bolts in housings and those used as anchors should be checked for tightness, damage, and corrosion, noting apparent lubrication characteristics (see Figure 12.3.11). Grinding noises can be caused by lack of lubricant. In sleeve bearings, the bushings should be inspected for damage and excessive wear. Evidence of seal damage in anti-friction bearings should be noted. Any unusual noise should be investigated. The trunnion bearings should be checked for excessive wear, lateral slip, and loose bolts.

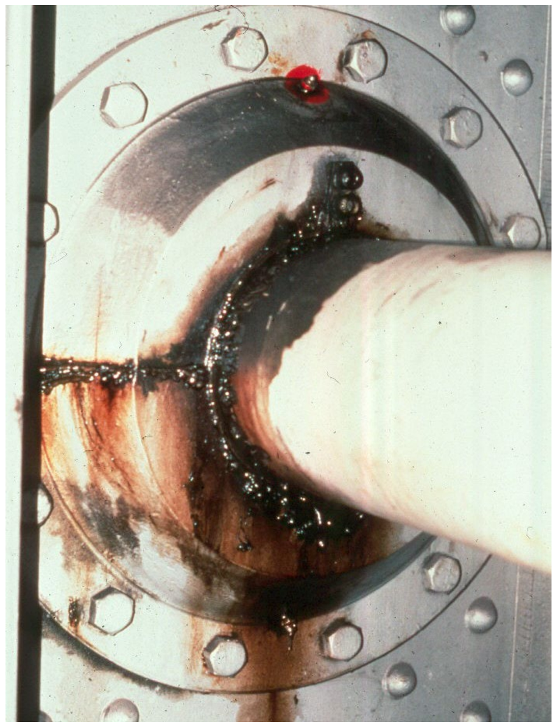


Figure 12.3.11 Leaking Bearing

Brakes

Braking devices should be inspected for proper setting of braking torque and for complete release of the brakes when actuated. On shoe brakes, drums and shoes should be checked for wear, damage, and corrosion, for misalignment of shoes with drums, and for clearance when released. Inspectors should determine if worn brake linings need replaced. Inspectors should also check for proper actuation without leakage by actuators. It should be verified that linkages and hand releases are free but not too loose. On enclosed hydraulic disc brakes, inspectors should make certain there is proper actuation without leakage at connections or seals. The brakes, limit switches, and stops (cylinders and others) should be checked for excessive wear and slip movement. It should be noted whether the cushion cylinder ram sticks or inserts too easily. The brake limit switches should be inspected for proper setting. The surface of the brake drum should be investigated for indications of contact with the brake shoes. The pressure developed by each disc brake power unit should be checked to be sure the brakes are releasing. Inspectors should also check the manual release on all brakes.

Drives - Electric Motors and Hydraulic Equipment

The housing and mountings should be checked for damage, corrosion, and fastener condition. Bearings should be inspected for lubrication and indications of wear (movement) and seal leakage at shaft extensions should be noted.

Inspectors should look for any leakage at connections and seals. Any corrosion on the cylinder rods should be noted. Inspectors should listen to motors and pumps and note any unusual noise. Power units should be checked to make sure that everything is functioning and that pressures are properly adjusted. Fluid should be periodically sampled and examined for contamination and wear metal. All main hydraulic power units should be checked for charge pressure setting and maximum pressure that can be developed by the unit. All filters should be routinely checked and replaced as needed. The fluid level in the reservoir should also be checked by the inspection team.

Auxiliary Drives

Emergency generators should be checked for operation and readiness, verifying that there are no oil leaks or abnormal noises. Mechanical service specialists and electrical inspectors are needed for more thorough inspections. Auxiliary motors and hand operators, with their clutches and other transmission parts, should be checked for adjustment and readiness to perform.

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 a few conditions. Any wear or slippage should be investigated if a belt drive is used. The condition of all belts and the need for replacement, if any, should be noted. If a friction drive is used, all bracing and bearings should be checked for tightness. If a liquid coupling is used, it should be ensured that the proper quantity of fluid is used. Inspectors should look for leaks.

Locks

The center locks and tail locks (if used) on double-leaf bascule spans, and the end locks on single-leaf bascule bridges, swing bridges, and vertical lift bridges should be examined. It should be noted whether there is excessive deflection at these joints or vibration on the bridge. The locks should be inspected for fit and for movement of the span or leaf (or leaves). Inspectors should check lubrication and for loose bolts. It should be verified that the lock housing and its braces have no noticeable movement or misalignment. The paint adjacent to the locks might have signs of paint loss or wear if there is movement. Inspectors should check lock bars, movable posts, linkages, sockets, bushings, and supports for damage, cracks, wear, and corrosion.

Rear locks in the withdrawn position should be checked for clearance from the path of the moving leaf as it opens and should be checked for full engagement when the leaf is closed. The gap, if any, should be measured between the lock plate and the moving leaf bearing plate. Each rear lock hydraulic drive unit should be checked for leakage of oil and operation for correct length of movement of the lock.

On bascule bridges, inspectors should check if the front live load bearings fit snugly. Also, inspectors should observe the fit of tail locks at the rear arm and of supports at the outer end of single-leaf bridges.

Actuators should be examined for operational characteristics, including leakage if hydraulic. Both the quantity and quality of the lubricant should be noted. Alignment should be checked and the type of wear that is occurring should be analyzed.

Live Load Shoes and Strike Plates

The live load shoes and strike plates should be inspected for deficiencies and corrosion. Inspectors should investigate the fasteners and should note contact surface conditions. Alignment and movement under load should be checked.

Air Buffer Cylinders and Shock Absorbers

Within air buffer cylinders and shock absorbers, indications of lack of pressure or stickiness during operation should be noted. Piston rod alignment should be checked with strike plate. The condition of the rod and housing should be noted and inspectors should verify if hydraulic fluid leakage is present. The air filter and function of any pressure reading or adjusting devices should be checked along with the operating pressure, if possible. It should be verified that the air buffers have freedom of movement and development of pressure when closing. The fully open bumper blocks and the attaching bolts should be examined for cracks in the concrete bases.

Machinery Frames, Supports, and Foundations

Inspectors should check that there is no cracking in the steel or concrete. Corrosion and damage should be noted. Deflection and movement under load should be examined. The linkages and pin connections should be ensured to have the proper adjustment and should be verified to be in functional condition. Motor mounting brackets should be checked to ensure they are secure and tight.

Wedges

The wedges and the outer bearings at the rest piers should be checked for alignment and amount of lift. This can be recognized by excessive vibration of span or uplift when load comes upon the other span. The live load bearings and wedges under the trusses or girders at the pivot pier should be examined for proper fit alignment and amount of lift.

Hydraulic Inspection Considerations

A hydraulic power specialist is needed for the inspection of the hydraulic equipment. The functional operation of the bridge should be observed and investigated for abnormal performance of the equipment.

Inspectors should 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 the equipment, the needs for fluid cleanliness, and the need for personnel safety, the inspection team should not open the reservoir or hydraulic lines. In addition, the inspection team should not shut off or adjust any part of the power circuit without complete understanding of their function and knowledge of the effect such action will most likely have upon the system.

Reference the *AASHTO Movable Bridge Inspection, Evaluation and Maintenance Manual, 2nd Edition, 2017* for further information on inspecting these items.

Potential issues checked during a hydraulic inspection include the following:

- Leakage anywhere in the system should be noted.
- Significant leakage (inspectors should immediately inform the bridge authority).
- Inspectors should check for corrosion of reservoir, piping, and connections. Sight gauges should be inspected for proper fluid level in reservoir.
- Gauges with low fluid levels, gauges exhibiting damage, or gauges that cannot be read should be noted.
- Unusual noises from any part of the system should be noted. Filter indicators should be checked to make sure filters are clean.

A sample of the hydraulic fluid should be collected for analysis by a testing laboratory during periodic inspections.

Electrical Inspection Considerations

An available electrical specialist is necessary for the inspection of the electrical equipment. The functional operation of the bridge should be observed and abnormal performance of the equipment should be noted. The operational methods and safety features provided should be checked. The maintenance methods being followed should be evaluated and the frequency of services performed should be checked.

Reference the *AASHTO Movable Bridge Inspection, Evaluation and Maintenance Manual, 2nd Edition, 2017* for further information on inspecting these items.

Power Supplies

Inspectors should examine the normal power supply, standby power supply, and standby generator set (for emergency operation of bridge and service lighting) and should note the following:

- Insulation readings on coated wiring should be taken to determine the condition of the insulation, noting the weather conditions, namely temperature and humidity.
- Inspectors should make sure all cable connections are properly tightened.
- The voltage and the current to the motors should be measured at regular intervals during the operation of the bridge.
- The collector rings and windings should be checked on the generator set.
- Starting circuitry should be tested for automatic starting and manual starting.
- Inspectors should check if the unit is vibrating when running under load.
- Inspectors should make sure cable splices are in good condition.

If no standby power supply has been provided, inspectors should determine whether a portable generator and manual transfer switch may be applicable for this bridge.

Motors

Span drive motors, lock motors, brake thruster motors, and brake solenoids should be examined for the same items as given for power supplies.

Transformers

Dry transformer coil housings, terminals, and insulators should be checked, along with their temperature under load. The frames and supports should be examined for rigidity to prevent vibration. Liquid-filled transformer should be checked in the same way, along with checking the oil level and quality when looking for leakage. Oil insulation test records should be examined.

Circuit Breakers

Circuit breakers (e.g., air, molded case, and oil) and fuses, including the arc chute, contact surfaces, overload trip settings, insulation, and terminal connections should be checked. Oil insulation test records should be examined. Inspectors should observe the closing and tripping operation. All fuse types and sizes being used should be recorded.

Wires and Cables

Both the power and bridge control wiring and cables should be examined. Inspectors should note whether the submarine cables are kinked, hooked, or deteriorated, especially at the exposed area above and below the water. In tidal areas, inspectors should look for marine and plant growth. It should be noted if the ends of the cable have been protected from moisture. The insulation value of each wire should be recorded as measured. Inspectors should look for cracking, overheating, and deterioration of the insulation. Wires and cables should be checked for wear against surfaces and especially sharp edges. Inspectors should check the adequacy of supports and that dirt and debris do not accumulate against the conduit and supports. Terminal connections, clamps, and securing clips should be checked for tightness, corrosion, and that there are wire numbers on the end of each wire. The weight of the wires or cables is designed to be carried by clamps, clips, or fittings and not by the wiring itself.

Conduit

It should be checked if conduit is far enough away from surfaces to avoid debris from collecting against it. It should be noted if conduit is adequately supported and pitched to drain away from junction boxes and pull boxes, so that water is not trapped within. Also, it should be noted if conduits have covers with seals. Deteriorated conduit should be reported so that it can be replaced with new conduit. PVC conduits should be checked for signs of PVC cement at fittings and connections to ensure conduit is properly connected. PVC cement is usually dark gray or purple.

Cabinets

The programmable logic controller (PLC) cabinets, control consoles and stations, switchboards (see Figure 12.3.12), relay cabinets, motor control centers (MCC), and all enclosures should be examined for deficiencies, debris, drainage, corrosion, signs of vermin nesting, operations of heater to prevent condensation, and their ability to protect the equipment inside.

Figure 12.3.12 Open Switchboard

Inspectors should check the operation of traffic signals, traffic gates, traffic barriers, and navigation lights. It should be verified that the bridge is open to provide the clearance shown on the permit drawing before the green span light turns on. The traffic warning equipment and control circuits, including the advanced warning signals (if used), traffic signals, gates, barriers, and the public address and communication equipment should be examined.

Junction Boxes

The covers on junction boxes (JBs) should be examined for an effective seal, dry interior, functioning breather-drains, heaters having enough power to prevent condensation inside, and terminal strips all secured to the bottom of horizontal JBs or to the back of vertical JBs.

Meters

Inspectors should observe if voltmeters, ammeters, and watt meters are freely fluctuating with a change in load. Inspectors should check that switches and meters are operable.

Control Starters and Contactors/Relays

The operation of this equipment should be checked under load, and watched for arcing between contacts, snap action of contacts, deterioration of any surfaces, and drainage of any moisture. Inspectors should look for signs of corrosion and overheating.

Limit Switches

Inspectors should check if limit switches are operating properly and if not, they should notify the appropriate personnel. It should be verified that the interior is clean, dry, and springs are active.

Transmitters and Receivers

Inspectors should check for power to the field and signal being sent from the transmitter to the receiver. The receiver tracking the rotation of the bridge should be observed as it operates. The mechanical coupling between the driving shaft and the transmitter, should be examined, checking for damage and misalignment.

Service Lighting and Electrical Outlets

It should be checked if power is going to each light and outlet. Inspectors should note if there is a shield or bar present for protecting each bulb and socket. It is desirable to have service lights available when power is removed from movable bridge controls and equipment.

12.3.4 Evaluation

Rating systems have not been specifically developed to aid in the inspection of movable bridges. These systems currently only address the structural aspects of the movable bridges, the typical concrete, steel, and timber components. Agencies may develop other rating systems for the mechanical and electrical portions.

The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specification for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Also reference the *AASHTO Movable Bridge Inspection, Evaluation, and Maintenance Manual* that includes information about movable bridge element descriptions for agency defined elements (ADEs): structural, mechanical, and electrical.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

Section 12.4 Floating Bridges

12.4.1 Overview of Common Deficiencies

Common deficiencies that occur on floating bridges are:

- Cracking.
- Spalling.
- Delamination.
- Overload damage.
- Collision damage.
- Abrasion.
- Loss of watertight seals on access doors and hatches.
- Damaged cable connections.
- Coating failure.
- Corrosion and section loss.
- Inherent defects.
- Fungi.
- Insects and marine borers.
- Chemical attack.

Floating bridges may be constructed from steel, concrete, or timber. Therefore, deficiencies will most likely depend on the material used to construct the bridge. Refer to Chapter 7 for specific information regarding deficiencies of each material type.

12.4.2 Inspection Methods

Because of their uniqueness and depending on the material used, floating bridges can prove challenging to an inspection team. 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 movable bridge section, those inspection methods and areas should be considered by the inspection team.

Like movable bridges, floating bridges may also have mechanical and electrical aspects. Refer to Section 12.3.3. Inspectors should consider these systems when planning the inspection.

Visual and Physical

Visual inspection of each pontoon cell should reveal any surface deficiencies, such as cracks or leaks. Pontoons have access hatches to allow for maintenance and inspection (see Figure 12.4.1). Pontoons are typically considered confined spaces. Refer to Chapter 2 for an explanation of confined spaces and the associated inspection guidelines.

Figure 12.4.1 Inspector Opening Pontoon Access Hatch

Inspectors should measure and record the depth of any water found in each cell. The length, location and width of cracks found should be accurately measured and recorded. For steel pontoons and cables, corrosion and rust should be removed down to bare metal and calipers or a D-meter should be used to measure and record remaining section thickness or diameter. A hammer should be used to check for delaminated areas in concrete pontoons.

Advanced Inspection Methods

Many of the advanced inspection tools used above water have been adopted for underwater use. Refer to Chapter 17 for the advanced inspection methods of timber, steel, and concrete.

Anchors may be embedded 100 or more feet below the water surface. Underwater divers and equipment with the ability to detect any deficiencies are necessary for inspection of anchors. Refer to Chapter 16 for Underwater Inspection information. Underwater cameras, sonar, and other specialized equipment can provide access to cables and anchors.

Review of Bridge File

The owner of floating bridges generally keeps a complete file available for the engineer who is responsible for the inspection and maintenance of the bridge. Review the bridge file with preventative maintenance measures in mind. Refer to Chapter 2 for information on planning the inspection. Refer to Chapter 3 and Chapter 18 for general record keeping and documentation.

The file commonly includes (if applicable), but is not limited to, the following:

- Inspection procedures for floating bridges.
- Bridge-specific information for underwater inspections.
- Complete set of design plans and special provisions.
- As-built plans for the bridge.
- Copies of inspection reports.
- Copies of maintenance reports.
- Copies of repair plans.
- Navigational waterway guidelines.
- Health and safety plan.
- Personnel safety plan.
- Public safety plan.
- Bridge-specific safety plan.

12.4.3 Inspection Areas and Deficiencies

During the inspection of any type of floating structure, the inspection team should be sure to note any deficiencies that are detrimental to the bridge members. Most of the bridge structure material deficiencies that are listed in Chapter 7 as potential problems also apply to floating bridges. Some of the unique inspection areas are explained in this section. Inspection areas are similar to those listed in Chapters 8-10, 11, 13, and 14 for other portions of the bridge.

Pontoons

The floor of each pontoon cell should be examined for standing water. Pontoon walls and surfaces should be inspected for cracks and leaks. Inspectors should examine access doors, locks and hatches verifying that they are watertight and in proper working condition. The bilge pumping system should be checked and verified that it is in working order. Any noted problems with the pumping system should be conveyed to specialized maintenance personnel responsible for the system. The anchor cable saddle inside the pontoon should be examined. Inspectors should verify the soundness of connections in the pontoons. The presence and function of any cathodic protection system on the anchor cables should also be verified.

Joints

When continuous pontoons are utilized, the joint between the pontoons should be inspected. Typically, a rubber membrane or grout is used between the pontoons. The alignment of the pontoons across the structure should be examined 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

Inspectors should examine the cable ends at the pontoon portals and should check for cable misalignment and fraying. Inspectors should check for broken wires that may indicate undue stress on the cable securing the pontoon (see Figure 12.4.2). Cables should also be checked for corrosion or section loss (see Figure 12.4.3).

Figure 12.4.2 Frayed Cables Removed from a Floating Bridge

Figure 12.4.3 Cable 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. Inspectors should look for any indication of anchor movement, misalignment, or undermining of the anchor. The ballast on open-cell gravity block anchors should be checked to verify if there is enough material to keep the anchors in place.

12.4.4 Evaluation

Rating systems have not been specifically developed to aid in the inspection of floating bridges. These systems currently only address the structural aspects of the floating bridges, the typical concrete, steel, and timber components. Agencies may develop other rating systems for the mechanical and electrical portions.

The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specification for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

Section 12.5 Ferry Transfer Bridges

12.5.1 Overview of Common Deficiencies

Common deficiencies that occur on ferry transfer bridges are:

- Corrosion.
- Fatigue cracking.
- Overloads.
- Coating failure.

Refer to Chapter 7 for a more detailed presentation of the properties of steel, types and causes of steel deficiencies, and the examination of steel.

12.5.2 Inspection Methods

Like other movable bridges, ferry ramps may have a traffic control gate to regulate the traffic entering the bridge (see Figure 12.5.1). The inspection methods for ferry transfer bridges should parallel the methods described in Section 12.3.2 for movable bridges, with regards to applicable safety concerns.

Figure 12.5.1 Ferry Transfer Bridge Traffic Control Gate

Visual and Physical

Refer to Chapter 10 for visual and physical inspection methods for steel superstructures.

Advanced Inspection

Refer to Chapter 10 for visual and physical inspection methods for steel and Chapter 17 for advanced inspection methods.

Review of Bridge File

A ferry transfer bridge is similar to a movable bridge, in that some additional considerations for inspection and the preparation for that inspection are necessary. Refer to Chapter 2 for information on planning the inspection. Refer to Chapter 3 and Chapter 18 for general record keeping and documentation.

The bridge file for a ferry transfer bridge may include:

- Inspection procedures.
- Complete set of design plans and special provisions.
- As-built plans for the structural steel, architectural, mechanical, hydraulic, and electrical systems.
- Machinery Maintenance Manual.
- Electrical Maintenance Manual.
- Hydraulic Maintenance Manual.
- Copy of maintenance methods being followed.
- Copy of the latest Operator's Instructions being followed.
- Copies of inspection reports.
- Copies of maintenance reports.
- Copies of repair plans.
- Up-to-date running log on all spare parts that are available, on order, or out of stock.
- Health and safety plan.
- Personnel safety plan.
- Public safety plan.
- Bridge-specific safety plan.

12.5.3 Inspection Areas and Deficiencies

Ferry transfer bridges have many of the same members and other details as movable bridges and therefore the same areas of focus. Refer to Section 12.3.2 for specific mechanical and electrical systems and deficiencies to inspect.

12.5.4 Evaluation

Rating systems have not been specifically developed to aid in the inspection of ferry transfer bridges. These systems currently only address the structural aspects of the ferry transfer bridges, the typical concrete, steel, and timber components. Agencies may develop other rating systems for the mechanical and electrical portions.

The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

This page intentionally left blank.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 13 TABLE OF CONTENTS

CHAPTER 13 LIST OF FIGURES

Chapter 13 Inspection and Evaluation of Bridge Bearings

Section 13.1 Introduction

A bridge bearing is a component that provides an interface between the superstructure and the substructure. The three primary functions of a bridge bearing, shown in Figure 13.1.1, can be summarized as:

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

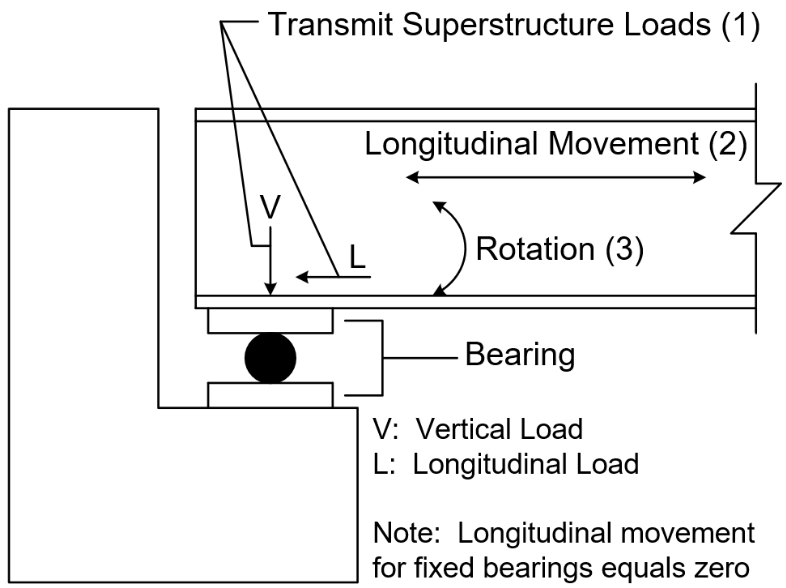


Figure 13.1.1 Three Functions of a Bearing

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

Bearings that are not designed to provide for longitudinal translation or movement of the superstructure can be referred to as fixed bearings. Bearings that do provide for longitudinal translation or movement of the superstructure are generally known as movable bearings. Both fixed and movable bearings permit rotation that occurs as loads are applied or removed from the bridge (see Figure 13.1.2). There may be occasions such as short span concrete structures (spans less than 50 ft) where the designer may not provide for bearing rotation and omit the bearings entirely. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

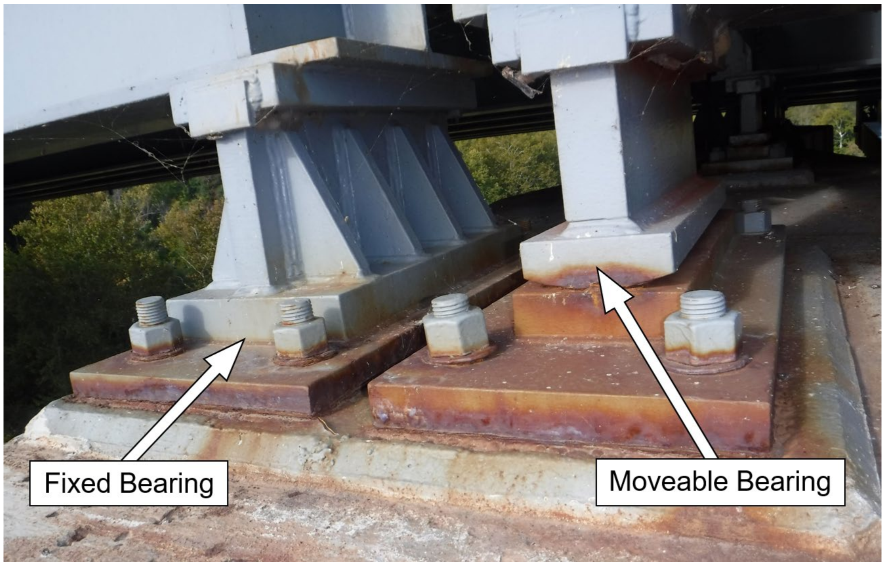


Figure 13.1.2 Fixed and Movable Bearings

Section 13.2 Characteristics

13.2.1 Basic Parts of a Typical Bearing

A bridge bearing typically consists of four basic parts; sole plate, bearing or bearing device, masonry plate, and anchor bolts (see Figure 13.2.1). Some bearings may not have all four basic parts. On older structures there may also be a levelling plate/pad or a mortar pad under the masonry plate to accommodate for bridge construction intolerances. With advances in construction surveying practices, this is rarely found on newly built structures.

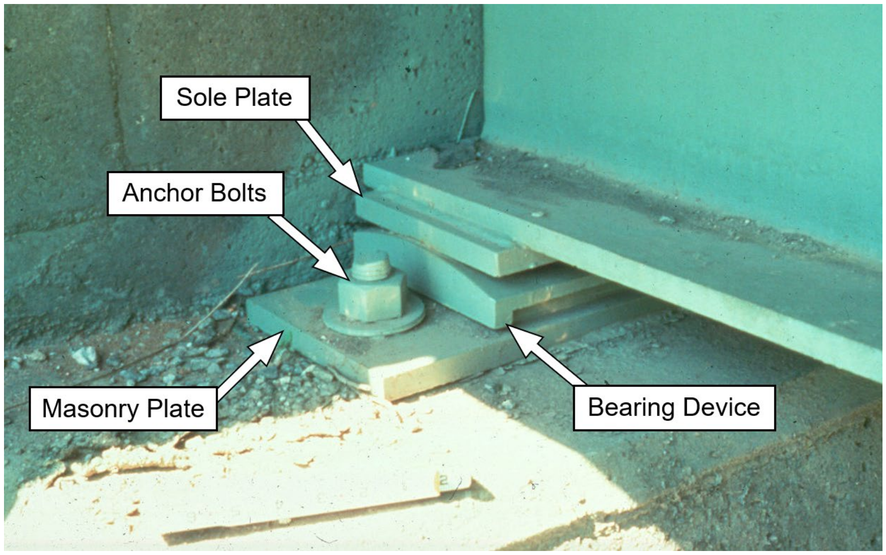


Figure 13.2.1 Basic Parts of a Typical Bridge Bearing

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

Sole Plate

The sole plate distributes forces from the superstructure to the bearing device. It typically consists of a steel plate that is attached to the bottom of the superstructure member. A sole plate may also be embedded into the bottom flange of a prestressed concrete girder. With some 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 can usually be found between the sole plate and masonry plate (when present). It provides the function of transmitting the forces from the sole plate to the masonry plate and may provide for longitudinal movement and rotation.

Masonry Plate

The masonry plate (when present) is typically a steel plate that is supported by the bearing seat of an abutment or pier. The masonry plate distributes vertical forces from the bearing to the substructure unit. Masonry plates may not be necessary for some bearing types such as elastomeric.

Anchor Bolts

The anchor bolts or rods (when present) connect the bearing device to the substructure unit. Anchor bolts are designed to restrain the masonry plate from longitudinal translation. The anchor bolts can, however, pass through or alongside the movable bearing element to provide restraint against transverse movement. If anchor bolts are not present, neoprene bearings may have been attached directly to the concrete with an epoxy compound. When an epoxy compound is not specified, the designers intend for the coefficient of friction between the pads and substructure to maintain proper alignment and contact between the bearing and substructure.

Section 13.3 Types of Bearings

The *AASHTO Manual for Bridge Element Inspection* classifies bearings into these seven categories:

- Movable bearings.
- Elastomeric bearings.
- Pot bearings.
- Disk bearings.
- Enclosed or concealed bearings.
- Fixed bearings.
- Other bearings.

13.3.1 Movable Bearings

Various movable bearing types have evolved out of the demand to accommodate superstructure movement, both reliably and efficiently. Seasonal changes impact the maximum and minimum ambient temperatures. Movable bearings can provide for movement due to these fluctuations in temperature.

Types of movable bearings include (*AASHTO Manual for Bridge Element Inspection*):

- 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 ft. Longitudinal movement is provided by one plate sliding upon another. The basic difference between types of sliding plate bearings is the method of lubrication. The most common types of sliding plate bearings are presented below.

Lubricated Steel Plates

The first generation of lubricated steel plates consisted of two steel plates with the surfaces of mating plates milled smooth (see Figure 13.3.1). Lubrication between the plates consisted of grease, graphite, or tallow, which is a grease substance made from animal fat. However, the lubricant typically held dirt that 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.

Figure 13.3.1 Lubricated Steel Plate Bearing

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 typically wore a groove in the larger one, to the point where the bearing was unable to move.

Lead Sheets Between Steel Plates

By placing a thin lead sheet between the steel plates, it may be possible to keep the plates from freezing to each other when they corrode. Lead sheets are generally used to reduce corrosion between the plates, thereby providing more freedom of movement. However, in this type of bearing, the lead sheet tends to shift and move out from between the plates. Lead is also considered a hazardous material (OSHA).

Bronze Bearing Plates

A bronze bearing plate was introduced between sole and masonry plates to avoid the corrosion problems of steel plates in contact with one another (see Figure 13.3.2). In lieu of corroding, bronze oxidizes when exposed to air and forms a patina, similar to weathering steel. A brown/blue-green coating on bronze is a sign of harmless corrosion. The lack of corrosion was used to maintain the freedom of movement. Although corrosion is reduced, the bronze, which has a higher ductility/lesser hardness than steel, becomes worn due to trapped dirt and the action of expansion and contraction. Eventually, a freezing of the plates may take place.

Figure 13.3.2 Bronze Sliding Plate Bearing

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 mating surfaces of the bearing device are removed and replaced with a graphite compound, which continuously lubricates the bearing surfaces. Some manufacturers claim that these bearings are corrosion resistant and maintenance is not necessary. 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. Rotational movement is provided by the radius side (bottom) (see Figure 13.3.3).

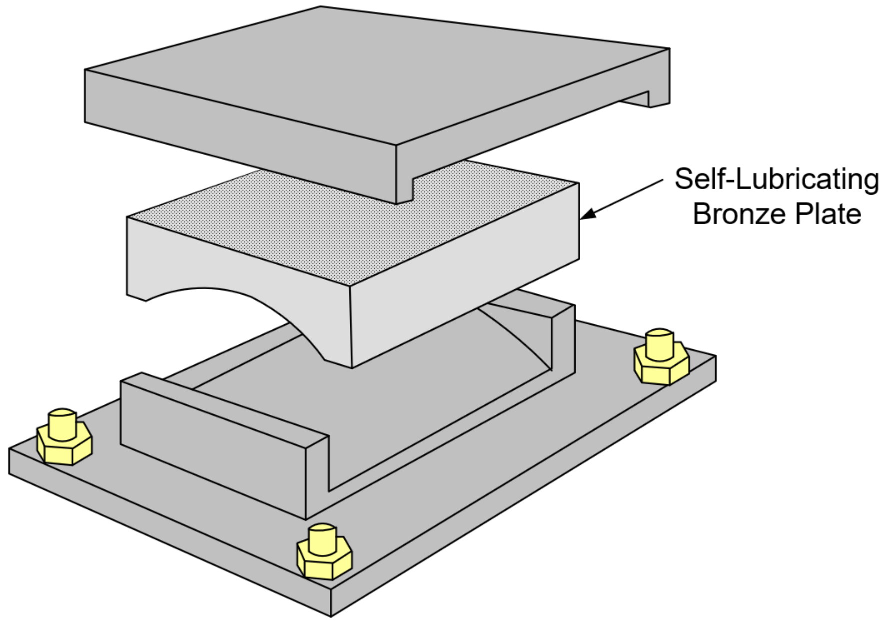


Figure 13.3.3 Self-Lubricating Bronze Sliding Plate Bearing

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 typically placed on the bridge seat with the superstructure placed directly on it. This is a simple but effective bearing that was commonly used on short span concrete slabs and girders that sit directly on concrete abutments. These bearing types provide limited longitudinal movement and are not typically used today in new construction applications.

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 movable 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. These fillers give strength to the PTFE and do not affect its low coefficient of friction.

Roller Bearings

A roller bearing consists of one or more cylinders that "roll" between the sole plate and masonry plate as the superstructure expands and contracts (see Figure 13.3.4). Roller bearings can be used in a wide variety of forms including single rollers and roller nests.

Single Roller Bearings

The single roller is one of the simplest types of movable bearings (see Figure 13.3.4). Rollers can vary in size, with specified diameters ranging from 6 to 15 inches. The larger rollers are less susceptible to corrosion problems, but 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.

Figure 13.3.4 Single Roller Bearing

An unrestrained roller may gradually work itself out from underneath the bridge superstructure. For this reason, pintle pins are typically used to keep the roller in place (see Figure 13.3.5). These pins fit tightly into the roller but loosely into the upper and lower plates. The loose fit provides for the necessary structure movement. If pintles are not present, tabs or bars may be attached to the masonry plate to prevent the roller from rolling out of position, as shown in Figure 13.3.4.

Roller Nest Bearings

First used in steel bridges in the early 1900s, roller nests consist of a group of rollers, each about 1.5 to 4 inches in diameter and may or may not incorporate pintles (see Figure 13.3.5). When they are 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 the bearing freezing. Careful maintenance of protective covers and skirts is necessary when attempting to seal this bearing, which is typically unsuccessful (see Figure 13.3.6).

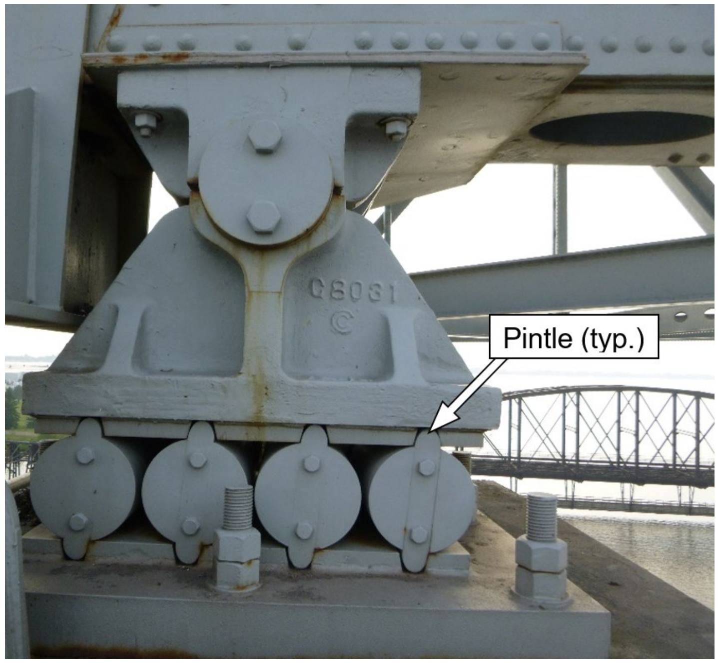


Figure 13.3.5 Roller Nest Bearing with Pintles

Figure 13.3.6 Failed Roller Nest Skirt

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 necessary (see Figure 13.3.7). The movement is accommodated through the rocking action of the bearing (see Figure 13.3.8). As with roller bearings, rocker bearings come in different forms, such as segmental rockers, rocker nests, and pinned rockers. To accommodate longitudinal movement, slotted holes may exist in the radius portion of the bearing or pintles are also used in some rockers (see Figure 13.3.11).

Figure 13.3.7 Rocker Bearing

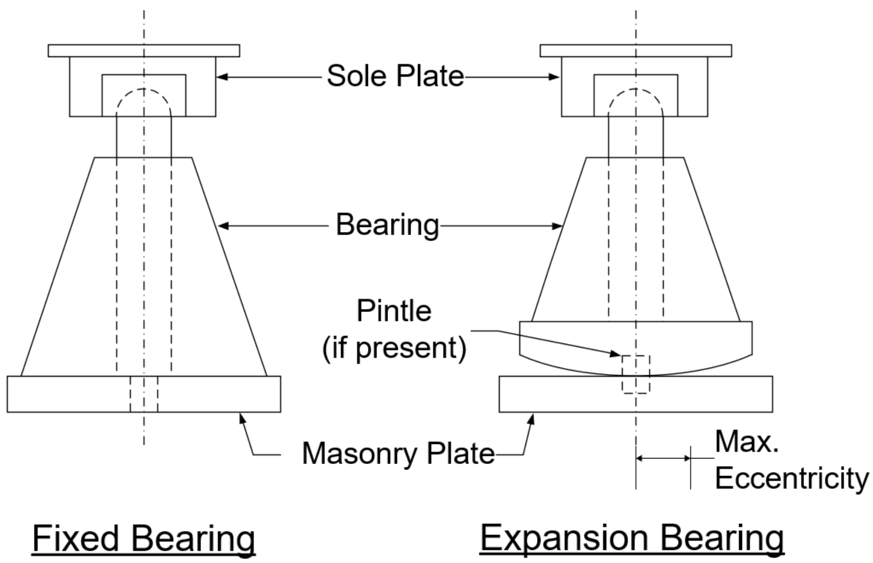


Figure 13.3.8 Typical Fixed and Expansion Rocker Bearing Details

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 can become very heavy and difficult to handle. Since only a small portion of the roller bearing is in contact with the sole plate and masonry plate, the unused portion may be cut away and a substantial weight savings obtained (see Figure 13.3.9).

Figure 13.3.9 Segmental Rocker Bearing

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

Rocker Nest Bearings

When several rockers are grouped, or combined, they form a rocker nest bearing (see Figure 13.3.10). 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 frozen bearing.

Figure 13.3.10 Rocker Nest Bearing

Pinned Rocker Bearings

The pinned rocker has been widely used historically and are still present on older bridges. The top is basically a large pin and helps to keep the bearing aligned correctly. Longitudinal movement is provided by the rotation enabled by the pin and the rolling provided by the rocker. When exposed to adverse environmental conditions, however, the pin can corrode and freeze. Pinned rocker bearings can be quite large and are commonly used for relatively long spans and heavy loads.

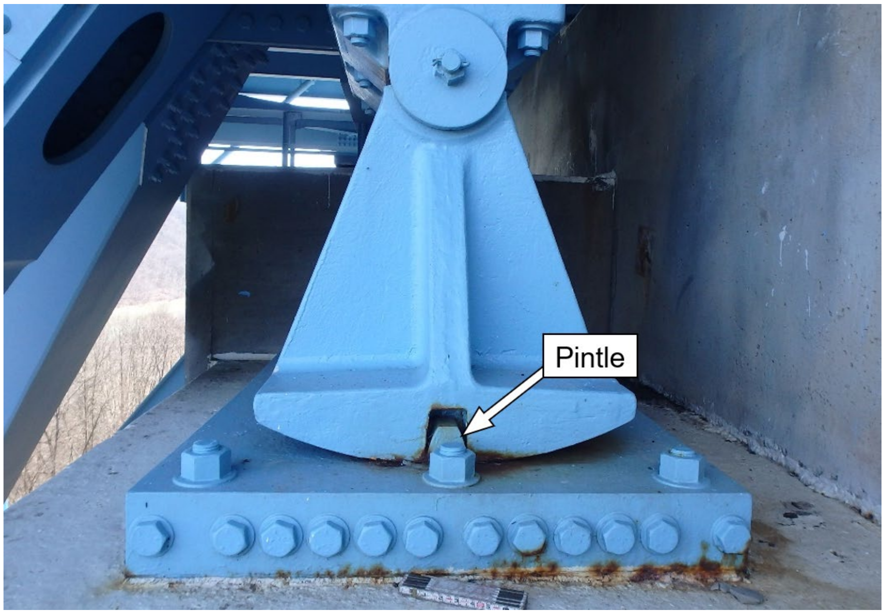


Figure 13.3.11 Pinned Rocker Bearing

13.3.2 Elastomeric Bearings

Elastomeric bearings include both plain and laminated neoprene pads. Neoprene is a heavy synthetic rubber-like material that deforms slightly under compression or shear. Elastomeric bearings are the most common bearings for new construction, particularly when the loads are relatively small, based on cost effectiveness and performance.

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 13.3.12). Neoprene bearings are popular for steel beam bridges as well.

Figure 13.3.12 Plain Neoprene Bearing Pad

Expansion and contraction are typically achieved through a shearing deformation of the neoprene (see Figure 13.3.13). These bearings are typically of uniform thickness, but rotation of the superstructure can result in differential thickness (see Figure 13.3.14).

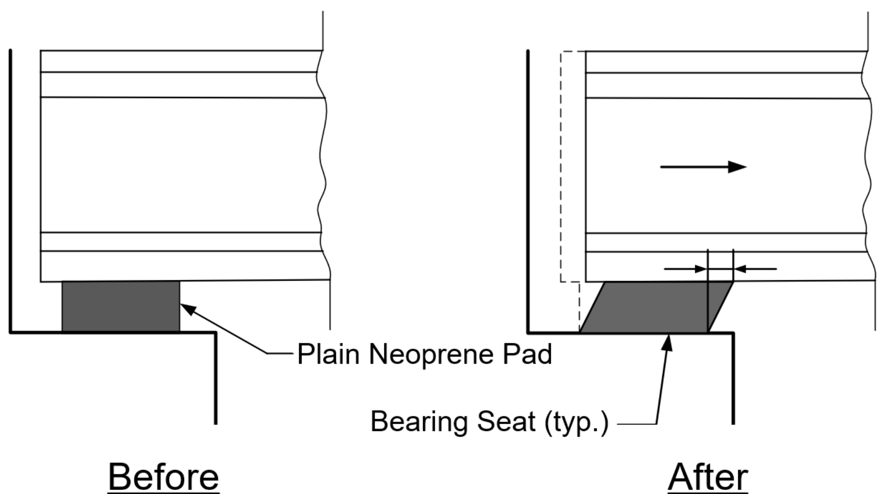


Figure 13.3.13 Shear Deformation of a Plain Neoprene Bearing

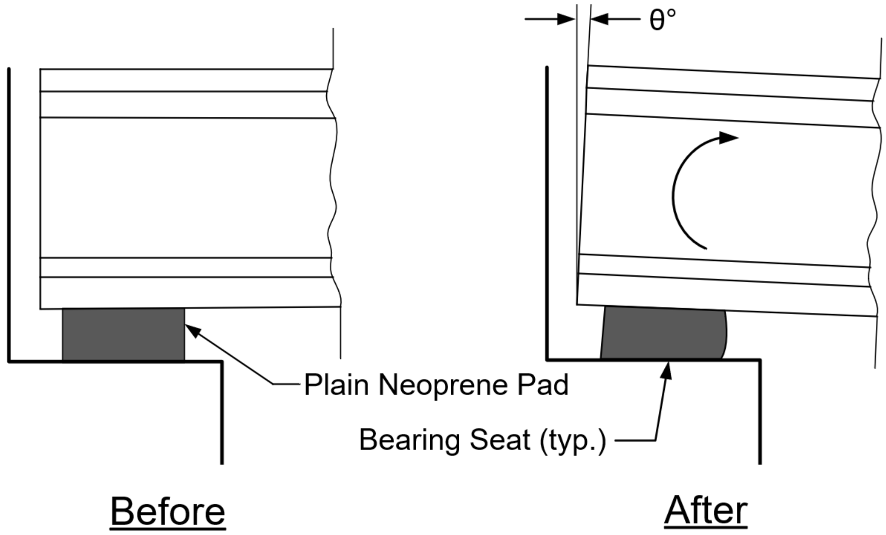


Figure 13.3.14 Rotational Deformation of a Plain Neoprene Bearing

Various mechanisms can be 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. Sometimes holes are formed into the bottom of the neoprene pads and into box beams when they are fabricated, and these bearing holes are placed over the anchor bolts or dowels to help eliminate walking out of position.

Laminated Neoprene Pads

A laminated neoprene bearing is simply a stack of neoprene pads reinforced with thin steel or fiberglass plates (see Figure 13.3.15). The plates are not visible as they are encased in neoprene (see Figure 13.3.16). Laminated bearing pads are generally used on longer structures where the expansion and contraction demands 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 can dramatically decrease its useful life. The laminated bearing eliminates this excess bulging and permits expansion and contraction without excessive wear.

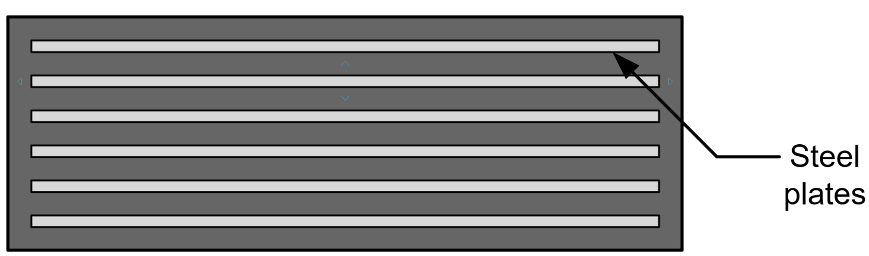


Figure 13.3.15 Cross Section: Laminated Neoprene Bearing Pad

Figure 13.3.16 Laminated Neoprene Bearing Pad

13.3.3 Pot Bearings

Pot bearings are a type of High-Load Multi-Rotational (HLMR) bearing, providing for greater loadings, as well as accommodating the multi-dimensional rotations of a structure.

Neoprene Pot Bearings

A neoprene pot bearing (see Figure 13.3.17) 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 that rests on a neoprene or elastomeric pad, enabling 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.

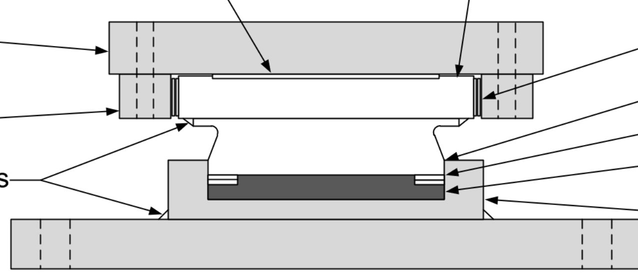


Figure 13.3.17 Guided Neoprene Pot Bearing

Higher loadings can be accommodated since the pot contains the elastomeric pad. Guide bars in the pot bearing restrict transverse movement (see Figure 13.3.18). A fixed bearing version of this configuration generally does not possess the stainless steel plate or the PTFE disk.

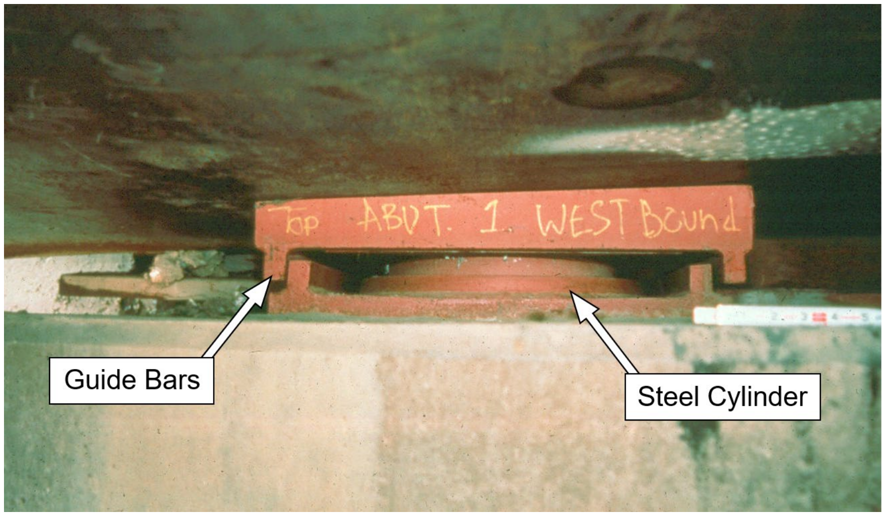


Figure 13.3.18 Neoprene Pot Bearing with Guide Bars

Spherical Pot Bearings

Spherical bearings provide multi-directional rotation. They are similar to neoprene pot bearings, except that the polytetrafluoroethylene (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 provides 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 13.3.19) or rectangular in shape. Beneath the pot is the masonry plate that anchors the bearing to the substructure unit.

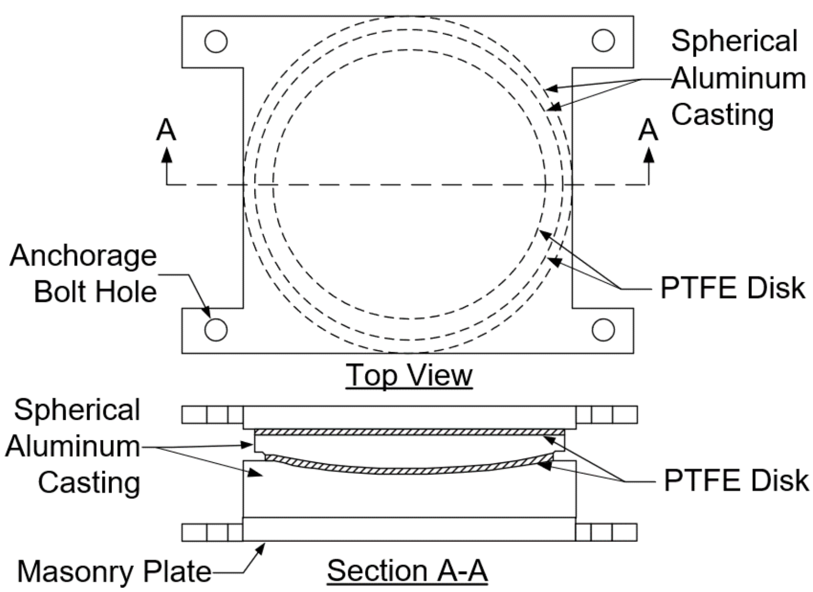


Figure 13.3.19 Spherical Pot Bearing

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

13.3.4 Disk Bearings

Disk bearings are also considered HLMR bearings. 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 13.3.20).

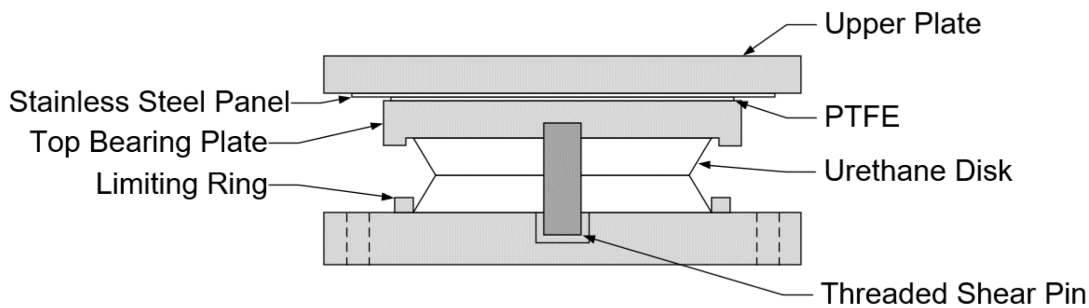


Figure 13.3.20 Disk Bearing Cross Section

Disk bearings typically have a very low profile (see Figure 13.3.21). They 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 13.3.21 Disk Bearing

13.3.5 Enclosed or Concealed Bearings

For some bearings, the line-of-sight between the inspector and the bearing may be limited. These bearings are said to be enclosed or concealed bearings and cannot be adequately evaluated through a visual inspection (see Figure 13.3.22). Examples of bearings that may be considered enclosed or concealed include bridges with integral end diaphragms.

Figure 13.3.22 Enclosed or Concealed Bearing

It is typically important to note the difference between a bridge with concealed bearings and a bridge with integral abutments, which has no bearings. Check the plans if there is a bearing that cannot be viewed.

13.3.6 Fixed Bearings

Fixed bearings (see Figure 13.3.23) are classified as only providing rotational movement. They rely on the rotation around the pins to accommodate end rotation. Fixed bearings prevent longitudinal and transverse movement.

Figure 13.3.23 Fixed Bearing

Figure 13.3.24 shows a sketch of a fixed bearing. As with the movable 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 longitudinal and transverse 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.

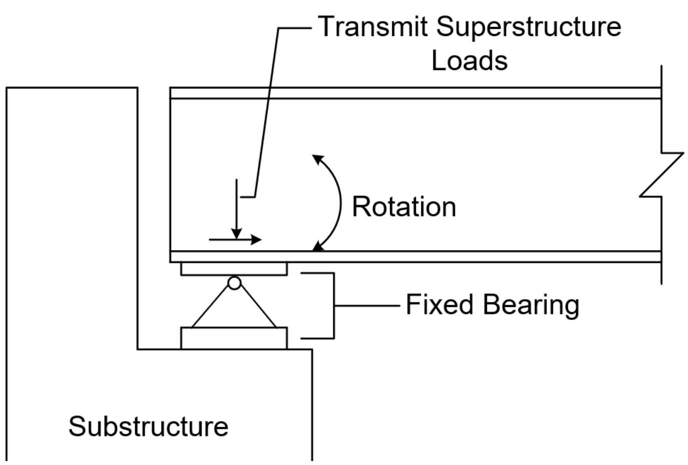


Figure 13.3.24 Fixed Bearing Schematic

13.3.7 Other Bearings

Other bearings are not a specific bearing element type defined by AASHTO. Instead, other bearings represent any types not included under the listed bearing types (movable, expansion, pot, disk, fixed bearings, or enclosed/concealed). Specific types of bearings that may qualify as other may be those that are still in service but may no longer be utilized in modern bridge construction. These bearings include unique bearings utilized by various bridge owning agencies.

Pin and Link Bearings

Pin and link bearings are 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. This bearing type consists of two vertically oriented steel plates pinned at the top and bottom to provide longitudinal movement (see Figure 13.3.25). A disadvantage of this type of bearing is that, as the superstructure expands and contracts, the deck rises and falls slightly. Another disadvantage is that pins can fracture when frozen by corrosion.

Figure 13.3.25 Pin and Link Bearing

Restraining Bearings

Restraining bearings serve to hold a bridge down in the case of uplift. Uplift usually occurs on cantilever anchor spans. The devices used to resist uplift can be as simple as long bolts running through the bearings on short span bridges or as complex as chains of eyebars on larger structures (see Figure 13.3.26). Lock nuts are typically used with bolted restraining devices to resist uplift. Pin and link members can also be considered restraining devices. The type of restraining device used depends on the magnitude of the uplift force.

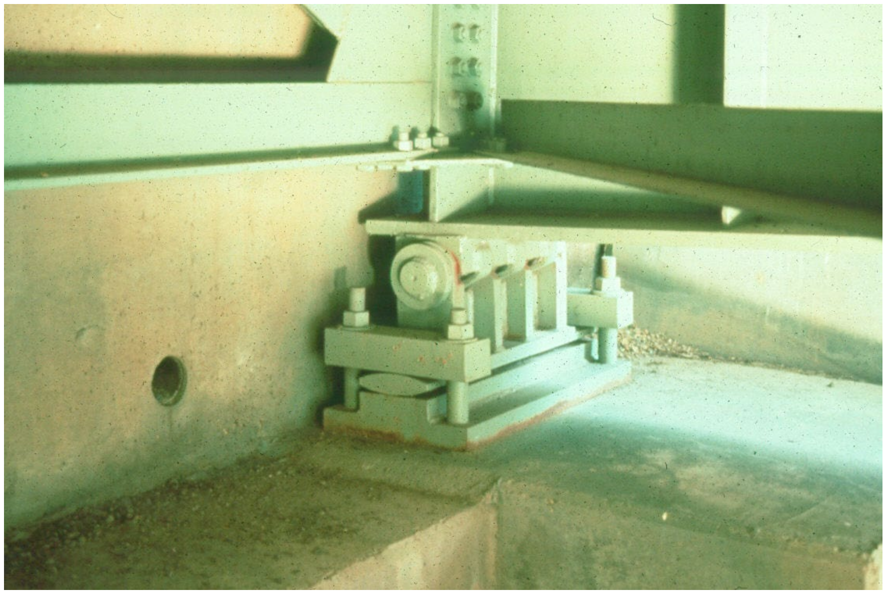


Figure 13.3.26 Restraining Bearing

Isolation Bearings

Isolation bearings were developed to protect structures against extreme longitudinal and transverse loadings such as earthquakes. Isolation bearings may also be used to accommodate longitudinal movements due to large truck loadings. These bearings operate by enabling larger than normal relative movement, which reduces lateral loads applied to the structure. Types of isolation bearings include lead core, friction pendulum, and high-damping rubber.

Lead Core Bearings

Lead core bearings are a type of isolation bearing. These bearings are typically similar to laminated neoprene bearings in that they are a sandwich of neoprene and steel plates (see Figure 13.3.27 and Figure 13.3.28).

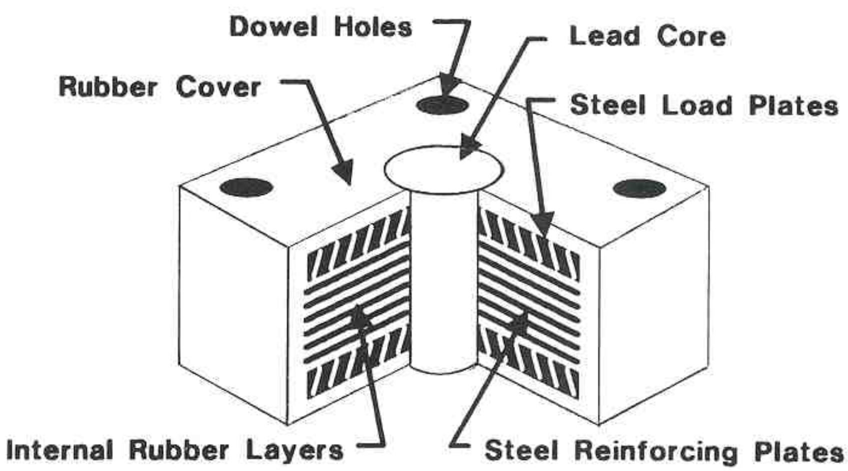


Figure 13.3.27 Sketch of a Lead Core Isolation Bearing

Figure 13.3.28 Lead Core Isolation Bearing

These bearings contain a lead core that stiffens the bearing to help resist the effects of high longitudinal and transverse bridge loading. During seismic loads, the lead core is designed to yield, thereby making the bearing more flexible and enabling it to isolate the bridge from the effects of earthquake motion. The downside to lead core bearings is the possibility of needing 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.

Friction Pendulum Bearings

Another bearing type designed to protect against extensive damage from loads such as earthquakes loading is a friction pendulum bearing. These bearings are generally designed to reduce lateral loads and shaking movements transmitted to the structure (see Figure 13.3.29). They can protect structures and their contents during strong, high magnitude earthquakes and can operate near fault pulses and deep soil sites.

Figure 13.3.29 Friction Pendulum Bearing

Friction pendulum bearings incorporate the characteristics of a pendulum to lengthen the natural period of the isolated structure to avoid the strongest earthquake forces (see Figure 13.3.30). The period of the bearing is selected by choosing the radius of curvature of the concave surface. It is typically independent of the loads of the superstructure. Torsion motions of the substructure can be minimized because the center of stiffness of the bearings automatically coincides with the center of mass of the superstructure.

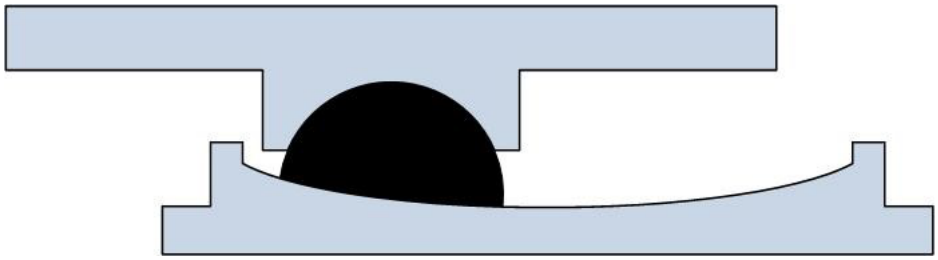


Figure 13.3.30 Schematic of a Friction Pendulum Bearing

High-Damping Rubber 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 high load events, such as earthquakes, a special hysteretic rubber compound in the bearing dissipates the energy. As a result, the structure is isolated from the shaking forces of the earthquake and is less likely to collapse.

Section 13.4 Overview of Common Deficiencies

13.4.1 Steel Deficiencies

Common potential deficiencies that may occur in steel bearings include:

- Corrosion.
- Fatigue cracking.
- Overloads.
- Heat/fire damage.
- Coating failures.
- Loose, missing, or deteriorated fasteners (connections).
- Out-of-plane distortion (including buckling).
- Excessive rotation.
- Misalignment.
- Movement.
- Loss of bearing contact.

Refer to Chapter 7 for a detailed explanation of the properties of steel, types and causes of steel deficiencies.

13.4.2 Neoprene Deficiencies

Common potential deficiencies that may occur in neoprene bearings include:

- Bulging.
- Splitting.
- Tearing.
- Misalignment.
- Movement.
- Loss of bearing contact.

Section 13.5 Inspection Methods

13.5.1 Visual and Physical Inspection

Most deficiencies in bearings are first detected by a visual assessment. In order for this to occur, a hands-on inspection, or inspection where the inspection team is close enough to touch the area being evaluated, is necessary. More exact visual observations can also be employed using a magnifying unit after cleaning the suspect area. A mirror or a small endoscope may also be used to visually access the area behind the bearing.

Physical inspection methods may also supplement visual inspection. These methods are dependent upon the type of material and may include the use of a hammer, wire brush, grinder, or sand blaster remove any loose or flaked material. Be aware of any necessary precautions regarding health and environmental issues when inspecting structures with lead-based paint. Degreasing spray may also be used to help remove paint and reveal a potential deficiency. Measurement of the deficiencies to check irregular dimensions, deformations, or section loss is also considered a physical inspection method.

13.5.2 Advanced Inspection

If the extent of the steel deficiency on a bearing cannot be determined by the visual and/or physical inspection methods described above, advanced methods may be used.

There are several advanced methods for steel bearing inspection that may be performed in the field. Typical methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Acoustic Emission (AE).
- Crack Propagation Gage (CPG).
- Dye Penetrant Testing (PT).
- Eddy Current Testing (ET).
- Magnetic Particle Testing (MT).
- Magnetic Flux Leakage (MFL).
- Ultrasonic Testing (UT) (see Figure 13.5.1).
- Phased Array Ultrasonic Testing (PAUT).

Figure 13.5.1 Ultrasonic Testing Inspection of a Pin in a Bearing

Other methods that may be considered destructive testing methods include the following.

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

Advanced methods are described in detail in Chapter 17.

Section 13.6 Inspection Areas and Deficiencies

For routine inspections, all bridge bearings should be accessed and observed visually or via another method. When deficiencies are suspected, or the full severity or extent of deficiencies cannot be determined using routine methods, then an In-depth or Special inspection should be completed using hands-on methods.

When inspecting a bearing, the inspection team should first determine if the bearing was initially intended to be fixed or movable. If the bearing was designed to provide for longitudinal and transverse translation or movement of the superstructure, then it is a movable bearing; if not, then it is a fixed bearing. The inspection team should refer to the design plans if available. It is important that the inspection team assess whether movable bearings still provide intended movement. Bearings should be examined for uplift; there should be solid contact between the bearings and superstructure and between the bearings and the substructure.

Bearings demand suitable support. A distance of several inches should exist between the edge of the masonry plate and the edge of the supporting member, abutment, or pier. Any loss to the supporting member near the bearing should be noted (e.g., spalling of a concrete bridge seat) (see Figure 13.6.1).

Small maintenance problems with bearings can grow progressively worse if ignored, eventually causing a break in the load path. Inoperable bearings can transfer significant overstress to the superstructure or substructure. Visual, physical, or advanced inspection methods may be necessary to evaluate the following inspection areas and deficiencies. This section describes common problems related to specific areas and details to authenticity inspect.

13.6.1 Movement and Alignment

Inspectors should check that bearings are properly aligned in the expected movement direction (at temperature above the average design temperature (approximately 68 °F) the movable bearings align away from the fixed bearings and vice versa) with the bearing surfaces clean and in full contact with each other. Misalignments in the bridge railings or expansion joints may be an indication of bearings that are misaligned. If only partial contact is made, damage can occur to the bearing device, superstructure, or substructure. This damage can occur when the superstructure or substructure has moved longitudinally 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 of the support and results in a higher stress that could overstress the bridge seat (see Figure 13.6.1).

Figure 13.6.1 Spalling of Concrete Bridge Seat Due to High Edge Stress

Also, such redistribution of the load may cause buckling to occur in 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 permitting the anticipated longitudinal movement of the superstructure. It may also signify movement in the substructure.

The inspection team should record the temperature during the inspection. Normal air temperature thermometers or special thermometers with magnets are available to measure the actual temperature of the superstructure and bearing surface. The movement of the bearings should be measured and compared to the recorded temperature. The bearings should be in the expanded position (away from the fixed joint) 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 typically 68 degrees Fahrenheit unless otherwise noted.

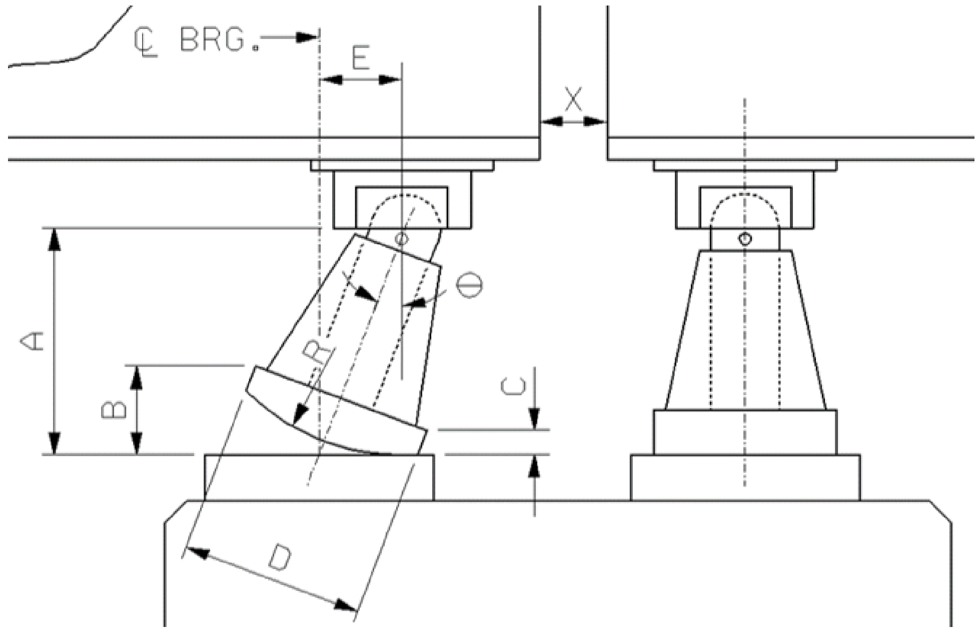
13.6.2 Steel Bearings

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. Most other materials are non-corrosive or corrosion resistant. The following discussions concentrate the inspection of steel bearings.

Steel bearing elements should be checked for any material deficiencies, pitting, section loss, deterioration, and debris build-up, which can cause the bearing to bind up or freeze (see Figure 13.6.2). Evidence of a frozen bearing includes bending, buckling, improper alignment of members, or cracks in the bearing seat. Inspectors should check for corroded, bent, broken or missing anchor bolts (see Figure 13.3.1).

Figure 13.6.2 Heavy Corrosion on a Steel Rocker Bearing

Bearing tilt should be verified to be in the proper direction based on the temperature at the time of the inspection. There are several common measurements that are verified during rocker bearing inspections (see Figure 13.6.3).



Legend:

Dimension A – rocker height
Dimension B – high corner (field entry)
Dimension C – low corner (field entry)
Dimension D – rocker plate width
Dimension E – translation (field entry)
Dimension R – radius
Dimension X – clear distance (field entry)
Angle theta (θ) – angle of tilt (field entry)

Figure 13.6.3 Rocker Bearing Inspection Measurements

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:

- Settlement or movement of the bearing support away from the portion of the bridge being supported.
- Excessive deflection or vibration in the bridge.
- Bent, corroded, loose, missing, or broken fasteners that are used to attach the bearing to the superstructure or the substructure (see Figure 13.6.4).
- Worn bearing elements.
- Uplift in continuous bridge superstructures.
- Excessive earth pressure on abutments, which tilts the backwall into the beams.

Figure 13.6.4 Bent Anchor Bolt due to Excessive Movement

Proper alignment and adequate movement of bearing can be determined by performing the following check. 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 13.6.5). 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.

Inspectors should check to verify that bearing devices are supported directly by the masonry plate (see Figure 13.6.5). Bearings employing bronze sliding plates alignment with steel masonry plates should be examined, particularly on bridges exposed to a salt air environment, for signs of galvanic corrosion between the dissimilar metals bronze and steel. Also, inspectors should be aware of the bronze plate walking out of place in the longitudinal direction.

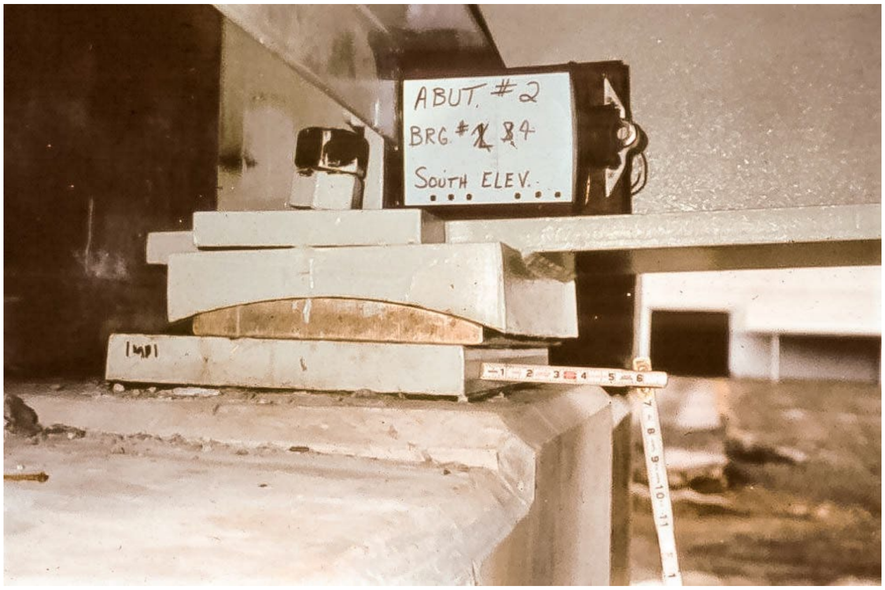


Figure 13.6.5 Longitudinal Misalignment in Bronze Sliding Plate Bearing

The position of the roller or roller nests should be examined to observe if the pintles are exposed or missing. Such conditions may indicate excessive superstructure expansion or contraction movement or undesirable substructure movement. See Figure 13.6.6 for an example of a damaged roller nest bearing.

Figure 13.6.6 Damaged Roller Nest Bearing

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.

Rockers should be inspected for proper tilt (see Figure 13.6.7). They should be tilted in accordance with the recorded temperature and they should appear stable. If rocker bearings are tilted in the direction inconsistent with the temperature, they may be frozen due to excessive corrosion or ratcheting (see Figure 13.6.8). Ratcheting occurs when a build-up of debris or rust accumulates under the rocker and does not enable it to rotate back into the proper position. Instead, the bearing center is pushed away from the center of the masonry plate or bearing pedestal.

Figure 13.6.7 Excessive Tilt in a Segmental Rocker

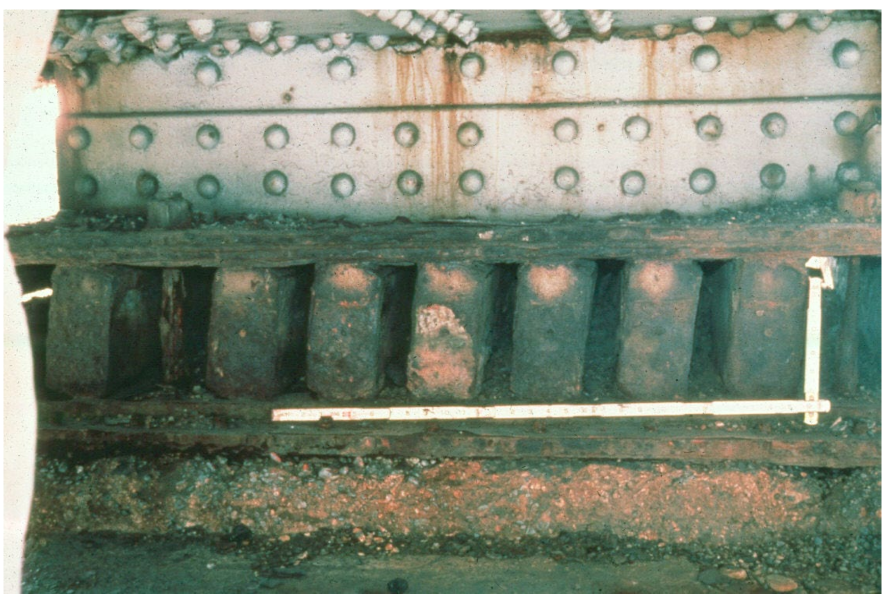


Figure 13.6.8 Frozen Rocker Nest

In extreme circumstances, a rocker bearing, or the entire line of bearings may overturn due to overextension or possible substructure movement (see Figure 13.6.9). In this case, there is generally severe misalignment of the deck between spans (see Figure 13.6.10). Inspectors should check for inadequate superstructure support from the substructure cap or bearing area to ensure collapse is not immanent. Also, the superstructure should be examined for stresses introduced due to the failure of the bearing assembly. The substructure should also be inspected for any damage that may have occurred.

Figure 13.6.9 Overturned Rocker Bearing

Figure 13.6.10 Dropped Span and Loss of Bearing Area due to Rocker Bearing Failure

Although not typically necessary, the pot bearing rotation should be measured if it appears to be excessive. The top and bottom plates of a pot bearing are usually designed to be parallel when 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 13.6.11:

$$\text{Rotation in degrees} = \tan^{-1}[(\text{height}_1 - \text{height}_2)/\text{plate length}]$$

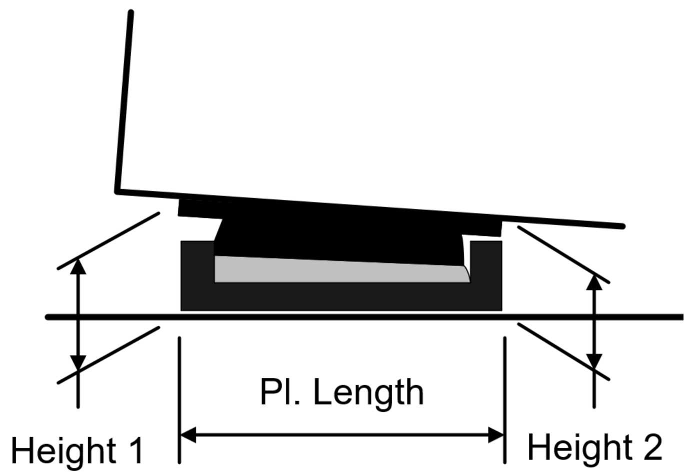


Figure 13.6.11 Tilt on a Pot Bearing

Since the pot bearing provides multidirectional rotation, the inspection team should check rotation along both sides of the bearing.

Pot bearings should be examined for proper seating of the various parts with respect to one another. If accessible, the inspection team should check that the neoprene pad is properly seated within the pot and that the top plate is situated properly over the part below. It should be determined if the neoprene is being extruded from the pot. It may be possible for elastomeric material to liquify under high pressure and therefore leak. The inspection team should check for failure of the seal rings. Guide bars should be inspected for proper alignment (pot should be equidistant from the guide bars), wear, binding, cracking, and deterioration.

Welds should be examined for any cracks and for any separation between the PTFE and the steel surface to which it is bonded. Although they are usually hidden from view, the inspection team should check any exposed portions of the neoprene aspects for splitting or tearing. Inspectors should look for any buildup of dirt and debris in and around the bearing that could affect the smooth operation of the bearing.

Inspection of pin and link bearings are essentially the same as that described for pins and hangers in Chapter 10. The amount of corrosion and ability of the connection to move freely is of critical concern, especially for suspended span bridges.

Inspection of restraining bearings and pin and link bearings for the condition of the main tension elements (i.e., hanger plates, eyebars, and anchor rods or bolts) and pins is the primary concern.

13.6.3 Elastomeric Bearings

Items that are typically common to both steel bearings and elastomeric bearings are sole plates, masonry plates, and anchor bolts (when present). Only the elastomeric bearings, or items specific to them, are discussed in this section. Inspectors should check if elastomeric bearings are acting as designed to transmit longitudinal and vertical forces, as well as rotations (see Figure 13.6.12). Excessive bulging can lead to splitting of the neoprene pads. Racking of the structure can lead to delaminations between the layers.

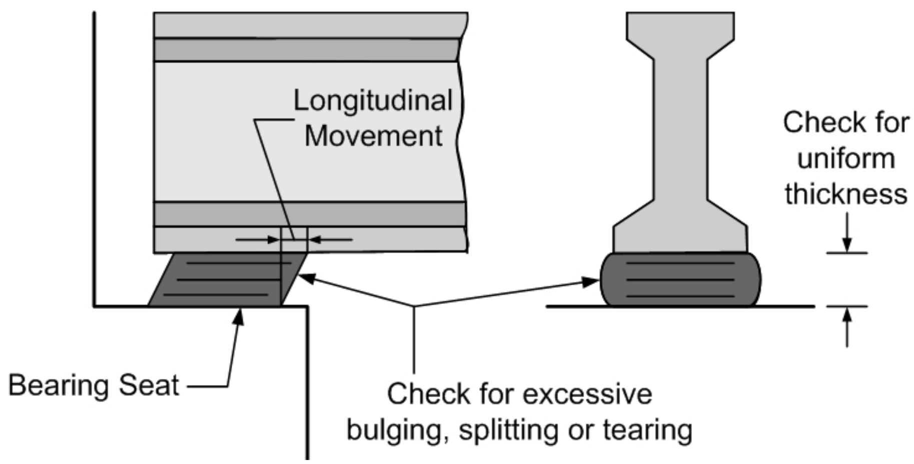


Figure 13.6.12 Elastomeric Bearing Inspection Checklist Items

Neoprene and Isolation Bearings

Neoprene bearing pads should be inspected for excessive bulging (approximately greater than 15 percent of thickness) (see Figure 13.6.13). Slight bulging in the sides of the pad can be expected. Although difficult to determine, the inspection team should check for excessive bulging for the height or thickness of the pad. As expansion and contraction of the structure takes place, the bulge tends to roll on the beam or bridge seat.

Figure 13.6.13 Neoprene Bearing Pad Excessive Bulging and Splitting

The bearing pad should be inspected for any splitting or tearing (see Figure 13.6.14). Close observation should be given to laminated neoprene bearings. Improper manufacturing can sometimes cause a failure in the area where the neoprene and interior steel shims are bonded.

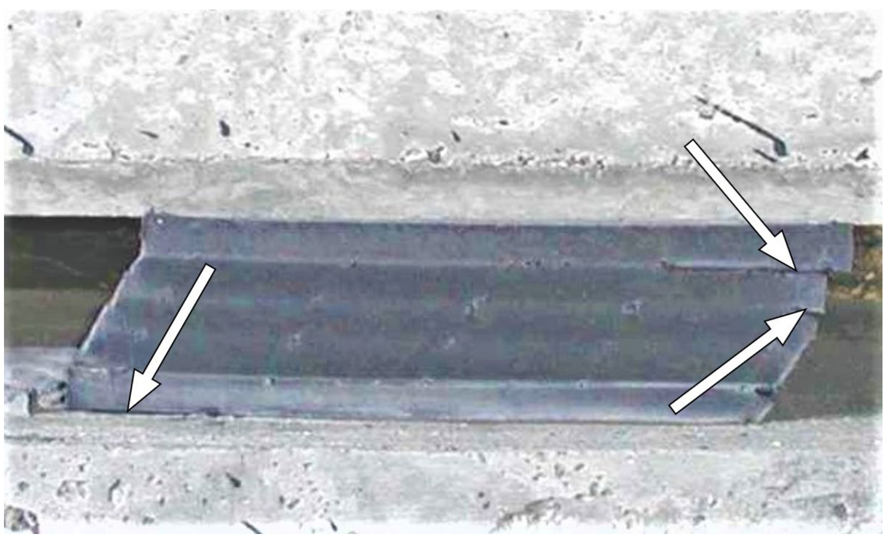


Figure 13.6.14 Loss of Contact with Substructure and Debonded Laminations

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

The inspection team should examine a plain (unlaminated) pad for any apparent growth in the length of the pad at the masonry plate. This growth indicates excessive strain in the pad. If this condition persists, the pad eventually experiences a shearing failure. Pad growth is not usually a problem with laminated bearings.

Close observation should be given to the area where the pad is bonded to the sole plates, masonry plates, or the substructure. This is where a neoprene bearing frequently fails. Sometimes the pad tends to de-bond and gradually move or "walk" out from under the beam or girder (see Figure 13.6.15). Excessive tilt or debonding from the bearing seat can lead to cracking or spalling of the bearing seat due to pressure points, or failure of the pad itself.

Figure 13.6.15 Plain Bearing Pads Walking Out from Under Beams

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 feature 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 13.6.16). 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 should be closely inspected. It may be necessary to replace the entire lead core bearing assembly after an earthquake if plastic deformation is observed. It is generally accepted that if the outside of the bearing shows deformation, the lead core has been compromised.

Figure 13.6.16 Lead Core Isolation Bearing

13.6.4 Enclosed and Concealed Bearings

It is typically difficult to determine if concealed or enclosed bearings are functioning as intended, as they cannot be visually seen. The plans should be reviewed before inspection to gain understanding of the method in which the bearing is connected to the superstructure, the orientation of any piles, and the presence and expected behavior of pavement relief joints.

Abnormal cracking in the substructure, particularly the bearing area or the superstructure, can be an indication that the bearing is not operating properly. Unexpected distortion in the superstructure or deck can indicate problems with the bearing. Unconventional movement may produce excessive unconsolidated soil in front of the abutment.

Section 13.7 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of highway bridges. The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI) for* **component condition** *rating* (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

According to the *SNBI*, FHWA now considers bearings to be bridge components. Bridge bearings (*SNBI* Item B.C.07) receive their own condition rating. The condition of protective coatings and other protective systems are not to be considered when rating bearings unless problems with the coating are indicative of problems with the underlying bearing material. The superstructure condition rating is only affected when serious bearing conditions exist that may cause distress, or local failures for the supported primary load-carrying members.

In accordance with the *AASHTO MBEI*, bearings are considered to be National Bridge Elements. The condition of protective coating for steel bearings is to be considered when rating elements using Element 515.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 14 TABLE OF CONTENTS

CHAPTER 14 LIST OF FIGURES

This page intentionally left blank.

Chapter 14 Inspection and Evaluation of Substructures

Section 14.1 Introduction

The substructure includes the portions of the bridge below the bearings or below the springline of an arch. Its purpose is to transfer the loads from the superstructure to the foundation soil or rock. The substructure is generally made of abutments and portions of wingwalls, as well as piers and bents in the case of multiple-span bridges. Consider the condition of integral abutment wingwalls to the first construction or expansion joint. The walls of three-sided and four-sided rigid frame bridges are also considered substructures. For bridges that have substructures not visible for inspection, use appropriate visual condition indicators from the superstructure or surrounding foundation materials to determine the condition. This chapter outlines the characteristics and inspection methods of the different substructure types. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

Section 14.2 Characteristics

14.2.1 Abutments

Design Characteristics

An abutment is a substructure unit 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 at the ends of a bridge.

It is common for abutments to have wingwalls which retain the approach roadway fill or embankment. Refer to Section 14.2.2 for further details regarding wingwalls.

Abutment types can be classified based on their design and construction characteristics with respect to the approach roadway embankment. The most common abutment types include:

- Full height or cantilever (see Figure 14.2.1).
- Stub, semi-stub, or shelf type (see Figure 14.2.2).
- Open or spill-through type (see Figure 14.2.3).
- Integral (see Figure 14.2.4).
- Semi-integral (see Figure 14.2.5).

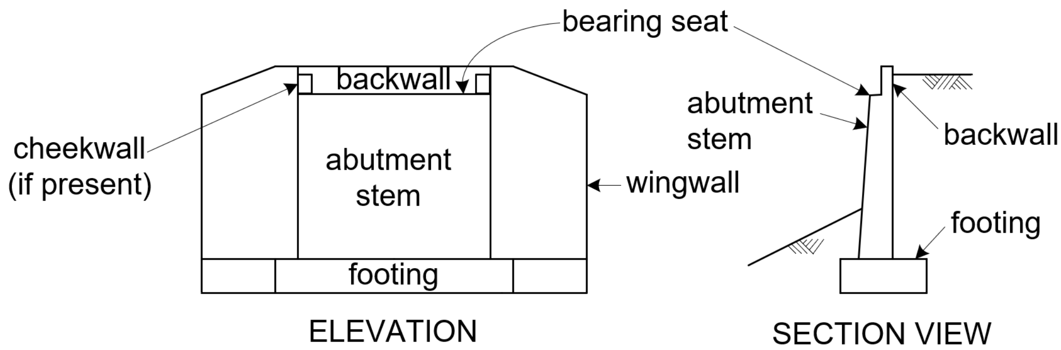


Figure 14.2.1 Sketch of Full Height Abutment

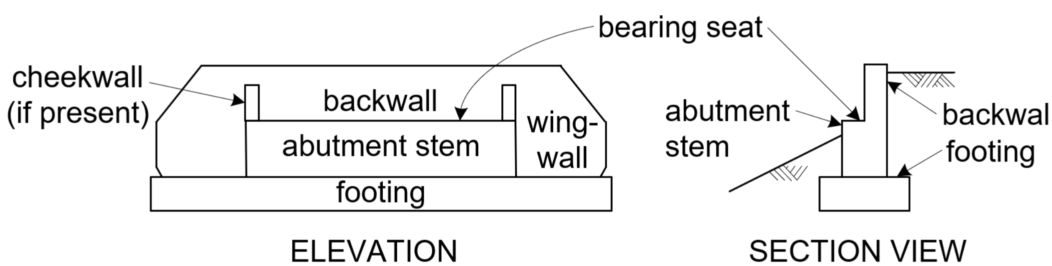


Figure 14.2.2 Sketch of Stub Abutment

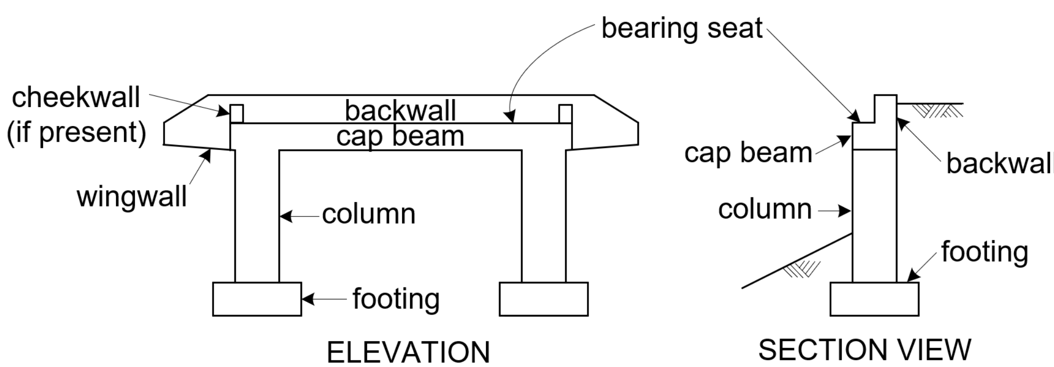


Figure 14.2.3 Sketch of Spill Through/Open Abutment

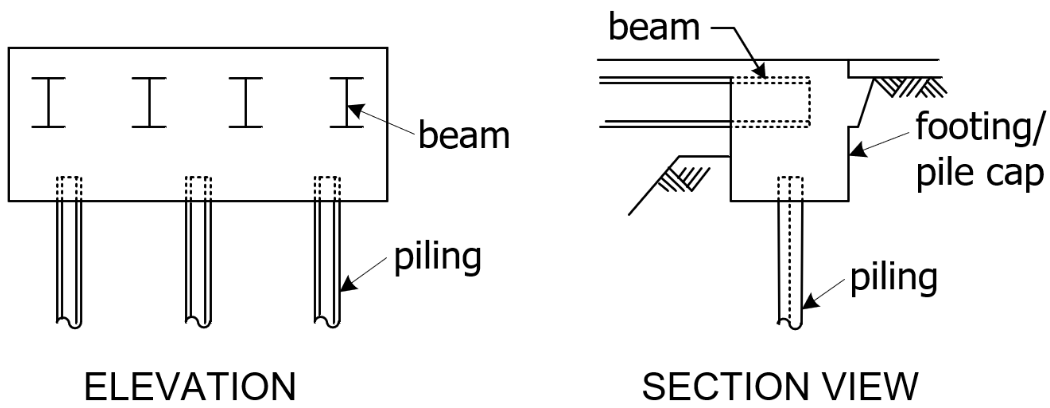


Figure 14.2.4 Sketch of Integral Abutment

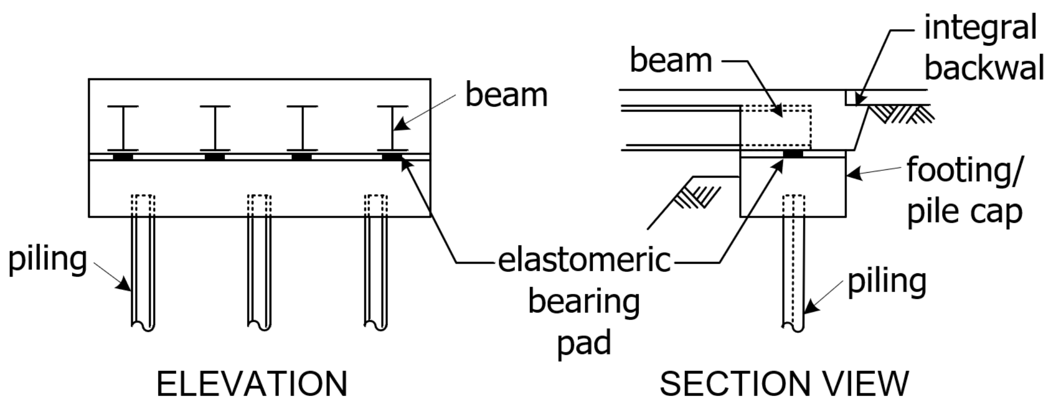


Figure 14.2.5 Sketch of Semi-Integral Abutment

Substructure units may have spread footings as shown or deep foundations, with piles or drilled shafts. Foundation types are presented in more detail in Section 14.2.4.

Additional less common abutment types include:

- Gravity.
- Counterfort.
- Pile bent with lagging.
- Crib.
- Cellular or vaulted.
- Reinforced soil.

Additional types of abutments with economic advantages used to support highway superstructures include:

- Mechanically Stabilized Earth (MSE) (see Figure 14.2.6).
- Geosynthetic Reinforced Soil (GRS) (see Figure 14.2.7).

Foundations typically consist of spread footings or deep foundations. Refer to Section 14.2.4 for a detailed description of abutment foundation types. The vertical face of an MSE wall is typically comprised of precast concrete panels.

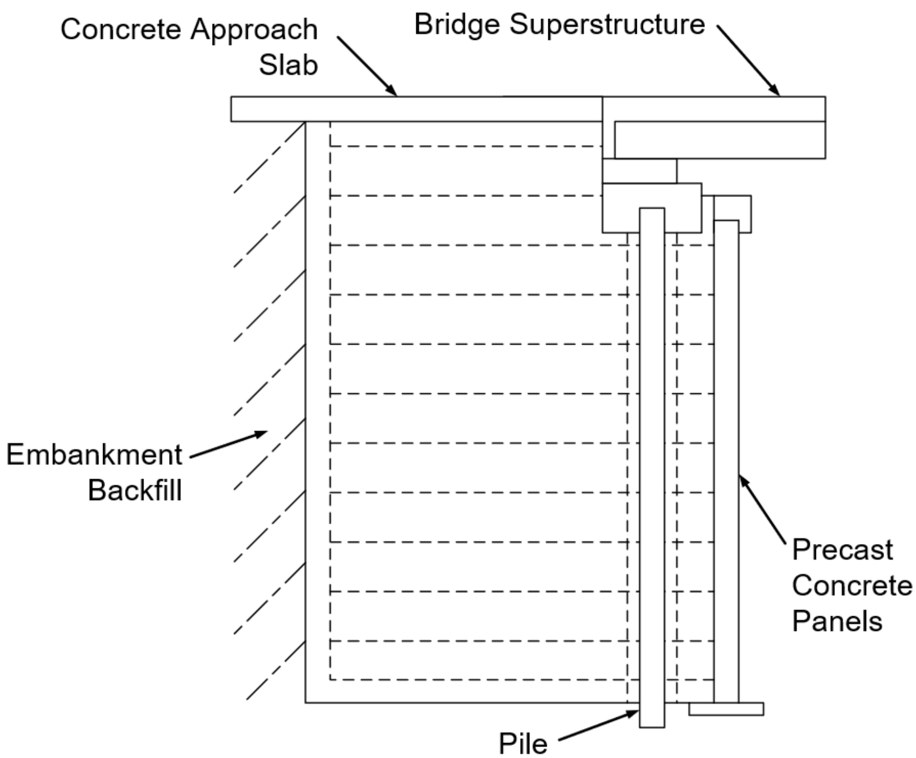


Figure 14.2.6 Cross Section - Mechanically Stabilized Earth Abutment

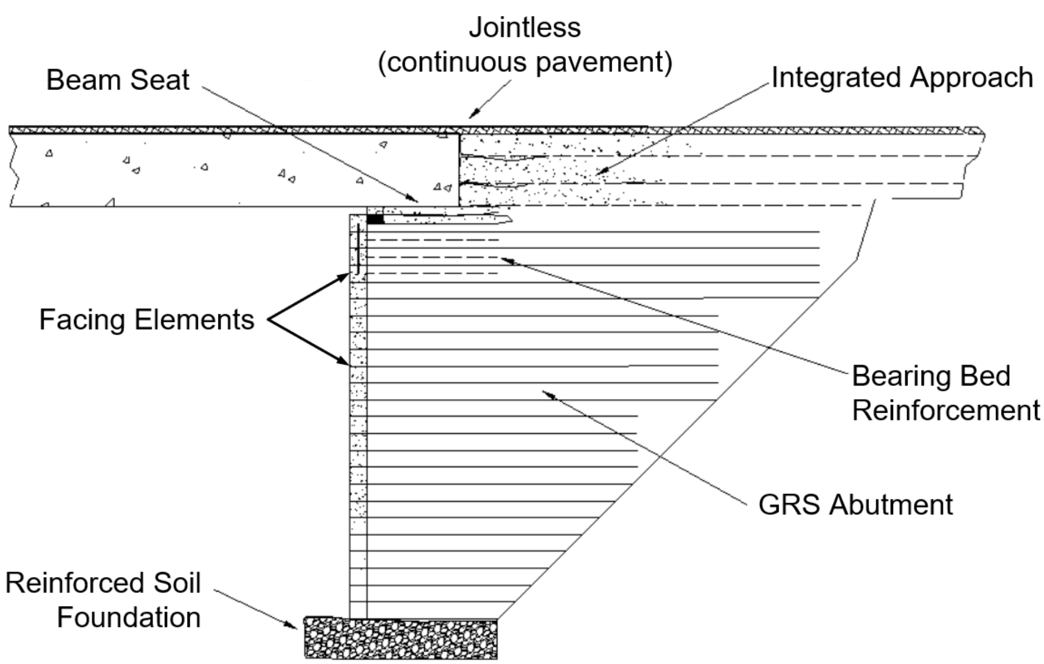


Figure 14.2.7 Cross Section - Geosynthetic Reinforced Soil Abutment

The primary materials used in abutment construction typically are unreinforced concrete, reinforced/prestressed concrete, stone masonry, steel (although not very common), timber, metallic or geosynthetic reinforcing strips, or a combination of these materials (see Figure 14.2.8 through Figure 14.2.12).

Figure 14.2.8 Plain Unreinforced Concrete Gravity Abutment

Figure 14.2.9 Reinforced Concrete Cantilever Abutment

Figure 14.2.10 Stone Masonry Gravity Abutment

Figure 14.2.11 Steel Pile Bent Abutment

Figure 14.2.12 Timber Pile Bent Abutment with Reinforced Concrete Cap

Full Height Abutments and Stub Abutments

Full height abutments are typically used when shorter spans are desired or if there are Right-of-Way restrictions or terrain issues (see Figure 14.2.13). 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 14.2.14). Longer spans are typically necessary when stub abutments are used. Using stub abutments reduces the cost of the substructure but increases the cost of the superstructure.

Figure 14.2.13 Full Height Abutment

Figure 14.2.14 Stub Abutment

Water can build up horizontal pressure behind an abutment, especially in full height abutments. Enabling the water to exit from behind the abutment relieves this pressure. Weep holes, when provided, are typically four inches in diameter and enable 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 generally sloped to drain the water out at the end of the abutment.

Spill-Through Abutments

Spill-through, or open, abutments tend to be 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 bearing seat and between (through) the supporting columns. Only the topmost few feet of the embankment are actually retained by the abutment cap (see Figure 14.2.15).

Spill through/open abutments can reduce construction costs, as the quantities of concrete and heavy reinforcement are greatly decreased as compared to an abutment with a typical stem. This substructure type provides for the conversion of the abutment to a pier if additional spans are added in the future.

Spill through/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. Riprap or slope paving can sometimes be used to counter these problems. This abutment type is not suitable adjacent to streams due to susceptibility to scour.

Figure 14.2.15 Spill Through/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 provide for thermal movements but can deteriorate quickly and create a wide range of maintenance demands 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 provide a solution to these problems.

In this design, the superstructure and substructure are integral and act as one unit without an expansion joint or bearing (see Figure 14.2.16). Relative movement of the abutment with respect to the backfill enables the structure to adjust to thermal expansions and contractions. Pavement joints at the ends of approach slabs are typically provided to accommodate the relative movement between the bridge and the approach roadway pavement.

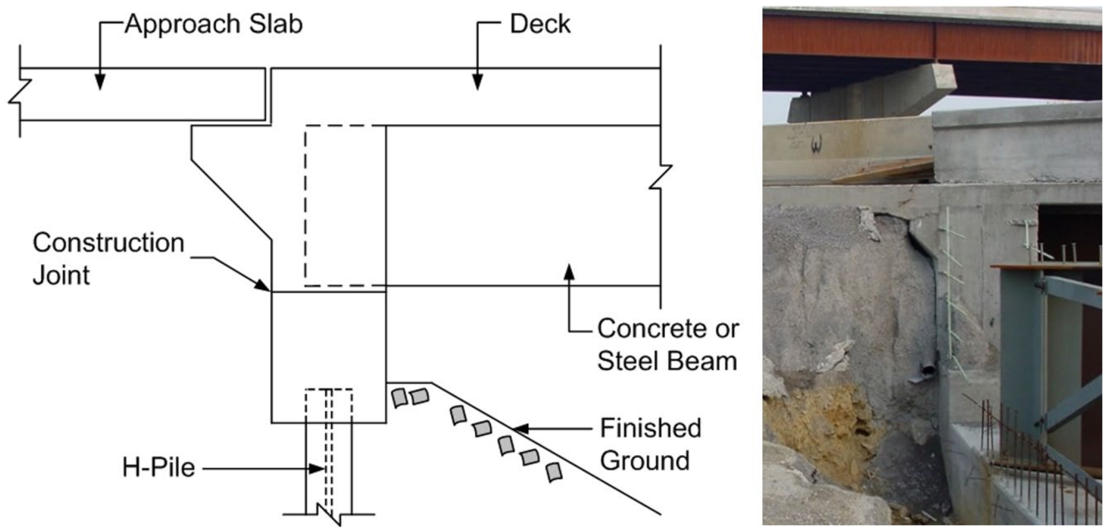


Figure 14.2.16 Integral Abutment Schematic

The advantage of the integral abutment is that it lacks bearing devices and joints to repair, or replace, or maintain (see Figure 14.2.17). There are two main disadvantages of integral abutments. The first issue is potential settlement of the roadway approach over time due to improper compaction of backfill. A second drawback is possible cracking of the abutment concrete, which could be due to restricted movement caused by over compaction of backfill or rotation in the superstructure due to heavy skews.

Figure 14.2.17 Integral Abutment

Semi-integral abutments may be similar to integral abutments, however, the superstructure and the top of the abutment act as one unit, but the bottom portion acts independently of the superstructure. This is achieved by a joint between the top and bottom portions of the abutment that provides for unrestrained rotation and thermal movement.

Mechanically Stabilized Earth Abutments

A Mechanically Stabilized Earth (MSE) abutment typically consists of precast concrete panels, soil reinforcing strips (flat strips or welded bar grids typically made of galvanized steel or geosynthetics), and backfill to create a structural unit for supporting the superstructure and approach roadway embankment (see Figure 14.2.18). The reinforced soil mass consists of select granular backfill. The tensile reinforcements and their connections may be proprietary, and may employ metallic (i.e., strip- or grid-type) or polymeric (i.e., sheet-, strip-, or grid-type) reinforcement (see Figure 14.2.19). The soil reinforcing strips hold the wall facing panels in position and provide reinforcement for the soil. Geotextiles may be used to cover the joint between the panels. Geotextiles are placed behind the precast panels to keep the soil from escaping through the joints. These geotextiles still enable excess water to flow out. Tie backs, if present, are typically used when lateral earth forces cannot be resisted by the footing alone. Tie backs are steel bars or strands grouted into the soil or rock behind the abutment stem.

Figure 14.2.18 Mechanically Stabilized Earth Abutment

Two MSE abutment design concepts have typically 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 are typically constructed by driving piles within steel shells and placing fill around the pile and provide vertical support for the bridge. The MSE wall provides lateral support for the approach roadway embankment.

Problems have occurred using the first concept when the MSE wall reinforced soil supports the bearings directly, since the MSE walls can bulge out when they support vertical superstructure loads. Most common MSE wall construction practices use stub abutments on piles behind the MSE wall.

A row of precast vertical concrete panels is 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 14.2.19). This process, which enables the wall to remain stable during construction, is repeated until the designed height is attained.

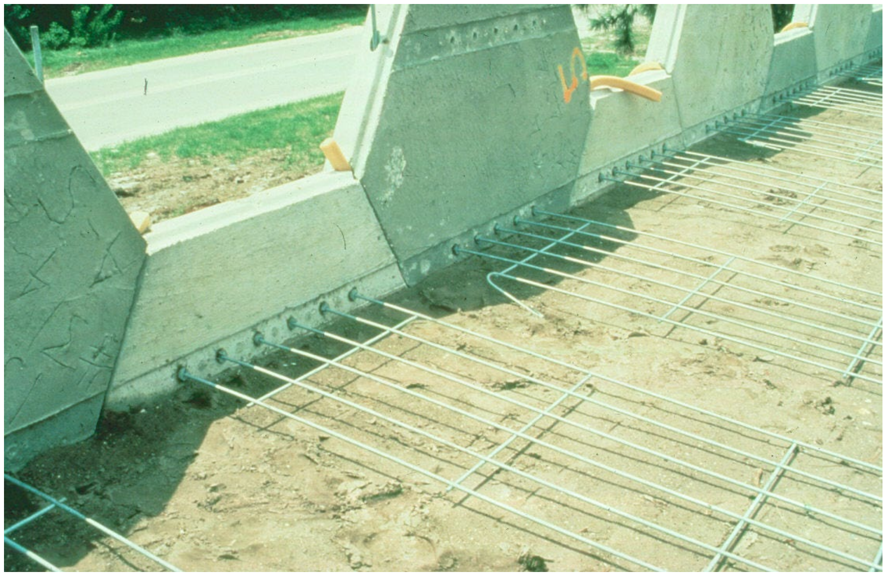


Figure 14.2.19 Mechanically Stabilized Earth Wall Under Construction

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 aesthetics when compared to a reinforced concrete full height abutment. Disadvantages can include difficulty in repairing failed soil reinforcement and the possible settlement. Reference *Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volume I & II, Publication No. FHWA-NHI-10-024/25* for a detailed description of these systems.

Geosynthetic Reinforced Soil Abutments

Another less common 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. Structural backfill 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 in union and obtain their strength through proper spacing of the layers of reinforcement. An advantage of GRS abutments is their simplicity to construct. GRS technology works well with simple overpasses; however, they are generally not desirable for sites where severe flooding or scour may occur (see Figure 14.2.20 and Figure 14.2.21).

Figure 14.2.20 GRS Bridge Abutment at the FHWA Highway Research Center

Figure 14.2.21 GRS Abutment

Reference *Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems, Publication No. FHWA-HRT-17-080* for a detailed description of these systems. The reinforced soil concepts, using metallic or geosynthetic reinforcement, are also commonly used as retaining walls or wing walls.

Other Abutments

Other abutment types primarily include gravity abutments for vehicle carrying structures. They are generally constructed of concrete (see Figure 14.2.22), dry stacked stone or stacked stone with mortar. Diaphragms or backwalls are placed on top of the abutment to help support the approach roadway embankment material. There are typically no piles under a gravity abutment.

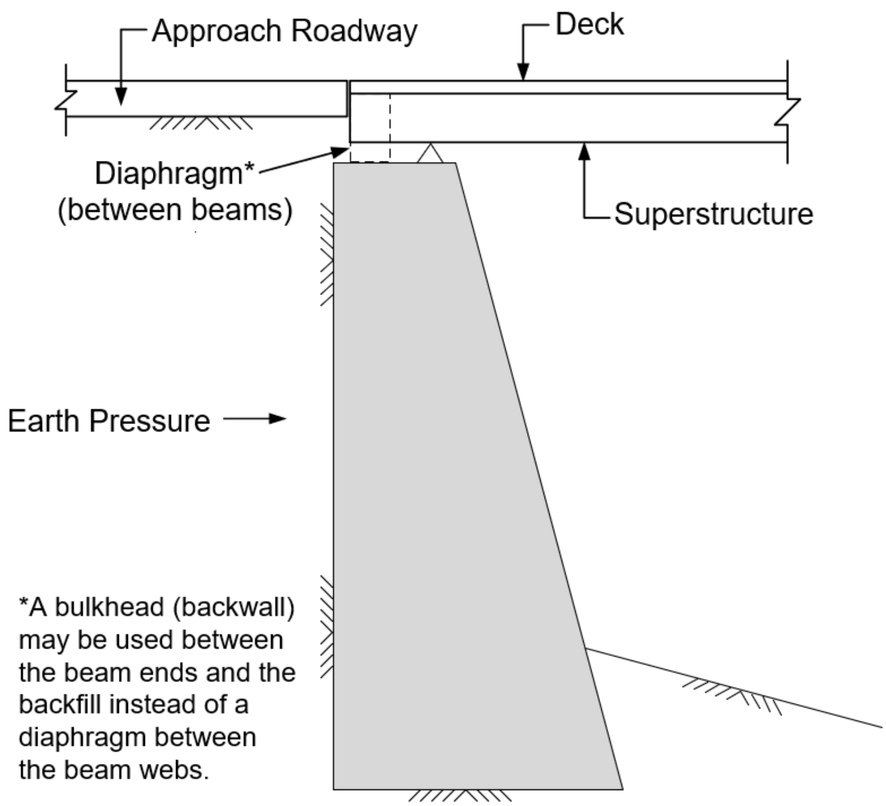


Figure 14.2.22 Gravity Abutment

Counterfort abutments, cellular/vaulted, and cribbed abutments are also other types of abutments which may exist in some substructures.

Counterfort abutments utilize thin walls called counterforts that connect the vertical stem to the footing. The counterforts are typically spaced at regular intervals along the stem, causing both the vertical stem and heel to act as continuous slabs, which is a distinction from the cantilever action in cantilever abutment stems.

A cellular abutment, or vault, is an abutment in which the space between the wings, vertical stem, approach slab, and footings is hollow.

Cribbed abutments are most commonly constructed of timber but can also be made of precast concrete or steel. They consist of individual pieces stacked horizontally, in a log-cabin style, to create an overall structure.

Primary and Secondary Members

Common abutment members include:

- Bearing seat.
- Backwall.
- Abutment stem.
- Footing/pile cap.
- Precast panels.
- Spread footings.
- Deep foundations (piles and drilled shafts).
- Tie backs.
- Soil reinforcing strips.
- Geotextiles.
- Cheek wall.

Any portion of the abutment that carries primary live load (transient load) are considered primary members.

The bearing seat provides a surface to support the bridge superstructure. The backwall retains the approach roadway fill or embankment and keeps it away from the bearing seat. It also may provide support for the approach slab and for the expansion joint, if one is present. The abutment stem, or breast wall, retains the soil behind the abutment and serves as the bearing seat. The foundation, spread footing or deep foundation (piles, drilled shafts, etc.), transmits the weight of the abutment, soil backfill loads, and the reaction forces from the superstructure to the supporting soil or bedrock. 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.

The cheek wall is mostly cosmetic but also protects the end bearings from the environment, (see Figure 14.2.23). A cheek wall is not always present.

Figure 14.2.23 Cheek Wall

Steel Reinforcement

The pattern of primary steel reinforcement used in concrete abutments depends on the abutment type (see Figure 14.2.24). In a cantilever abutment, primary tension reinforcement includes 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 and inclined bars (if present) are generally 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 14.2.25). The spirals or ties may be considered primary reinforcement in seismic zones. Bars used for temperature and shrinkage reinforcement are typically considered secondary reinforcement.

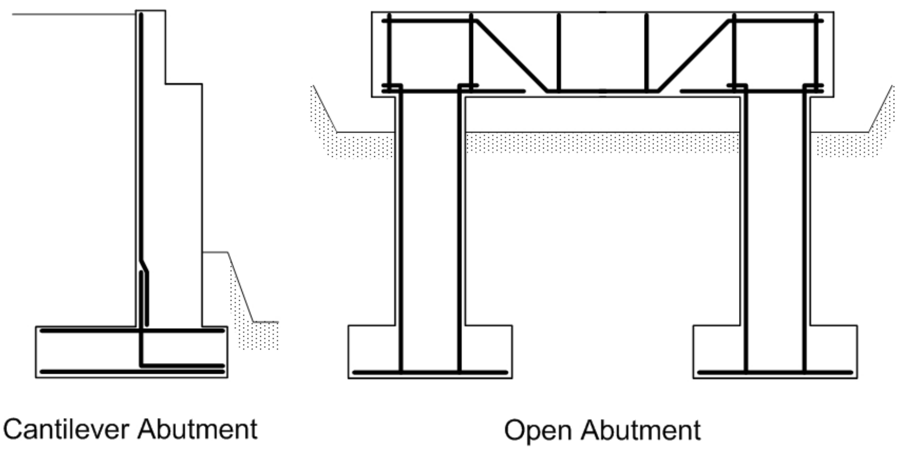


Figure 14.2.24 Primary Reinforcement in Concrete Abutments

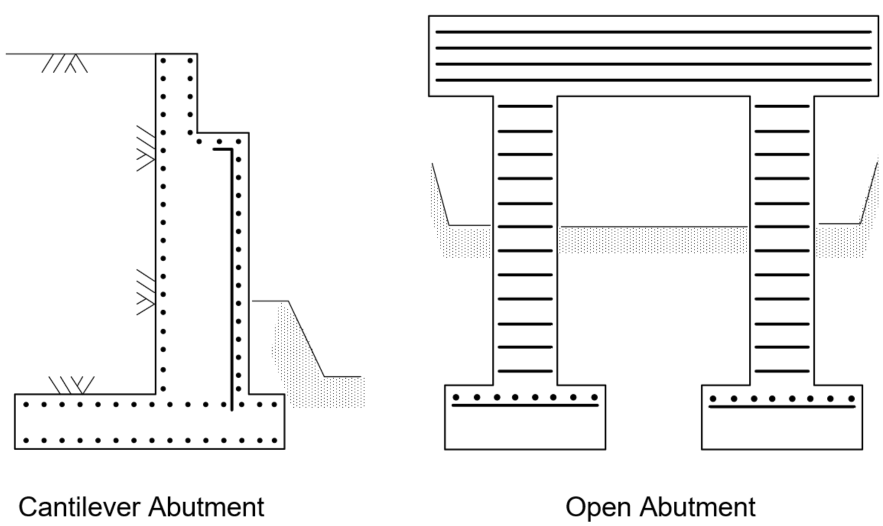


Figure 14.2.25 Secondary Reinforcement in Concrete Abutments

14.2.2 Wingwalls

Design Characteristics

Wingwalls are 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 14.2.26). Cantilever wingwalls may have vertical or sloped faces. A wingwall is similar to an abutment except that it is not needed 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.

In some cases, wingwalls may be aligned with and not discernable from retaining walls that extend for considerable length. In such cases, the beginning of a wingwall should be considered to begin at the first joint away from the abutment and extend to the second joint. If no joint is visible, the wingwall length should be considered approximately equal to the visible height of the abutment, beginning at the projected 'footprint' of the superstructure. Wingwalls do not typically influence the overall rating of the substructure, however, the inspection team should consider the condition of integral abutment wingwalls to the first construction or expansion joint when determining the substructure condition rating (*SNBI* Subsection 7.1, Item B.C.03 Commentary).

Figure 14.2.26 Typical Wingwall

Like abutments, wingwalls may be constructed of concrete, stone masonry (see Figure 14.2.27), steel, timber, or a combination of these materials.

Figure 14.2.27 Masonry Wingwall

Geometrical Configurations

There are several geometrical configurations of wingwalls, and their use is dependent upon the design of the structure:

- Straight - extensions of the abutment wall (see Figure 14.2.28 and Figure 14.2.29).
- Flared - form an acute angle with the bridge roadway (see Figure 14.2.30).
- U-shaped wingwalls - parallel to the roadway (see Figure 14.2.31).

Figure 14.2.28 Typical Straight Wingwall

Figure 14.2.29 Straight Cribbed Wingwall

Figure 14.2.30 Flared Wingwall

Figure 14.2.31 U-shaped Wingwall

Construction Classifications

There are several construction classifications of wingwalls:

- Monolithic – constructed with the abutment, without vertical expansion or construction joints between the wingwall and abutment; for cast-in-place concrete, reinforcing steel from the abutment is interconnected with the wingwall.
- Integral – constructed separately from the abutment with a vertical construction joint; for cast-in-place concrete, the wingwall is interconnected with the abutment through the extension and embedment of the abutment reinforcing steel in the wingwall (see Figure 14.2.32).
- Independent – constructed separately from the abutment; usually an expansion or construction joint separates the wingwall from the abutment (see Figure 14.2.33); for cast-in-place concrete, reinforcing steel from the abutment is not interconnected with the wingwall.

Figure 14.2.32 Integral Wingwall

Figure 14.2.33 Reinforced Concrete Full Height Abutment with Independent Wingwall (MSE Construction)

Primary and Secondary Members

Members of a wingwall can be similar to members of an abutment, except that the superstructure does not bear directly on the wingwalls. Since wingwalls generally do not encounter live load, they may be considered secondary members. Wingwalls do not typically include bearing seats, backwalls, or cheek walls.

Steel 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 14.2.34). Secondary reinforcement is used to resist temperature and shrinkage (see Figure 14.2.35).

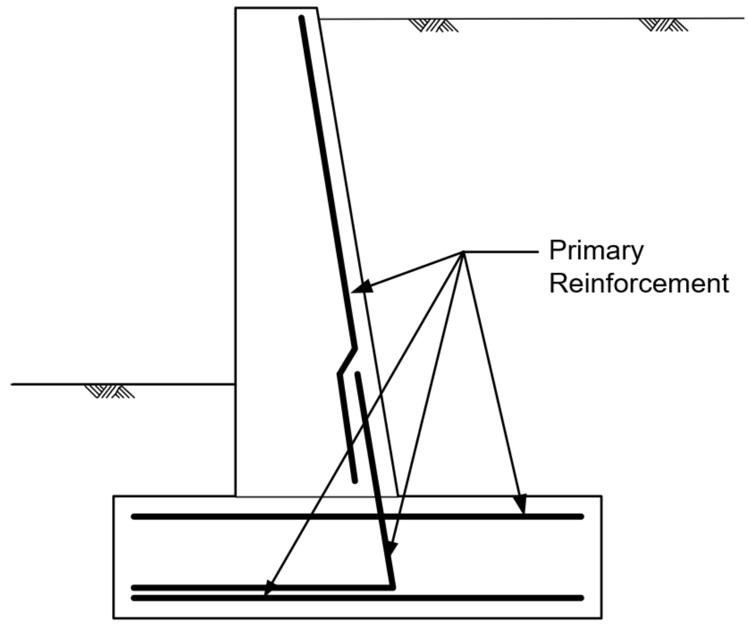


Figure 14.2.34 Primary Reinforcement in Concrete Cantilever Wingwall

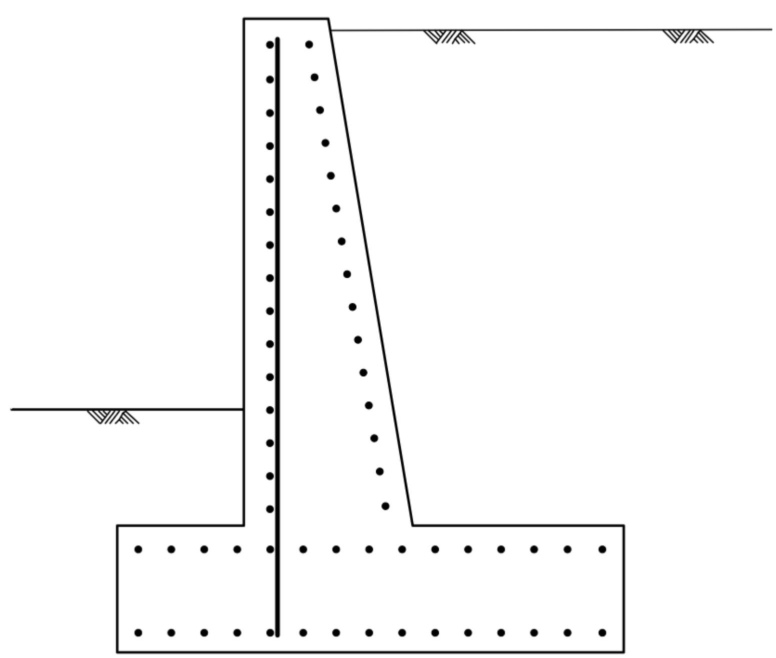


Figure 14.2.35 Secondary Reinforcement in Concrete Cantilever Wingwall

14.2.3 Piers and Bents

Design Characteristics

There is no functional difference between piers and bents. Both serve as an intermediate support between abutments. A pier generally has only one column or shaft supported by one footing (see Figure 14.2.36). Bents have a cap supported by two or more columns with each column supported by an individual footing. Bents may also utilize piles instead of columns.

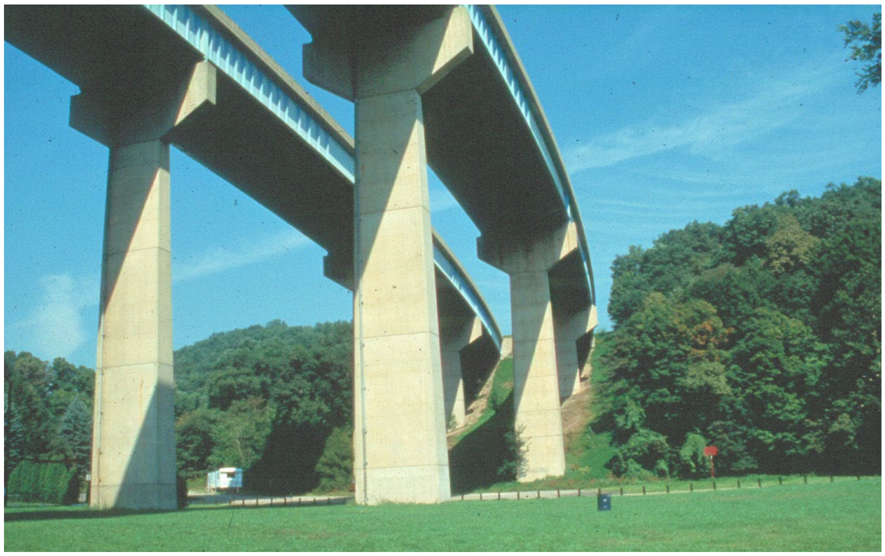


Figure 14.2.36 Typical Concrete Piers

The primary materials used in pier and bent construction are typically unreinforced concrete, reinforced concrete, prestressed concrete, stone masonry, steel, timber, or a combination of these materials (see Figure 14.2.37, Figure 14.2.38, Figure 14.2.49, and Figure 14.2.40). Reinforced concrete is commonly used due to its ability to be formed into different pier shapes.

Figure 14.2.37 Reinforced Concrete Piers under Construction

Figure 14.2.38 Stone Masonry Pier

Figure 14.2.39 Pile Bent with Diagonal Bracing

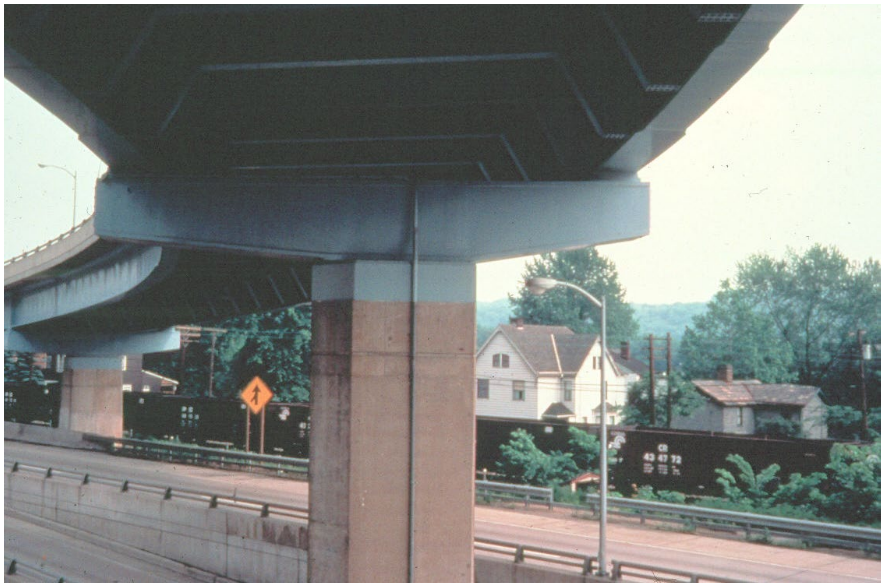


Figure 14.2.40 Combination: Reinforced Concrete Column with Steel Pier Cap

The most common pier and bent types are:

- Solid shaft or wall piers.
- Column piers.
- Column piers with web walls.
- Hammerhead piers.
- Column bents or open bents.
- Pile bents.

A few specialized types of piers include trestles, hollow, straddle, and integral piers.

Solid Shaft Piers

Solid shaft or wall piers are typically used when a large mass is advantageous or when a limited number of load points are necessary for the superstructure (see Figure 14.2.41).

Figure 14.2.41 Solid Shaft or Wall Pier

Column Piers

Column piers are typically used when there is limited ground space available under the structure or when narrow superstructure widths are necessary (see Figure 14.2.42). Column piers may have spread footings, pile caps, or may be extensions of drilled shafts under the surface.

Figure 14.2.42 Column Pier

A web wall can be connected to columns to add stability to the pier (see Figure 14.2.43 and Figure 14.2.44). 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. Web walls may have been installed during original construction or added as a retrofit to address seismic impacts or build-up of stream debris. Refer to plans in the bridge files to determine the structural nature of the web wall.

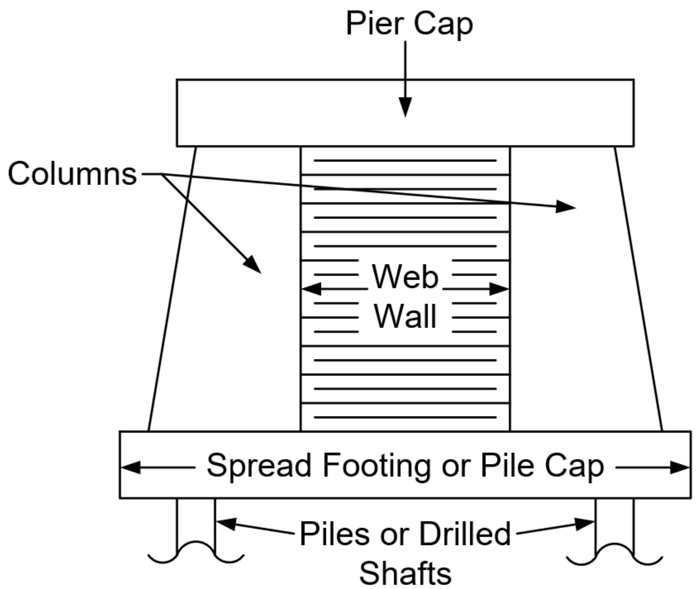


Figure 14.2.43 Column Pier with Web Wall Schematic

Figure 14.2.44 Column Pier with Web Wall

Hammerhead Piers

The cantilever or hammerhead pier is a modified column pier with double cantilevered cap for use with wide superstructures (see Figure 14.2.45 and Figure 14.2.46).

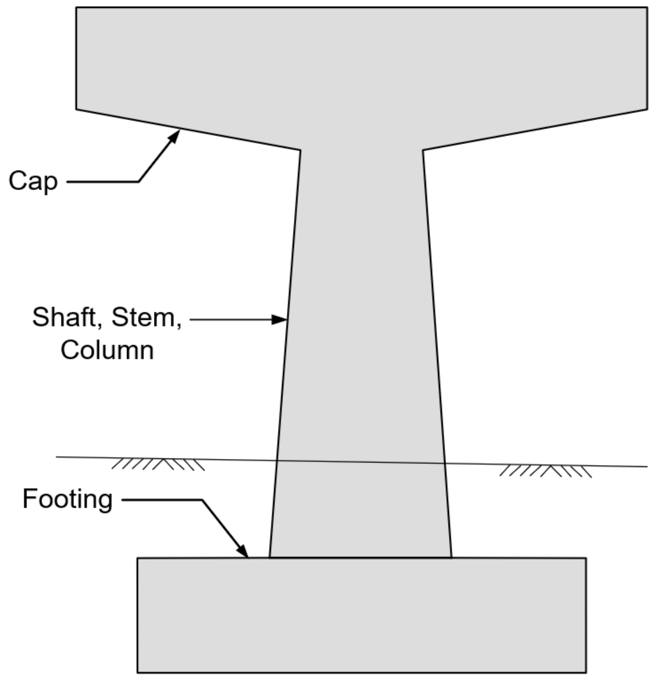


Figure 14.2.45 Hammerhead Pier Sketch

Figure 14.2.46 Hammerhead Pier

Column/Open Bents

The column bent is a common pier type for highway grade crossings (see Figure 14.2.47).

Figure 14.2.47 Column Bent or Open Bent

Pile Bents

Common pile types for pile bents include timber piles, steel h-piles, steel pipe piles, prestressed concrete piles, concrete cylinder piles and composite piles. Refer to Section 14.2.4 for more information on deep foundations. Typically, piles are driven in place and support a continuous cap (see Figure 14.2.48).

Figure 14.2.48 Concrete Pile Bent

Trestles

Although less common, some structures utilize trestles or towers as an intermediate substructure unit. Many of these structures are steel trusses with large vertical clearances (see Figure 14.2.49).

Figure 14.2.49 Steel Trestle or Tower

Hollow Piers

Hollow piers are usually tall shaft type piers built for bridges crossing deep valleys. Being hollow greatly reduces the dead load of a concrete pier and increases its ductility. Whether precast or cast-in-place, hollow piers are typically constructed in segments. If precast, the segments are attached using post-tensioning and the joints are epoxy-sealed. Hollow steel piers were common in the past due to availability of the material and inexpensive construction methods (see Figure 14.2.50). The decrease in the dead load, or self-weight, of the piers provides ease in transporting segments to the site, and the high ductility provides for enhanced performance against seismic forces.

Figure 14.2.50 Hollow Steel Hammerhead Pier

Integral Piers

Another specialized type of pier is an integral pier. 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 (see Figure 14.2.51 and Figure 14.2.52). The concrete cap is likely to be post-tensioned rather than conventionally reinforced.

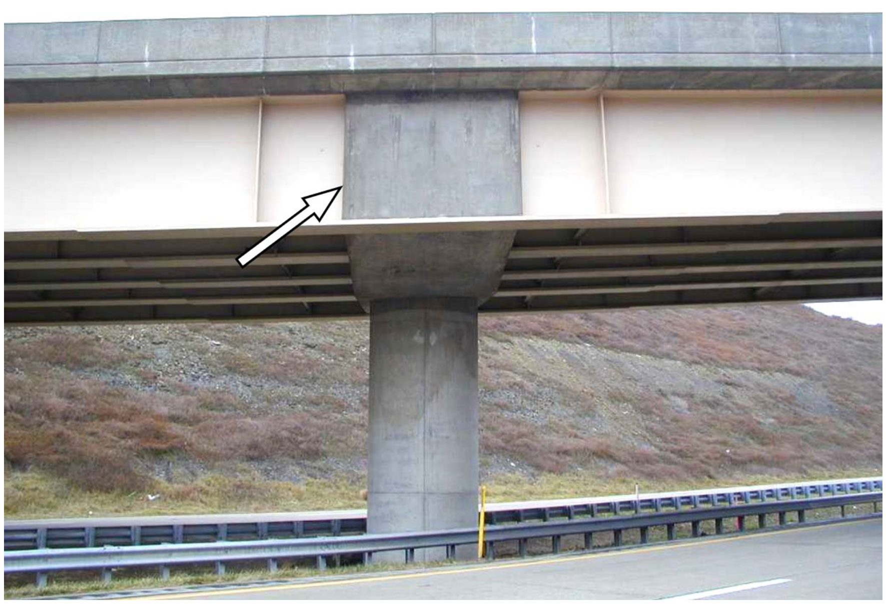


Figure 14.2.51 Integral Concrete Pier Cap

Figure 14.2.52 Concrete Column Pier with Integral Pier Cap

Primary and Secondary Members

The primary pier and bent members typically are:

- Pier cap or bent cap.
- Pier wall/stem/shaft.
- Column.
- Spread footing or pile cap.
- Piles or drilled shafts.

The pier cap or bent cap provides support for the bearings and the superstructure (see Figure 14.2.53). 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, as well as the superstructure loads to the supporting soil or bedrock. The footing also provides stability to the pier or bent against overturning and sliding forces.

Secondary members may consist of web walls and diagonal bracing used to reduce the unbraced length of compression members in bents.

Figure 14.2.53 Cantilevered Piers Joined by a Web Wall

Steel 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 tend to be more lightly reinforced, but still have significant vertical reinforcement to resist horizontal loads.

If primary steel is not necessary at a given location, then secondary reinforcement 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 needed 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 in union.

Figure 14.2.54, Figure 14.2.55, and Figure 14.2.56 illustrate typical reinforcement patterns.

Design specifications may call for epoxy coated reinforcement if the substructure is subjected to corrosive environments.

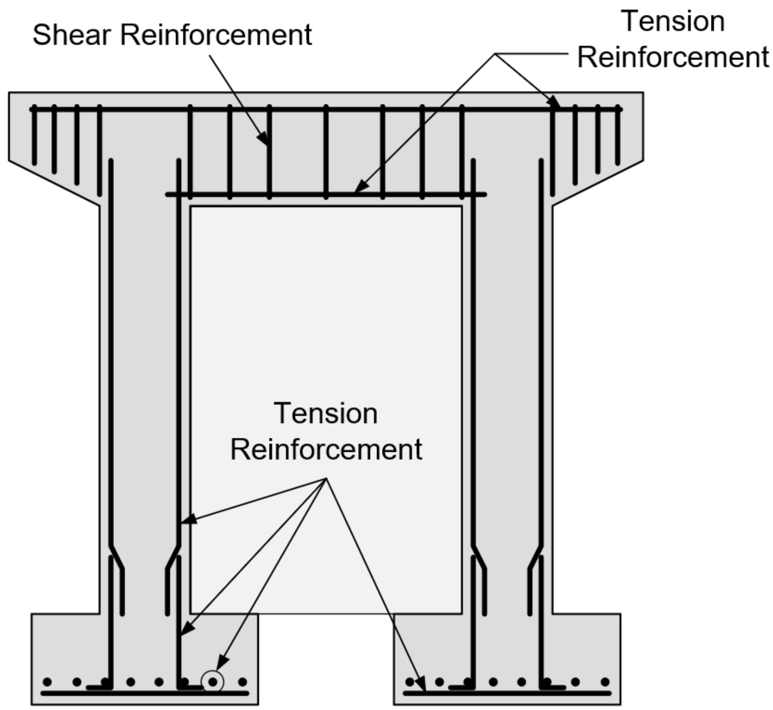


Figure 14.2.54 Primary Reinforcement in Column Bent with Web Wall

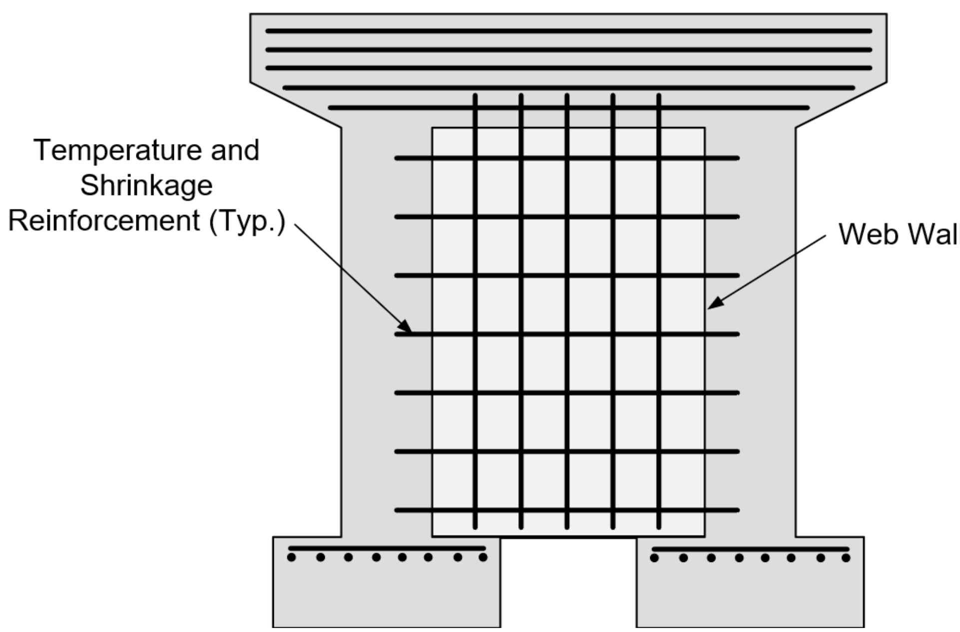


Figure 14.2.55 Secondary Reinforcement in Column Bent with Web Wall

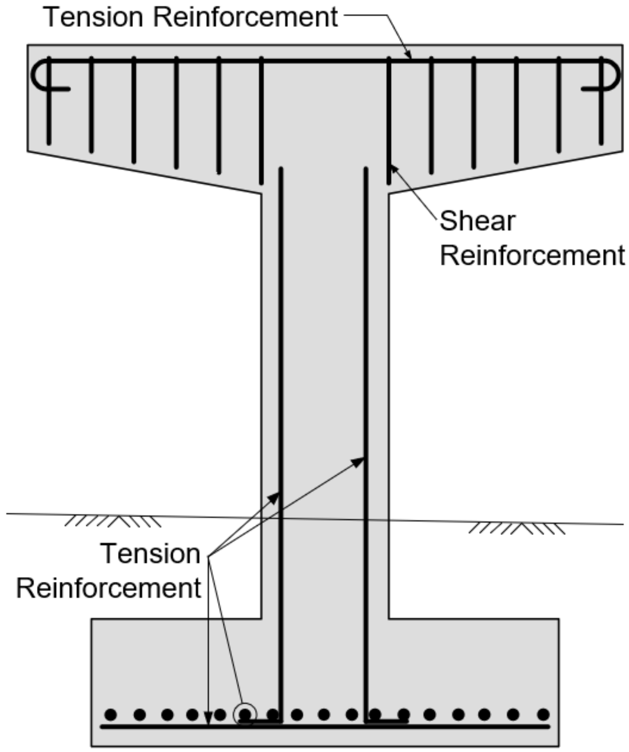


Figure 14.2.56 Primary Reinforcement for a Cantilevered Pier

14.2.4 Foundation Types

Foundations are critical to the stability of the bridge since the foundation ultimately supports the entire structure.

The two main types of bridge foundations are spread footings and deep foundations (see Figure 14.2.57). *SNBI* Item B.SB.06 reports the Foundation Type for each bridge. Spread footing foundation types are classified into either reinforced soil earth or footings, either on rock or soil. Deep foundations include piles, drilled shafts, or caissons.

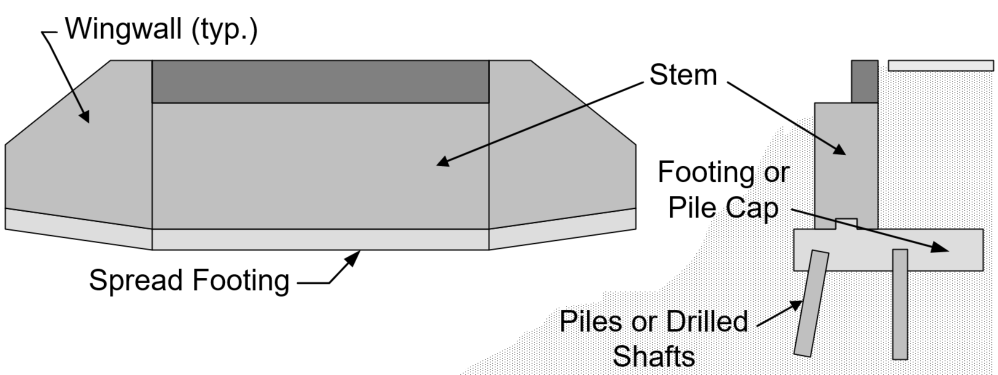


Figure 14.2.57 Bridge Foundation Types

Spread Footings

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 bedrock or soil. Though a spread footing is usually buried, it is generally covered with a minimal amount of soil. The depth of the footing and riprap provided are designed for the anticipated scour at the bridge site. In cold regions, the bottom of a spread footing is placed below the recognized maximum frost line depth for that area.

Deep Foundations

A deep foundation is used when the underlying soil is not suited for supporting the bridge, or to extend the foundation below the scour level.

A pile is a long, slender support that is typically driven into the ground. In some cases where driving is difficult to get to a desired pile tip elevation, piles may be placed in pre-drilled holes and then driven as appropriate. Various numbers and configurations of piles can be used to support a bridge foundation. Piles can be partially exposed and can consist of several materials and shapes, including:

- Steel H-shape (see Figure 14.2.58).
- Steel pipe.
- Concrete (cast-in-place or precast).
- Timber.
- Auger cast.
- Micropile.
- Composite.
- FRP composite.

Also reference *GEC 12 - Design and Construction of Driven Pile Foundations (FHWA-NHI-16-009 and FHWA-NHI-16-010)* for a list of pile types. This type of foundation transfers load to sound material well below the surface or, in the case of friction piles, to the surrounding soil.

Figure 14.2.58 Stub Abutment on Piles with Piles Exposed

Caissons, drilled shafts, or bored piles are other types of deep foundations used when the soil is not competent to support a spread footing. Holes are typically 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 (see Figure 14.2.59). Temporary steel casing is removed after the concrete is placed and is capable of withstanding the surrounding pressures. The minimum drilled shaft diameter used for bridge substructure construction is typically 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 14.2.59 Caissons with Permanent Steel Casing

Other deep foundation types include micro-piles and stone columns. Micro-piles are small diameter piles that are drilled and grouted, and generally grouped in union under a footing. Vibratory stone columns are an array of crushed stone pillars placed in the soil and consolidated using vibration. They act as compacted columns of aggregate that improve the geotechnical properties of the existing soil.

14.2.5 Substructure Protection

Piers can be vulnerable to collision damage from trucks, trains, ships, ice flows, and waterborne debris. Wall type piers are typically resistant to this type of collision damage and for this reason are often used in navigable waterways and waterways that may freeze. Collision walls also serve to protect columns (see Figure 14.2.60 and Figure 14.2.61).

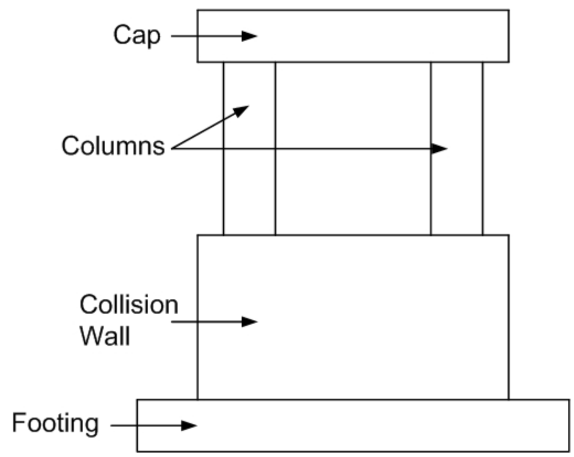


Figure 14.2.60 Collision Wall Schematic

Figure 14.2.61 Collision Wall

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 Figure 14.2.62 and Figure 14.2.63). Fenders are protective fences surrounding a pier to protect it from marine traffic. They may consist of timber pile arrangements, steel or concrete frames, or cofferdam sheets (see Figure 14.2.64 and Figure 14.2.65).

Figure 14.2.62 Concrete Collision Protection Block

Figure 14.2.63 Timber Dolphin

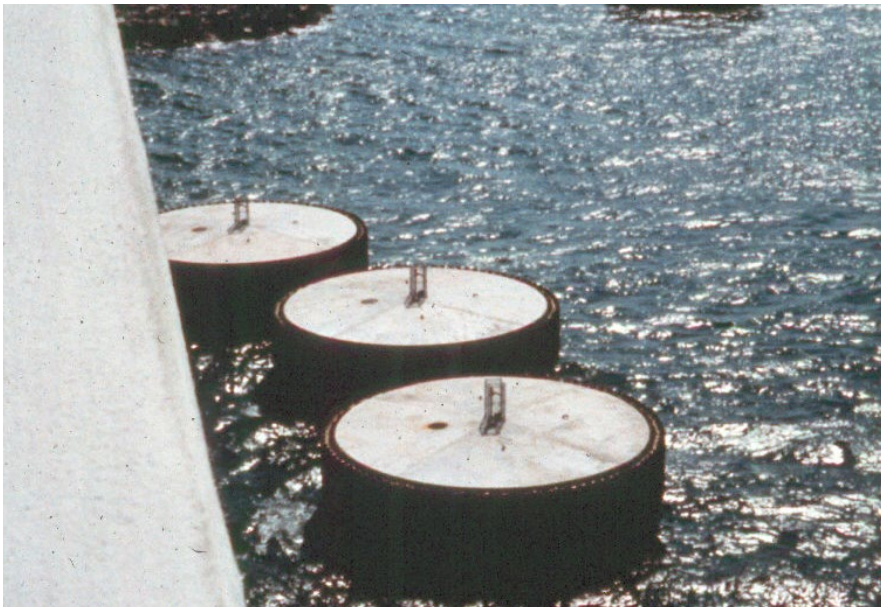


Figure 14.2.64 Concrete Dolphins

Figure 14.2.65 Steel Fender

Section 14.3 Overview of Common Deficiencies

14.3.1 Concrete Deficiencies

Common deficiencies that may occur on concrete substructures include:

- Cracking (structural).
- Cracking (non-structural).
- Scaling.
- Delamination.
- Spalling.
- Chloride contamination.
- Freeze-thaw.
- Efflorescence.
- Alkali silica reactivity (ASR).
- Ettringite formation.
- Honeycombs.
- Pop-outs.
- Impact damage.
- Abrasion.
- Overload damage.
- Fire/heat damage.
- Reinforcing steel corrosion.
- Carbonation.
- Loss of prestress.

Refer to Chapter 7 for a detailed explanation of the properties of concrete, types, and causes of concrete deficiencies.

14.3.2 Steel Deficiencies

Common potential deficiencies that may occur on steel substructures include:

- Corrosion.
- Fatigue cracking.
- Overload damage.
- Impact damage.
- Fire/heat damage.
- Coating failures.
- Loose, missing, or deteriorated fasteners.
- Out-of-plane distortion (including buckling).

Refer to Chapter 7 for a detailed explanation of the properties of steel, types, and causes of steel deficiencies.

14.3.3 Masonry Deficiencies

Common potential deficiencies that may occur on masonry or stacked stone substructures include:

- Missing stones or mortar.
- Weathering.
- Spalling.
- Splitting.
- Fire/heat damage.

Refer to Chapter 7 for a detailed explanation of the properties of masonry, types, and causes of masonry deficiencies.

14.3.4 Timber Deficiencies

Common potential deficiencies that may occur on timber substructure include:

- Checks, splits, shakes.
- Decay by fungi.
- Damage by insects and borers.
- Loose connections.
- Loose or deteriorated stressing rods.
- Surface depressions.
- Fire/heat damage.
- Impact damage.
- Abrasion.
- Damage from overstress.
- Weathering/warping.
- Failure of protective system.
- Chemical attack.

Refer to Chapter 7 for a detailed explanation of the properties of timber, types, and causes of timber deficiencies.

Section 14.4 Inspection Methods

14.4.1 Visual and Physical Inspection

The inspection methods are generally comparable for most substructure members. Differential settlement may be present if substructure units have:

- Concrete: vertical or diagonal cracks.
- Steel: abnormal distortion.
- Masonry: cracks in stone or gaps in mortar.
- Timber: loose or broken connectors.

Substructure settlement may also be detected by unusual vertical movement in the railing, deck, or superstructure. Misaligned bearings can also be an indication of differential settlement.

The specific visual, physical, and advanced inspection methods are typically dependent upon the type of material used in the substructure. The methods are generally similar to the inspection of superstructures. Refer to Chapter 9 (Concrete), Chapter 10 (Steel), or Chapter 11 (Timber and Stone Masonry), for specific material deficiencies and inspection methods.

Once the deficiencies are identified visually, physical methods are generally used to verify the extent of the deficiency. Other similar locations and details should be examined for similar deficiencies.

Areas of concrete or rebar deterioration identified visually should be examined physically using an inspection hammer to check for delaminated areas. The location, length, and width of cracks found during the visual inspection should be measured and recorded. Masonry substructure units can be inspected in a similar fashion as concrete. The condition of both stones and mortar should be checked.

For steel members, the most common physical inspection methods involve cleaning the member and inspecting for cracks and corrosion, which results in loss of member material. The inspection team should determine section loss by measuring with a straight edge and a tape measure if possible. A more exact method of measurement, such as calipers or an ultrasonic thickness gauge (D-meter), are typically used to measure the remaining section of steel. During the passage of traffic, inspectors should listen for abnormal noises caused by moving members and loose connections.

For timber members, the location and length of cracks and splits (through checks) should be measured and recorded, as well as the depth of penetration for checks and shakes found during visual inspection. An inspection hammer can be used to tap on areas and determine the extent of internal decay. This is done by listening to the sound the timber makes after being struck with the hammer. Similar to steel member inspection, inspectors should listen for abnormal noises caused by moving members and loose connections under moving traffic.

14.4.2 Advanced Inspection

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

There are several advanced methods for concrete inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Electrical Resistivity (ER).
- Galvanostatic Pulse Measurement (GPM).
- Ground Penetrating Radar (GPR).
- Half-Cell Potential (HCP).
- Impact Echo (IE).
- Infrared Thermography (IT).
- Linear Polarization (LPR).
- Magnetic Flux Leakage (MFL).
- Magnetometer (MM).
- Rebound and penetration methods.
- Ultrasonic Pulse Velocity (UPV).
- Ultrasonic Pulse Echo (UPE).
- Ultrasonic Surface Waves (USW).

Other methods that may be considered destructive testing methods include the following.

- Concrete cores.
- Borescopes.
- Moisture content.
- Petrographic examination.
- Alkali-Silica Reaction (ASR) evaluation.
- Reinforcing steel sample.
- Rapid chloride permeability testing.

There are several advanced methods for steel inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Acoustic Emission (AE).
- Crack Propagation Gage (CPG).
- Dye Penetrant Testing (PT).
- Eddy Current Testing (ET).
- Magnetic Particle Testing (MT).
- Magnetic Flux Leakage (MFL).
- Ultrasonic Testing (UT).
- Phased Array Ultrasonic Testing (PAUT).

Other methods that may be considered destructive testing methods include the following.

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

There are several advanced methods for timber inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Stress wave velocity.
- Ultrasonic testing.
- Vibration.

Other methods that may be considered destructive testing methods include the following.

- Drilling and probing.
- Moisture meter.

Advanced methods for common bridge materials are described in detail in Chapter 17.

Section 14.5 Inspection Areas and Deficiencies

Although all areas should be inspected during a Routine inspection, the areas to receive greater emphasis during inspections can be related to common pier and bent problems. Stability typically is a paramount concern; therefore, checking for various forms of movement is necessary during the inspection of piers or bents.

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

- Areas subjected to movement.
- Bearing areas.
- Shear zones.
- Flexural zones.
- Areas exposed to drainage.
- Areas exposed to traffic.
- Areas previously repaired.
- Scour and undermining.
- Dolphins and fenders.
- Other problem areas.

14.5.1 Areas Subjected to Movement

The most common types of movement observed during the inspection of substructures can be summarized as:

- 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, typically has little detrimental effect on the structure. Uniform settlements of up to 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 14.5.1 and Figure 14.5.2). It may also occur under a single substructure unit (see Figure 14.5.3 and Figure 14.5.4). 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. Differential settlement in substructure units could introduce cracking in decks or slabs. In severe cases this can result in complete loss of support for a span, resulting in collapse.

Some of the most common causes of movement are soil bearing failure, soil consolidation, scour, and undermining. Some less prevalent issues can include subsidence from mining or solution cavities, and liquefaction induced by seismic events.

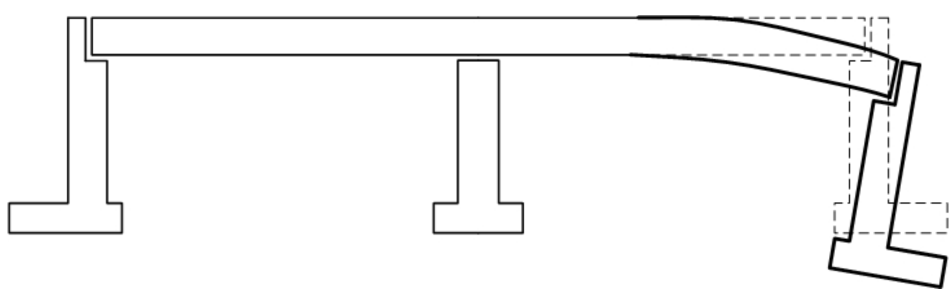


Figure 14.5.1 Differential Settlement and Rotation at an Abutment

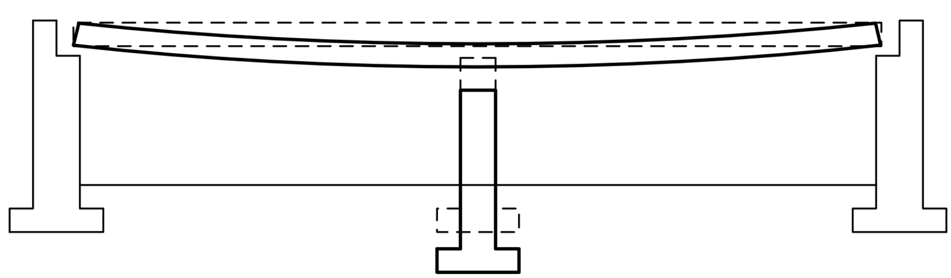


Figure 14.5.2 Differential Settlement for Bridge

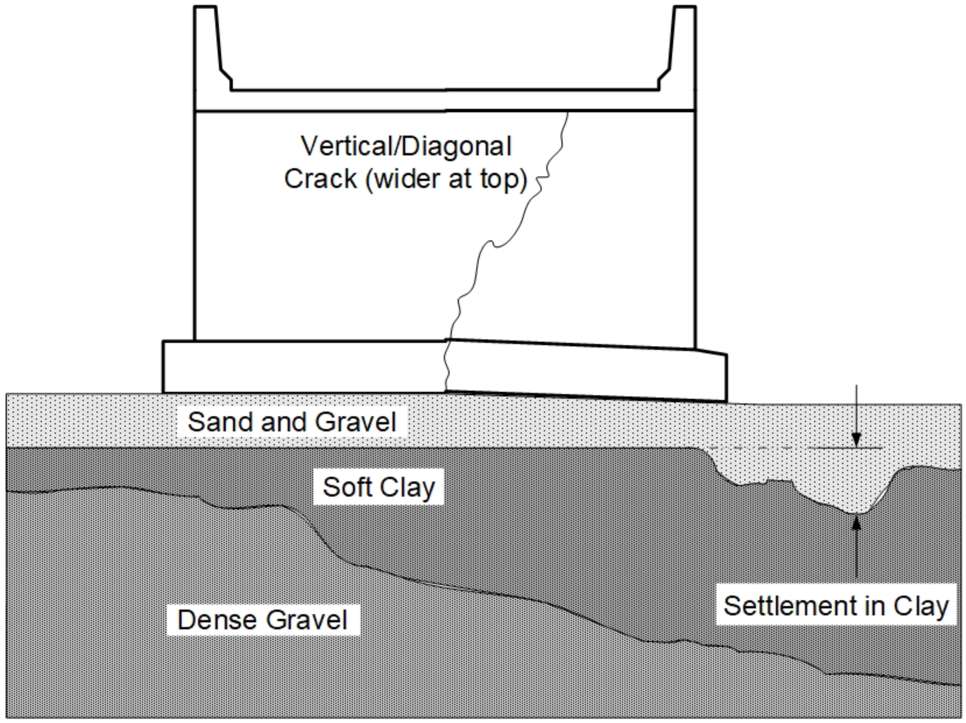


Figure 14.5.3 Differential Settlement under an Abutment

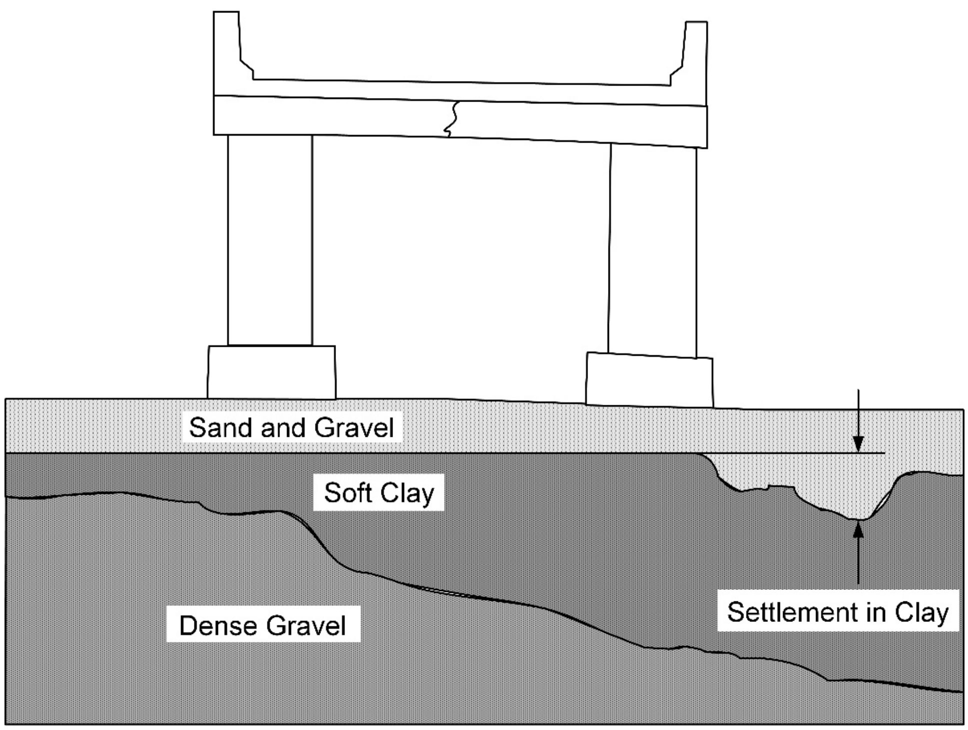


Figure 14.5.4 Differential Settlement under a Column Bent

Inspection for vertical movement, or settlement, should include:

- The joint opening should be checked for unusual opening widths and unevenness across the joints. In some cases, pavement expansion or approach fill expansion could conceivably cause vertical movement in the approach slab.
- For bridges with multiple simple spans, the joint in the deck above the pier as well as at adjacent piers and at the abutments should be examined.
- Inspectors should check for any new or unusual cracking in the substructure units. Inspectors should also check for growth (width, length) in any previously reported cracks (see Figure 14.5.5).
- Buckling in steel columns of the pier or bent should be investigated.
- The superstructure alignment should be checked for evidence of settlement. Inspectors should sight along parapets, bridge rails, etc. (see Figure 14.5.6).
- Any misaligned bearings should be checked, which can be an indication of differential settlement.
- Inspectors should examine the joint that separates the wingwall and abutment for proper alignment.
- Scour and undermining around the footings should be investigated.
- In some cases, bearing seat or top of pier elevations should be checked; surveying equipment may be necessary.
- The deck joints should be investigated. The deck joint openings should be consistent with the recorded temperature and show no evidence of differential settlement.

Figure 14.5.5 Crack in Abutment due to Differential Settlement

Figure 14.5.6 Deck and Railing Evidence of Pier Settlement

Earth retaining structures, such as abutments and retaining walls, can be susceptible to lateral movements, or sliding (see Figure 14.5.7). Lateral movement occurs when the horizontal earth pressure acting on the wall exceeds the friction and other forces (such as a shear key) that hold the structure in place.

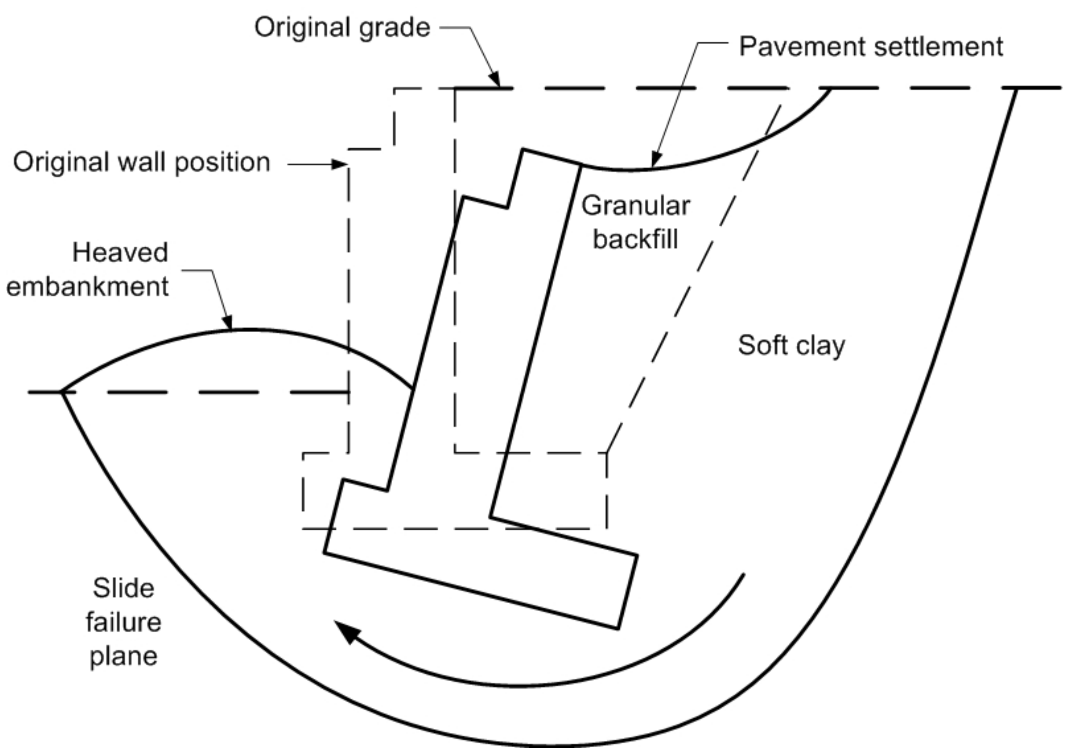


Figure 14.5.7 Lateral and Rotational Abutment Movement due to Slope Failure

The most common causes of lateral and rotational movement can include slope failure, increased hydrostatic pressure from blocked drains and weep holes, changes in soil characteristics (e.g., frost action and ice), and time consolidation of the original soil.

Inspection for lateral movement, or sliding, should include:

- The general alignment of the substructure unit should be inspected.
- Inspectors should examine the bearings and interfaces for evidence of lateral displacement (see Figure 14.5.8 and Figure 14.5.9).
- Inspectors should investigate the joint opening between the deck and the approach slab. The approach slab and roadway should be checked for settlement (see Figure 14.5.10).
- The deck joints should be investigated. The deck joint openings should be consistent with the recorded temperature and show no evidence of differential settlement.
- The opening in the construction joint between the wingwall and the abutment should be examined.
- The distance between the end of the superstructure and the backwall should be checked.
- Inspectors should examine for clogged drains (approach roadway, weep holes, and substructure drainage).
- Cracking or spalling that may otherwise be unexplained, should be inspected; in the case of inspections after earthquakes, such damage is readily apparent (see Figure 14.5.11).
- Inspectors should check for erosion of the embankment material (see Figure 14.5.12) and scour or undermining around the footings (see Figure 14.5.13).

Refer to Chapter 16 for a more detailed description of scour, undermining, and underwater inspection.

Figure 14.5.8 Bearing Displacement Indicating Possible Movement of Abutment

Figure 14.5.9 Local Failure in Timber Pile/Abutment Seat Interface Indicating Possible Movement of Abutment

Figure 14.5.10 Vertical Misalignment Between Approach Slab and Bridge Deck

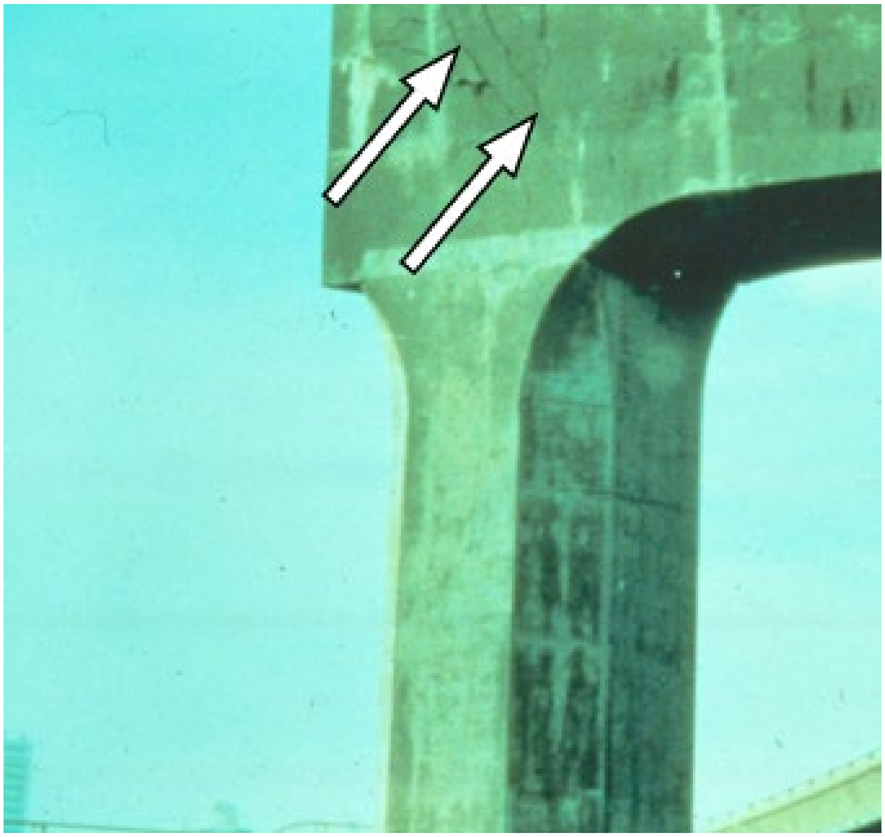


Figure 14.5.11 Diagonal Cracks in Bent Cap due to Earthquake

Figure 14.5.12 Erosion at Abutment Exposing Piles

Figure 14.5.13 Pier Movement and Superstructure Damage due to Scour/Undermining

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 14.5.14).

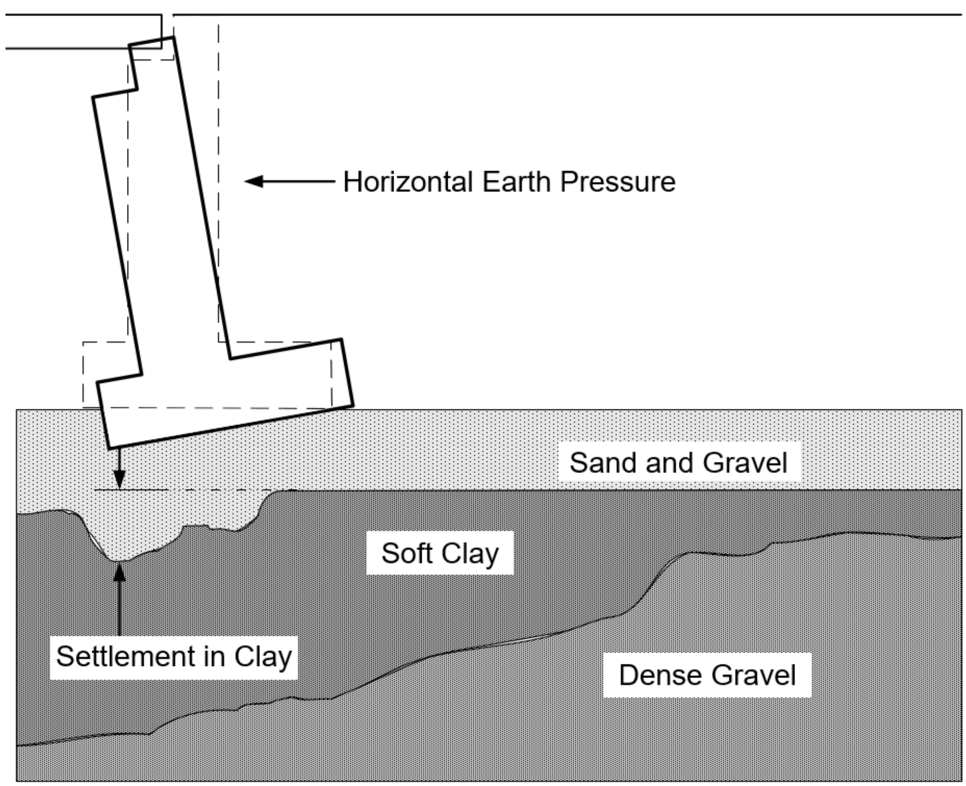


Figure 14.5.14 Rotational Movement of an Abutment

Abutments and walls are typically more susceptible to this rotational type of movement. The most common causes of rotational movement typically 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, should include:

- The vertical alignment of the abutment or pier should be checked using a plumb bob or level; keeping in mind that some abutments or piers are constructed with a battered or sloped front face (see Figure 14.5.15).
- The clearance between the beams and the backwall should be examined. Inspectors should investigate the clearance between the ends of the simply supported beams at piers.
- Inspectors should check for unusual joint openings in expansion joints and the joints between the abutment and wingwalls (see Figure 14.5.16).
- Clogged drains or weep holes should be inspected.
- Inspectors should investigate for unusual cracks or spalls.
- Scour or undermining around the footing should be checked. Refer to Chapter 16 for a detailed description of scour, undermining, and underwater inspection.

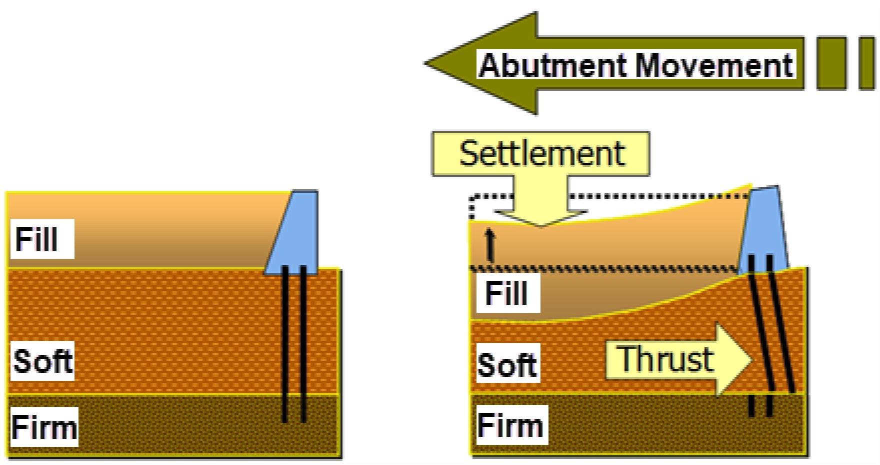


Figure 14.5.15 Rotational Movement due to Lateral Squeeze of Embankment Material

Figure 14.5.16 Rotational Movement of a Concrete Wingwall

14.5.2 Bearing Areas

Areas of bearing include:

- Bearing seats.
- Pier caps.
- The connection between the footing and the abutment, pier shaft, or bent column.
- The area where the footing is supported by earth or deep foundations.

In timber substructures, inspectors should look for crushing. Inspectors should look for cracking or spalling in concrete and masonry substructure members (see Figure 14.5.17 and Figure 14.5.18). Steel substructure members should be examined for buckling, fatigue cracks, or distortion.

Figure 14.5.17 Crack in Concrete Pedestal at Bearing Location

Figure 14.5.18 Cracking in Bearing Seat of Concrete and Stone Abutment

14.5.3 Shear Zones

Vertical forces can cause high shear zones in pier caps close to points of support. Horizontal forces can cause high shear zones on the bottom of the pier shaft or bent column, bottom of the backwall, and bottom of the abutment stem. In timber piers or bents, inspectors should look for splitting. The inspection team should also look for diagonal cracks in concrete and masonry substructure members (see Figure 14.5.19). Steel superstructure members should also be examined for buckling or distortion.

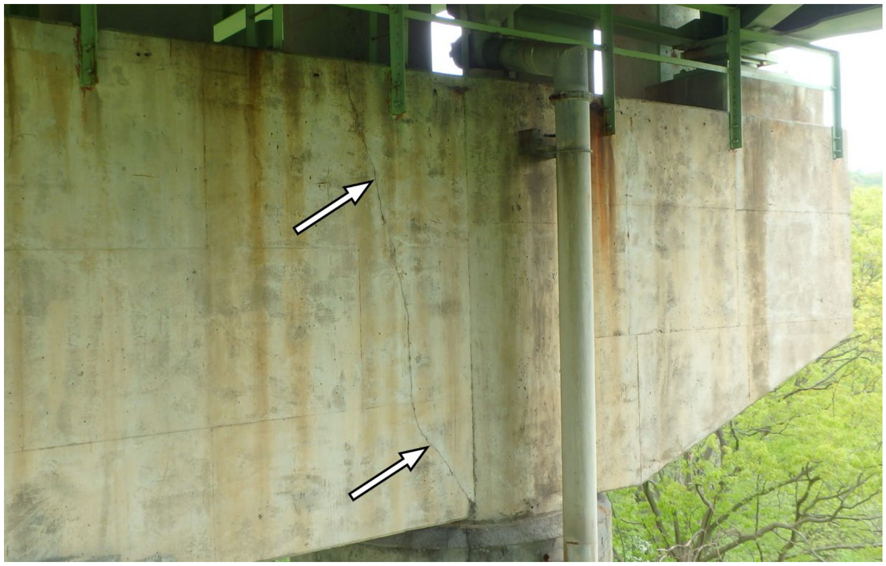


Figure 14.5.19 Crack in Shear Zone of a Pier Cap

14.5.4 Flexural Zones

High flexural moments caused by horizontal forces typically occur at the interface between the backwall and abutment stem. Similarly, bending moments tend to be larger at the bottom of the pier shaft or bent column. High flexural moments may also be present at the footing toe and abutment stem or pier/bent shaft. Moments cause compression and tension depending on the load type and location of the member neutral axis. Inspectors should look for deterioration caused by overstress due to compression or tension caused by flexural moments. The pier cap should be examined for signs of overstress in the positive and negative bending moment regions. Compression areas should be checked for timber splitting, concrete crushing, or steel buckling. Tension areas should be examined for cracking or distortion.

14.5.5 Areas Exposed to Drainage

Water can leak through the deck joints or flow over the shoulder areas to reach the substructure. The inspection team should examine areas such as backwalls (see Figure 14.5.20), bearing seats, and pier caps for signs of water leakage, and dirt and debris build-up. Inspectors should look for material deficiencies caused by exposure to moisture, such as corrosion and section loss on steel, spalls and delaminations on concrete (see Figure 14.5.21 and Figure 14.5.22), and decay on timber (see Figure 14.5.23). The substructure unit at the ground level or water level should be examined for similar deterioration. The inspection team can remove a small amount of soil adjacent to one or more steel and timber piles, as this is often an area of corrosion and decay.

Figure 14.5.20 Cracking and Efflorescence in Backwall

Figure 14.5.21 Widespread Concrete Spalling on Bent Cap due to Drainage

Figure 14.5.22 Concrete Spalling on Concrete Column

Figure 14.5.23 Timber Bent Cap with Protective Flashing and Decay Due to Drainage

Weep holes and subsurface drainage pipes should be checked to ensure that they are clear and functioning. Inspectors should be careful of any animal or insect nests that may be in the weep holes. The inspection team should look for signs of discoloration under the weep holes, which may indicate that the weep holes or substructure drainage pipes are functioning properly (see Figure 14.5.24). Presence of silt could indicate loss of fill material and lead to approach settlement. The condition should be checked for any drainage system that is placed adjacent to the abutment that may result in deterioration of the abutment.

Figure 14.5.24 Abutment with Weep Holes and Staining Underneath

14.5.6 Areas Exposed to Traffic

Inspectors should check for collision damage from vehicles passing adjacent to structural members (see Figure 14.5.25).

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

Figure 14.5.25 Collision Damage to Concrete Pier Column

14.5.7 Areas Previously Repaired

Any repairs that have been previously made should be thoroughly examined. The inspection team should determine if repaired areas are sound and functioning properly by looking for cracking, delamination, and other visible defects, and by hammer sounding.

For concrete members, effective repairs and patching are usually limited to protection of exposed reinforcement (see Figure 14.5.26). For steel members, the location and condition of any repair plates and their connections should be documented. For timber members, the inspection team should document the location and condition of repaired areas and their connections.

Figure 14.5.26 Repaired Concrete Pier Cap

14.5.8 Scour and Undermining

The inspection team should check for signs of scour or undermining (see Figure 14.5.27 and Figure 14.5.28). Refer to Chapter 16 for a more detailed description of scour, undermining, and underwater inspection.

Figure 14.5.27 Abutment Failure from Undermining due to Scour

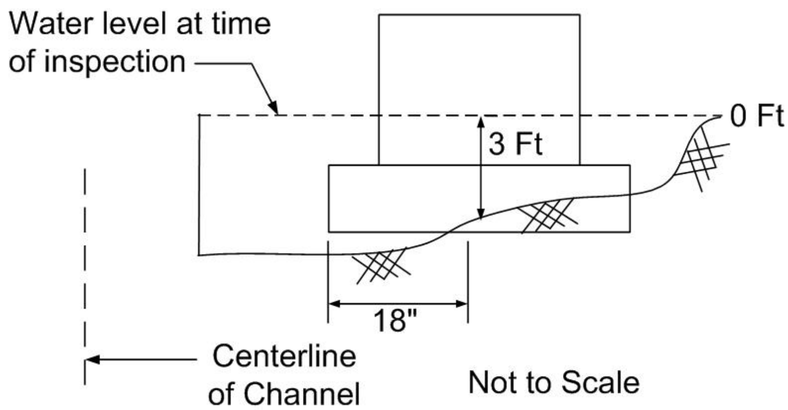


Figure 14.5.28 Inspection Sketch of Scour at a Pier

Inspection for scour should include probing around the substructure footings for signs of undermining (see Figure 14.5.29, Figure 14.5.30, and Figure 14.5.31). Sometimes silt loosely fills in a scour hole and offers no protection or bearing capacity for the footing.

Figure 14.5.29 Inspector Checking for Scour

Figure 14.5.30 Undermining of Concrete Wingwall

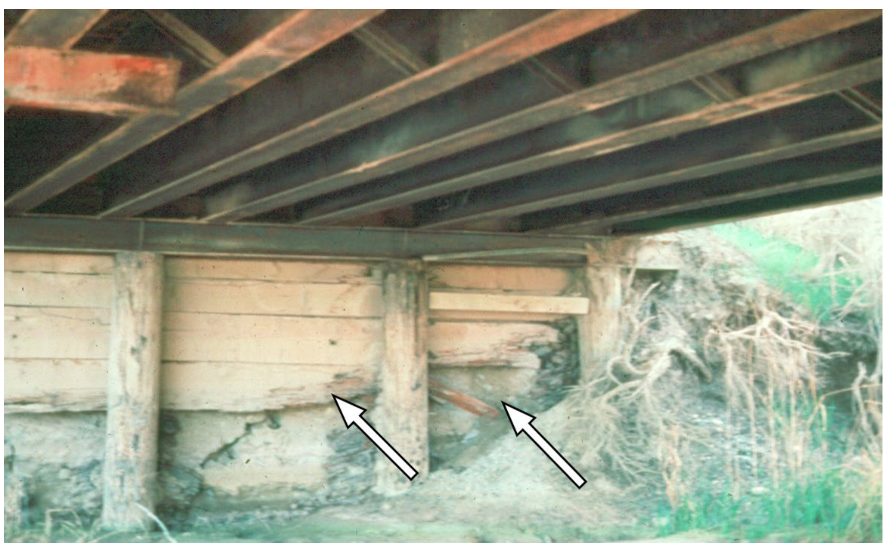


Figure 14.5.31 Decayed Timber Lagging and Abrasion Exposed by Scour

14.5.9 Dolphins and Fenders

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

In concrete pier protection members, inspectors should check for spalling and cracking of concrete or corrosion of the reinforcing steel. Inspectors should investigate for hourglass shaping of piles due to abrasion at the waterline and check for structural damage caused by marine traffic.

In steel pier protection members, inspectors should observe the splash zone (up to two ft above high tide or mean water level) carefully for corrosion. Where there are no tides, the area from the mean water level to two ft above it should be checked. The inspection team should examine steel members for corrosion and check for structural damage.

In timber pier protection members, the inspection team should observe the portions between the high waterline and the mud line for marine borers, caddisflies, and decay, and should check for structural damage (see Figure 14.5.32). Inspectors should check for hourglass shaping of piles at the waterline.

Figure 14.5.32 Timber Fender System with Deteriorated Piles

14.5.10 Other Problem Areas

Steel substructures may contain fatigue-prone details. Inspectors should closely examine these details for section loss due to corrosion and cracking (see Figure 14.5.33). Refer to Chapter 7 for a detailed description of fatigue-prone details and nonredundant steel tension members (NSTMs).

Figure 14.5.33 Steel Bent

Timber substructure members are generally susceptible to a few unique types of deterioration. The inspection team should look for decay due to insects (see Figure 14.5.34). Any damage due to insects or excessive wet and dry cycles just at or above the ground line should be examined (see Figure 14.5.35). Removal of a small amount of soil at some piles can reveal decay not otherwise visible.

Figure 14.5.34 Decay on Timber Abutment

Figure 14.5.35 Timber Bent Column Decay at Ground Line

For stone masonry substructure types, the joints between individual stones should be inspected for loss of mortar (see Figure 14.5.36). Inspectors should document specific joints so that deterioration can be monitored each inspection.

Figure 14.5.36 Stone Masonry Abutment with Deteriorated Joints

For structures over water, inspectors should be aware of built-up debris (see Figure 14.5.37). Damage to the piers or bents may be present and should be documented.

Figure 14.5.37 Timber Pile Bents with Debris Accumulation

Section 14.6 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of bridge substructures. The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI)* for **component condition** ratings (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Substructure material and type, including information about the foundation, are recorded using *SNBI* Items B.SB.01 through B.SB.07.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

This page intentionally left blank.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 15 TABLE OF CONTENTS

CHAPTER 15 LIST OF FIGURES

This page intentionally left blank.

Chapter 15 Inspection and Evaluation of Culverts

Section 15.1 Introduction

As discussed in Chapter 4, a culvert is a structure comprised of one or more barrels, beneath an embankment and designed structurally to account for soil-structure interaction. These structures are hydraulically and structurally designed to convey water, sediment, debris, and, in many cases, aquatic and terrestrial organisms through roadway embankments. Culvert barrels have many sizes and shapes and have inverts that are either integral or open, i.e., supported by spread or pile-supported footings. Many culverts take advantage of headwater submergence of the inlet to increase hydraulic efficiency and economy (*SNBI* Definitions). Additionally, some culverts have shallow cover but often have high wingwalls and headwalls that can also serve as parapets.

If a structure resembling a culvert does not convey water, it should not be classified as a culvert and condition not be recorded using *SNBI* Item B.C.04 Culvert Condition Rating. Condition should be recorded using superstructure and substructure (and deck if appropriate) condition rating items.

The NBIS definition of a bridge includes culverts with extreme ends of openings measuring more than 20 ft along the centerline of the road and also includes multiple pipes where the distance between openings is less than half of the smaller contiguous opening (23 CFR 650.305). Multiple culvert barrel installations with relatively small pipes can therefore meet the NBIS definition of a bridge.

The NBIS program does not apply to structures where the total opening length is less than or equal to 20 ft. However, some type of formal inventory and inspection is recommended for culverts that are not NBIS bridge length. In some cases, the failure of a culvert or other structure with openings less than 20 ft long can present a life-threatening hazard. Many of the principles and practices outlined in this chapter can still apply for these smaller structures. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

Section 15.2 Characteristics

15.2.1 Rigid Culverts

Design Characteristics

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 conventional bridges, culverts typically have no distinction between substructure and superstructure, but in the case of box culverts there is a distinctive top slab, two or more walls, and possibly a floor that function as one frame unit. Culverts typically also have no "deck" since earth backfill usually separates the culvert structure from the riding surface (see Figure 15.2.1).

Figure 15.2.1 Rigid Culvert

Types and Shapes of Rigid Culverts

Concrete culverts are the most common type of rigid culverts used today, but masonry and timber have been constructed historically as well. Types of culverts include:

- Four-sided frame (box culverts).
- Three-sided frame culverts.
- Pipe culverts.
- Arch culverts.

Box Culverts

One of the most common rigid culverts used today is the four-sided frame, or concrete box culvert (see Figure 15.2.2). 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 typically 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 generally used in a variety of circumstances for both small and large channel openings and can be easily adaptable to a wide range of site conditions, including sites where low profile structures are necessary.

In situations where the necessary size of the opening is very large; a multi-cell box culvert can be used (see Figure 15.2.3). It is important to note that although a box culvert may have multiple cells or barrels, it is still a single structure. The internal walls are provided primarily to reduce the unsupported length of the top slab and in some cases, to prevent the bottom slab from bowing upwards.

Figure 15.2.2 Concrete Box Culvert

Figure 15.2.3 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.

Reinforced cast-in-place (CIP) concrete box culverts are typically constructed with multiple cells to accommodate longer spans. The major advantage of cast-in-place construction is that the culvert can be designed to meet the specific geometric demands 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, storm drains, and drop inlets.

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 ft (see Figure 15.2.4). Some box sections may have spans of up to 20 ft if a special design is used.

Figure 15.2.4 Precast Concrete Box Culvert under Construction

There are very few timber culverts throughout the nation. Timber culverts are generally box culverts and are constructed from individual timbers similar to railroad ties (see Figure 15.2.5). These culverts are typically utilized in areas of seasonal flows, such as heavy flow in the spring and little to no flow during the summer months.

Figure 15.2.5 Timber Box Culvert

Three-Sided Frame Culverts

Frame culverts without a bottom slab or floor are typically cast-in-place or precast reinforced concrete, which is generally shaped similar to a box culvert (see Figure 15.2.6). 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 15.2.6 Concrete Frame Culvert

Pipe Culverts

Precast concrete pipe culverts are typically manufactured in two standard shapes: circular and elliptical. Circular pipe culverts are very common. In situations where the necessary size of the opening is very large; two or more concrete pipe culverts may be used (see Figure 15.2.7).

Figure 15.2.7 Twin Concrete Pipe Culvert

The size of the opening is primarily determined by the following factors:

- Magnitude of the peak design flow.
- Expected headwater (pooled water surface) at the inlet for the peak design flow.
- Permissible barrel and outlet flow velocities.
- 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 typically used in situations where horizontal or vertical clearance is limited. The oblong shape enables the pipe to fit where a circular pipe may not, but still provide an adequate hydraulic 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, 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 typically available with standard sizes as large as 12 ft 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 may be used.

Arch Culverts

An arch culvert is a curved-shape culvert that does not have a bottom, or floor (see Figure 15.2.8). This type of culvert, as well as embedded culverts (i.e., culverts having buried inverts), are commonly and effectively used at stream crossings necessary for providing passage for aquatic life.

Figure 15.2.8 Concrete Arch Culvert

A variation of the arch culvert is the pipe arch culvert. It is the same concept as the arch culvert, which utilizes earth fill to ensure the arch is in compression, but it has an integral floor serving as a tie between the ends of the arch.

Concrete arch culverts can be cast-in-place or precast, although they are almost exclusively precast concrete today.

Culverts constructed exclusively from stone masonry were usually in the shape of an arch (see Figure 15.2.9). Stone and brick are durable, low maintenance materials, but they are typically seldom used for constructing new culvert barrels.

Figure 15.2.9 Stone Masonry Arch Culvert

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 (legs).
- Frame culverts - top slab, wall (legs), foundation and footing.
- Arch culverts - culvert barrel, foundation, and footing.
- Pipe culverts - culvert barrel.

Wingwalls and headwalls are considered primary members when present. There are no secondary members for the culvert barrels.

Steel Reinforcement

Steel reinforcement for concrete culverts is in the form of primary or secondary reinforcement. Depending upon the potential for corrosion, chemical attack, or other steel reinforcement deficiencies, states may choose to use a protective system such as an epoxy coating. Some states have also incorporated stainless steel reinforcement into concrete culverts.

The typical reinforcing steel layout for box culverts is pictured in Figure 15.2.10. The primary reinforcing steel for box culverts resists positive and negative moment tension and shear forces. Tension reinforcement is placed transversely in the box culvert slabs, outside corners, and vertically in the walls. Shear reinforcement may be placed diagonally in each of the box culvert corners. Single cell precast concrete box culverts may use steel welded wire for tension and shear reinforcement. Longitudinal temperature and shrinkage reinforcement are placed in the slabs and the walls of box culverts.

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 15.2.11).

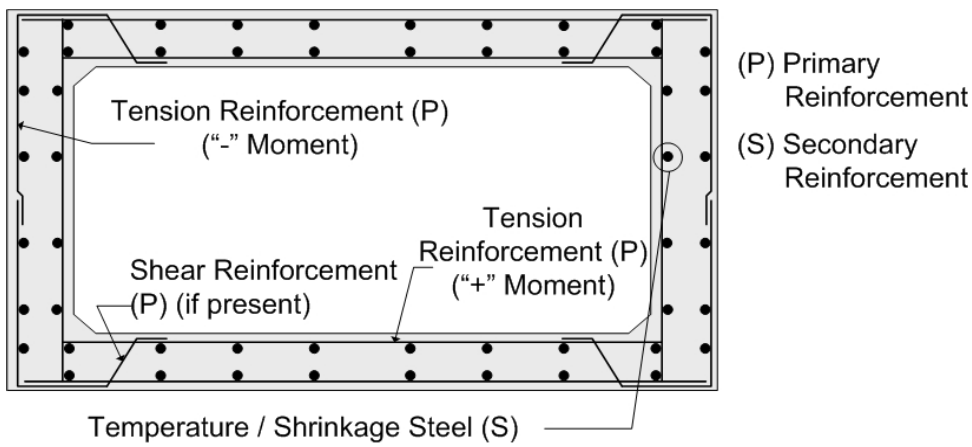


Figure 15.2.10 Steel Reinforcement in a Concrete Box Culvert

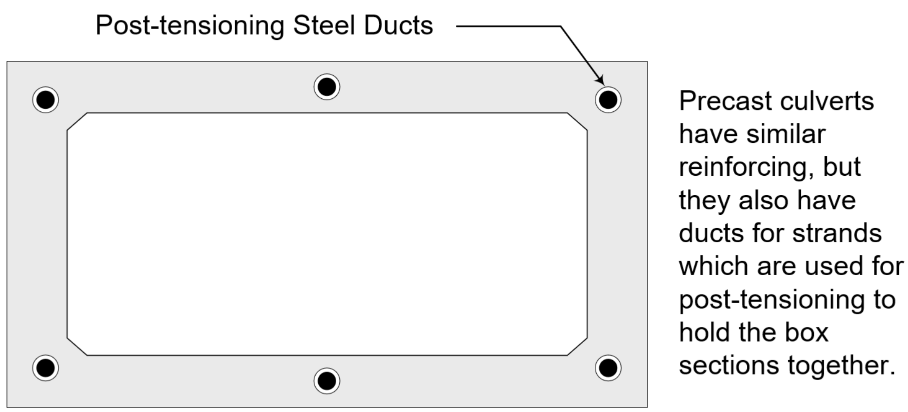


Figure 15.2.11 Precast Box Section with Post-tensioning Steel Ducts

The typical reinforcing steel layout for arch and pipe culverts is detailed in Figure 15.2.12 and Figure 15.2.13. It is very common for pipe culverts to be reinforced with steel welded wire mesh as opposed to steel bars. Primary reinforcement for arch and pipe culverts also resists tension and shear. Arch and pipe culvert primary reinforcement is placed transversely in the walls of the culverts. Secondary reinforcement for arch and pipe culverts is placed lengthwise along the barrel of the culvert, spaced uniformly from support to support.

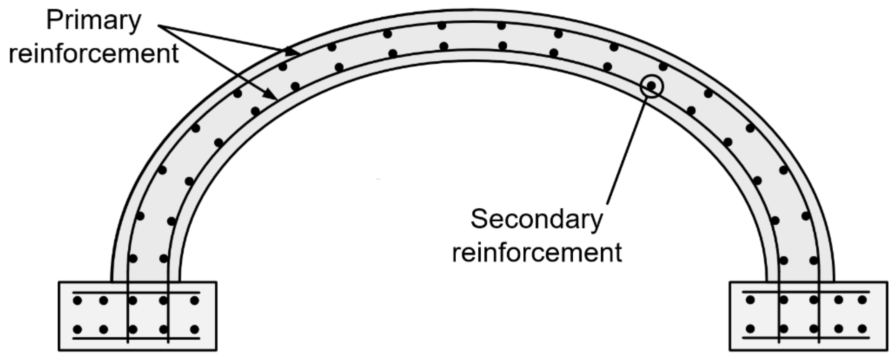


Figure 15.2.12 Steel Reinforcement in a Concrete Arch Culvert

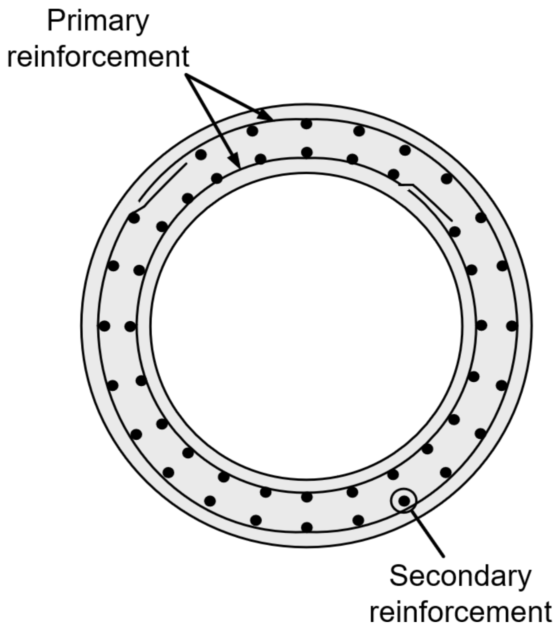


Figure 15.2.13 Steel Reinforcement in a Concrete Pipe Culvert

15.2.2 Flexible Culverts

Design Characteristics

By nature of their design, flexible culverts have little structural bending strength without proper backfill. The material from which they are made, such as corrugated galvanized steel, aluminum, or high density polyethylene (HDPE) can be flexed or bent and can be distorted significantly without cracking. Therefore, flexible culverts depend on support from the backfill to resist bending. In flexible culvert designs, proper interaction between the soil and structure is critical. Like all culverts, flexible culverts are designed hydraulically for full flow. Most flexible culverts have a circular or elliptical configuration (see Figure 15.2.14). Although less common, some flexible box and arch culverts are in use today (see Figure 15.2.15).

Figure 15.2.14 Pipe Arch Flexible Culvert

Figure 15.2.15 Flexible Box Culvert

Types and Shapes of Flexible Culverts

Flexible culverts are typically constructed from corrugated galvanized steel or aluminum pipe or field assembled structural plate products. Structural plate galvanized steel products are available as structural plate pipes, box culverts, or long span structures.

Culvert Shapes

Flexible culverts may be in a variety of shapes depending on site conditions (see Figure 15.2.16).

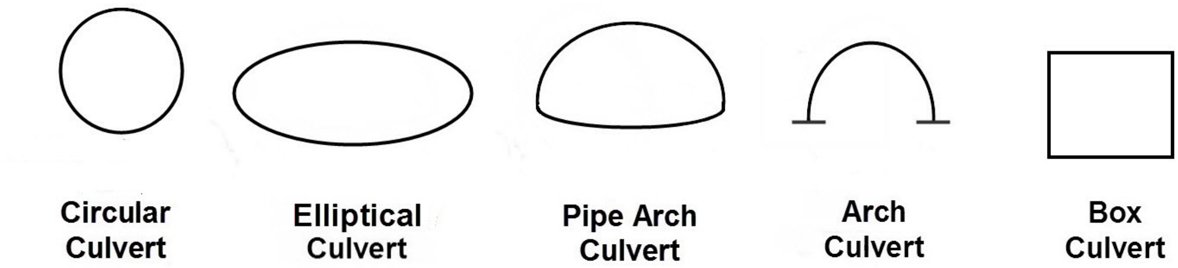


Figure 15.2.16 Corrugated Galvanized Steel Culvert Shapes

Circular culverts are generally used for smaller span applications, where the hydraulic flow may be fairly low. They are also commonly used when there is a larger embankment. The pipe diameter can range from less than a foot up to 26 ft, depending on material. Circular or round culverts may also be used next to each other.

The elliptical and pipe arch shapes are utilized most often when there is limited vertical clearance between the streambed and the roadway surface. They provide for an increased stream width with the elongated span length. Elliptical culverts can span up to 50 ft, whereas pipe arch culverts are not generally used for spans greater than 20 ft.

Arch culverts may be used in similar application to elliptical culverts. The arch shape offers less of an obstruction to the waterway than elliptical culverts or pipe arches and can be used to provide a natural stream bottom where the stream bottom is naturally erosion resistant.

Box or rectangular shaped culverts can be easily adaptable to a wide range of site conditions including sites where low profile structures may be necessary. Due to the flat sides and top, rectangular shapes are typically not as structurally efficient as other culvert shapes. Span lengths generally range from 10 to 21 ft.

Box Culverts

Corrugated steel box sections use standard corrugated galvanized steel plates with special reinforcing applied to the areas of maximum moments.

Aluminum box culverts utilize 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 typically suitable for shallow depths of fill.

Corrugated Pipe

Factory-made pipe is typically produced in two basic shapes: round and pipe arch. Both shapes are fabricated 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 galvanized steel pipe shapes are available in a wide range of standard sizes.

Structural Plate

Structural plate culverts are typically field assembled from standard corrugated plates. The plates are generally made from galvanized steel or aluminum. Standard galvanized steel plates have corrugations with a 6-inch pitch and a depth of 2 inches, whereas aluminum is field assembled with a 9-inch pitch and a depth of 2.5 inches. Plates are manufactured in a variety of thicknesses and are pre-curved for the size and shape of structure to be erected.

Structural plate pipes are available in three basic shapes:

- Round.
- Pipe arch.
- Arch.

Long Span Culverts

Long span steel culverts are assembled using conventional corrugated galvanized steel plates and longitudinal and circumferential stiffening members. There are three standard shapes for long span steel structures:

- Pipe arch.
- Low profile arch.
- High profile arch.

Each long span installation represents, to a certain extent, a custom design. The inspection team reviews the design or as-built plans when checking dimensions of existing long span structures. It is imperative that the designed shape of the culvert be maintained to retain the desired strength.

Long span aluminum structures are assembled using conventional 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 long steel spans.

Refer to manufacturer’s technical data sheets for different standard sizes and thicknesses used in flexible culvert shapes.

Plastic Culverts

Plastic culverts are most commonly made using HDPE. These round sections utilize one or more "walls" and are typically available up to 60 inches in diameter. Single-walled culverts are often corrugated on the inner and outer surfaces (see Figure 15.2.17). Dual-walled culverts have a smooth inner surface and a smooth or corrugated outer surface (see Figure 15.2.18).

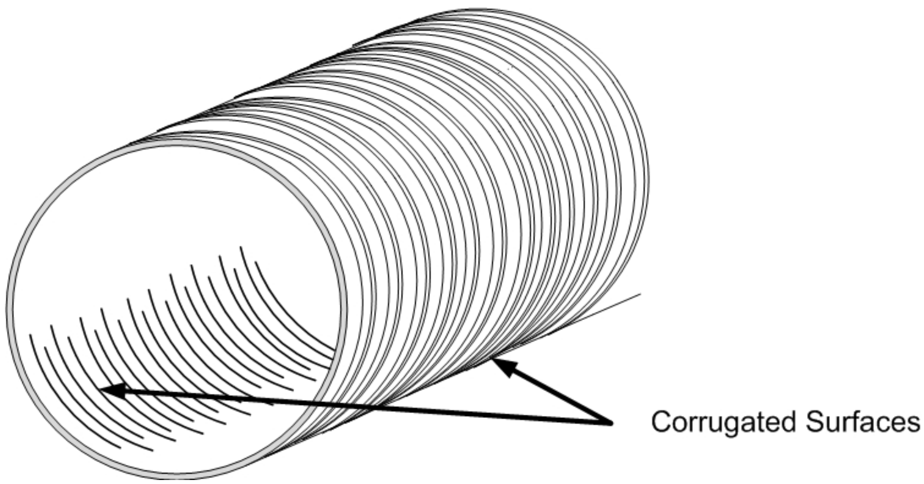


Figure 15.2.17 Schematic of a Single Walled Culvert

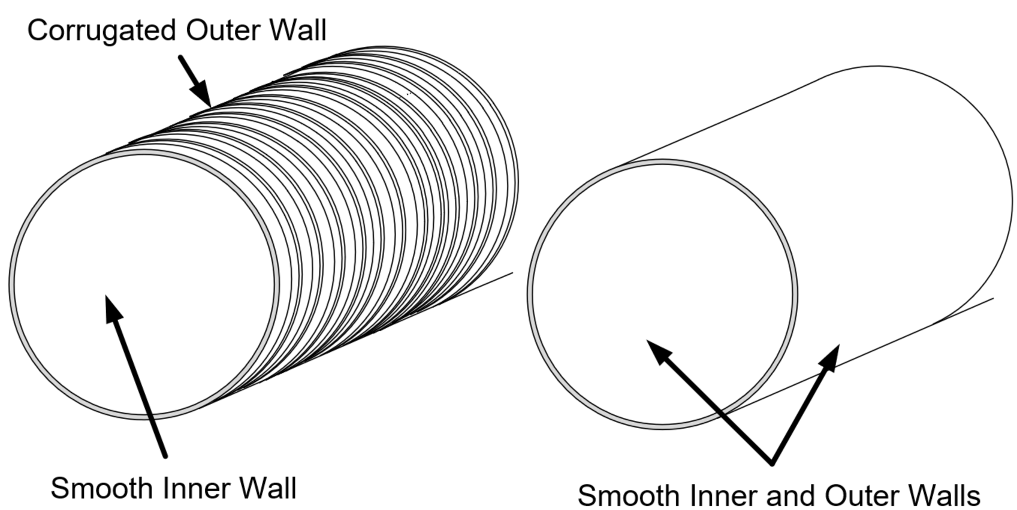


Figure 15.2.18 Schematic of Dual Walled Culverts

Plastic culverts offer several advantages over traditional corrugated metal pipe (CMP) sections:

- Strength-to-weight ratio – The favorable strength-to-weight ratio enables 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 much lighter weight compared to CMP or concrete pipe enables HDPE culverts to be installed using minimum labor and light-duty equipment. Lightweight pipe also provides a safer work environment, as 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 are not likely to rust. They also have shown good performance against corrosive chemicals, brackish water, and soil elements. Abrasion resistance is also typically greater for plastic culverts than for CMP culverts. However, plastic culverts may be 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 provides for easier placement of a curved pipeline. This flexibility can also be a concern, as the pipes may lose their shape or alignment by sagging resulting in loss of strength.

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.

15.2.3 Durability and Performance

Durability

Durability is a measure of a culvert’s ability to withstand chemical attack and abrasion. Durability problems are frequently the cause of deterioration. Metal culverts often experience corrosion, at and below the high water line (invert), that can lead to structural failure. Rigid culverts can be subject to chemical attack in strongly acidic environments such as drainage from mines or agriculture, and may also be damaged by abrasion.

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 including water depth, velocity, turbulence.
- Structural integrity – How much cover and 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 – Whether the foundation material provides 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 and lead to "piping" of water around and under the culvert. Piping can advance and cause culvert failure.
- The presence of vegetation, debris, and sedimentation buildup can greatly affect the means and efficiency of the flow through the culvert, as well as cause structural damage.
- Abrasion and corrosion caused by substances in the water, the surrounding soil or atmosphere.
- Scourability – susceptibility of the streambed to erosion and/or undermining.

Soil and Water Conditions that Affect Culverts

The chemical and physical characteristics of the soil, that come into contact with a culvert, can be analyzed to determine the potential for corrosion. The presence of base-forming 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.

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

- pH Extremes.
- Electrical Resistivity.
- Soil Characteristics.

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 acidic 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 acidic 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 typically 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, internally or externally, deviates from neutral, 7.0, it generally becomes more hostile. Concrete begins to see negative effects due to pH at around 6.5, but a pH less than 3 is the most harmful. Galvanized steel corrosion rates increase when the pH drops to 4 or below, as the protective oxide films dissolve at that point. Aluminum behaves in a similar manner to steel in that it will corrode at a similar pH. However, aluminum and steel corrode in different environments. HPDE is the most resistant to acidic water and is not negatively affected until the pH is in the range of 1.

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 likely increase electrical resistivity and should 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.

Section 15.3 Overview of Common Deficiencies

15.3.1 Concrete Deficiencies

Common deficiencies that may occur on precast or CIP reinforced concrete culverts include:

- Cracking (structural).
- Cracking (non-structural).
- Scaling.
- Delamination.
- Spalling.
- Chloride contamination.
- Freeze-thaw.
- Efflorescence.
- Alkali silica reactivity (ASR).
- Ettringite formation.
- Honeycombs.
- Pop-outs.
- Wear.
- Impact damage.
- Abrasion.
- Reinforcing steel corrosion.

Refer to Chapter 7 for a detailed explanation of the properties of concrete, types, and causes of concrete deficiencies.

15.3.2 Steel Deficiencies

Common potential deficiencies that may occur on galvanized steel flexible culverts include:

- Corrosion.
- Overloads.
- Impact damage.
- Heat damage.
- Coating failures.
- Embankment erosion at culvert entrance and exit.
- Roadway settlement.
- Loose, missing, or deteriorated fasteners.
- Distortion.

It is generally important to be aware of damage that may have occurred during installation or handling. These can be considered pre-existing deficiencies and not deterioration caused by in-service conditions.

Deficiencies for aluminum culverts are generally similar. Refer to Chapter 7 for a detailed explanation of the properties of steel, types, and causes of steel deficiencies.

15.3.3 Timber Deficiencies

Common potential deficiencies that may occur on timber culverts include:

- Inherent deficiencies – Checks, splits, shakes.
- Decay by fungi.
- Damage by insects and borers.
- Loose connections.
- Loose or deteriorated stressing rods.
- Surface depressions.
- Damage from fire.
- Impact damage.
- Damage from wear, abrasion, and mechanical wear.
- Damage from overstress.
- Weathering/warping.
- Failure of protective system.
- Chemical attack.

Refer to Chapter 7 for a detailed explanation of the properties of timber, types, and causes of timber deficiencies.

15.3.4 Masonry Deficiencies

Common potential deficiencies that may occur on masonry culverts include:

- Missing stones or mortar.
- Weathering.
- Spalling.
- Splitting.

Refer to Chapter 7 for a detailed explanation of the properties of masonry, types, and causes of masonry deficiencies.

Section 15.4 Inspection Methods

15.4.1 Visual and Physical Inspection

A logical sequence for inspecting culverts helps ensure that a thorough and complete inspection should be conducted. In addition to the culvert components, inspectors should 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 typically similar to the inspection of bridges.

The following sequence is applicable to typical culvert inspections:

- Overall site condition should be observed.
- The inspection team should look at the roadway at approaches and above culvert.
- The inspection team should look at the embankment adjacent to culvert and roadway.
- The inspection team should view the waterway from above pipe and at inlet/outlet (refer to Chapter 16).
- End treatments should be inspected.
- Culvert barrel geometry and alignment should be observed.
- The culvert barrel should be inspected.

For typical installations, it is usually beneficial to begin the field inspection with general observations of the overall condition of the structure and inspection of the supported roadway. These initial observations familiarize the inspection team with the structure so they may note any global issues. They may also point out anything that would demand a modification to the inspection sequence or indicate areas needing special observation. Inspectors should remain observant for changes in the drainage area that might affect runoff characteristics and hydraulic analyses.

The inspectors should select one end of the culvert and inspect the embankment, waterway, headwalls, wingwalls, and culvert barrel. Inspectors should then progress toward the other end of the culvert. The full length of the culvert should be inspected from the inside, if of sufficient size to safely enter. Culverts with small diameters can be inspected by looking through the culvert from both ends or by using a small movable camera. All sections of the culvert barrel including walls, floor, top slab, and joints should be visually. Inspectors should view the culvert barrels for deficiencies such as misalignment, joint deficiencies, cracking, spalling, section loss, and other material deficiencies. It is important to plan the inspection, if possible, so that water levels are low.

It should be noted that if a culvert is considered a confined space, the necessary precautions should be taken to enter the space. Refer to Chapter 2 for details on confined space inspections and safety measures.

Inspection methods for rigid culverts can vary by material. Visual activities include the inspection of concrete culverts for cracks, spalls, and other deficiencies. The inspection of masonry culverts for cracks, lose or missing mortar, vegetation, water seepage, crushing, missing stones, bulging, and misalignment is primarily a visual activity. Timber culverts should be inspected for checks, splits, shakes, decay, deflection, and loose fasteners.

Hammer sounding of the exposed concrete or masonry may be performed to determine areas of delamination. Sounding of the exposed timber may be performed to determine areas of internal decay.

Most deficiencies in flexible culverts are first detected by visual inspection. In general, corrugated metal culvert barrels should be inspected for cross-sectional shape and barrel deficiencies such as joint deficiencies (exfiltration or infiltration through joints or joint misalignment), seam deficiencies (exfiltration 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 typically at the 2 o'clock and 10 o'clock locations. An inward bulge at these locations may indicate potential failure of the structure. Sometimes surveying the culvert may be necessary to determine if there is any shape distortion, and if there is distortion or differential settlement.

Flexible culverts are inspected with physical methods that are atypical to other structure types. 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 may deform, or the hammer might break through the culvert wall if significant section loss exists. For aluminum structural plate, the bolts can be checked with a wrench.

For the structural plates, inspectors should look for section loss. This is typically achieved by using a wire brush, grinder, or a hammer to remove loose or flaked steel and then the remaining section should be measured, documented, and compared to a similar section with no loss.

Any minor deterioration or abrasion should be noted. Severe surface deterioration should be documented as a potential candidate for maintenance. When the invert is completely deteriorated (see Figure 15.4.1), it may be considered a critical finding. It should be noted in the report when linings are used to protect against chemical attack or abrasion. The condition of the lining, if present, should also be documented.

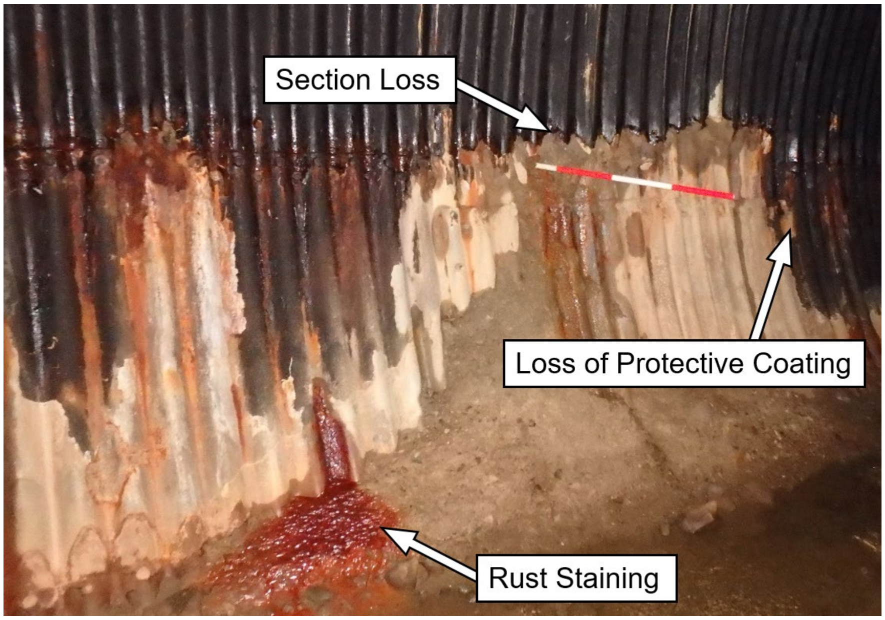


Figure 15.4.1 Advanced Deterioration of a Flexible Culvert Section

15.4.2 Advanced Inspection

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

There are several advanced methods for concrete and masonry inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Electrical Resistivity (ER).
- Galvanostatic Pulse Measurement (GPM).
- Ground Penetrating Radar (GPR).
- Half-Cell Potential (HCP).
- Impact Echo (IE).
- Infrared Thermography (IT).
- Linear Polarization (LPR).
- Magnetic Flux Leakage (MFL).
- Magnetometer (MM).
- Rebound and penetration methods.
- Ultrasonic Pulse Velocity (UPV).
- Ultrasonic Pulse Echo (UPE).
- Ultrasonic Surface Waves (USW).

Other methods that may be considered destructive testing methods include the following.

- Concrete cores.
- Borescopes.
- Moisture content.
- Petrographic examination.
- Alkali-Silica Reaction (ASR) evaluation.
- Reinforcing steel sample.
- Rapid chloride permeability testing.

There are several advanced methods for steel inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Acoustic Emission (AE).
- Crack Propagation Gage (CPG).
- Dye Penetrant Testing (PT).
- Eddy Current Testing (ET).
- Magnetic Particle Testing (MT).
- Magnetic Flux Leakage (MFL).
- Ultrasonic Testing (UT).
- Phased Array Ultrasonic Testing (PAUT).

Other methods that may be considered destructive testing methods include the following.

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

There are several advanced methods for timber inspection that may be performed in the field. The methods that can be completed by a bridge inspector or certified NDE technician include the following.

- Stress wave velocity.
- Ultrasonic testing.
- Vibration.

Other methods that may be considered destructive testing methods include the following.

- Drilling and probing.
- Moisture meter.

Advanced methods for common bridge materials are described in detail in Chapter 17.

Section 15.5 Inspection Areas and Deficiencies

To ensure that a culvert is functioning safely, the inspection team should evaluate the structural integrity, hydraulic performance, and roadside compatibility of the culvert.

- Structural Integrity – The identification of potential structural and material problems necessitates a careful evaluation of indirect evidence of structural distress as well as actual deterioration and distress in the culvert material.
- Hydraulic Performance – 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 – Headwalls and wingwalls higher than the road or embankment surface may constitute a fixed obstacle or hazard. Headwalls and wingwalls are presented in detail in Section 15.5.12. Abrupt drop-offs over the end of a culvert or steep embankments may represent rollover hazards to vehicles that leave the roadway.

15.5.1 Areas Subjected to Movement and Misalignment

Culvert movement or misalignment is generally checked by visual observation. Inspectors should look for sagging, cracking, or separation of joints. Sags can be detected during low flows by looking for areas where the water is deeper or where sediment has been deposited and can often be viewed by sighting along the top inside surface of the barrel. 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 may be to take profile elevations of the top of the culvert. Inspectors should also check horizontal alignment for bulging or straightness (see Figure 15.5.1). Smooth curvature in culverts that were constructed with a curved alignment should be observed. This can be checked by sighting along the walls and by examining joints for differential movement.

Figure 15.5.1 Sighting Along Culvert to Check Alignment

Alignment problems may also be caused by improper installation, undermining, or differential settlement of the fill. It is generally important to determine which of these problems may be causing the settlement or misalignment. Also, the inspection team should try to determine whether the undermining is due to piping (loss of fill from underneath the culvert), water exfiltration, or infiltration of backfill material. Inspectors should look for holes in the downstream side embankment. If the misalignment is due to improper installation or differential settlement, repeat inspections may be necessary to determine if the settlement is progressing or if it has stabilized.

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 that varies in magnitude based upon the culvert length. This may cause cracking of the culvert or separation of the joints. Common causes of vertical movement are soil bearing failure, consolidation of soil, scour, undermining, and subsidence from mining or solution cavities. Movement can also be caused by the live loads directly above a portion of the culvert, if the culvert is under designed or overloaded. Deficiencies and areas to inspect for vertical movement may include the following:

- Roadway and railing for evidence of settlement.
- Existing and new cracks in the roadway pavement or concrete.
- Cracking, scour, and undermining around the culvert footing, foundation, or wingwall.

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 can be slope failure, seepage, changes in soil characteristics (i.e., frost and ice), and time consolidation of the original soil. Deficiencies and areas to inspect for lateral movement may include the following:

- General alignment.
- Settled approach roadway pavement.
- Clogged drain or weep holes.

Rotational Movement

Rotational movement of the culvert is generally the result of unsymmetrical settlements or lateral movements due to horizontal earth pressure. Common causes can include undermining, scour, saturation of backfill, and improper design. Deficiencies and areas to inspect for rotational movement may include the following:

- Vertical alignment of the walls.
- Clogged drains or weep holes.
- Cracks in the culvert headwalls or wingwalls (see Figure 15.5.2).

Figure 15.5.2 Cracking of Culvert End Treatment Due to Foundation Rotation

15.5.2 Distress

Cracks

Concrete culverts often experience cracking. Because concrete is weak in tension, distress causing hairline longitudinal cracks in the crown or invert is an indication that the reinforcing steel has accepted part of the load. Longitudinal cracking 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 most likely be concentrated along the bottom of the pipe and may result in flexure or shear cracking (see Figure 15.5.3).

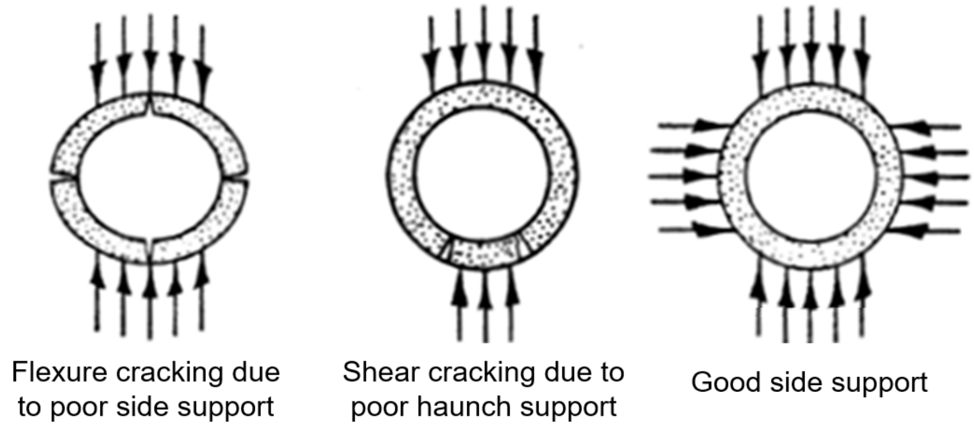


Figure 15.5.3 Longitudinal Cracks in Pipe Culvert

Transverse cracks may also be caused by improperly prepared bedding (see Figure 15.5.4). Properly prepared bedding evenly distributes the load, avoiding acute areas of higher loading (see Figure 15.5.5).

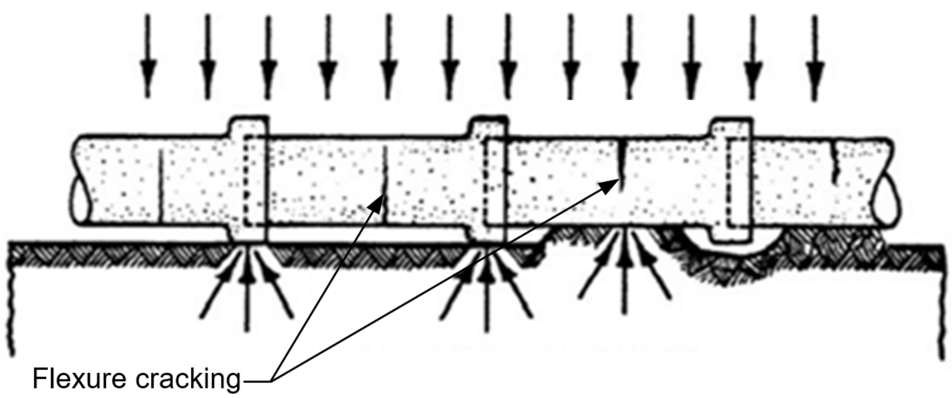


Figure 15.5.4 Transverse Cracks in Pipe Culvert

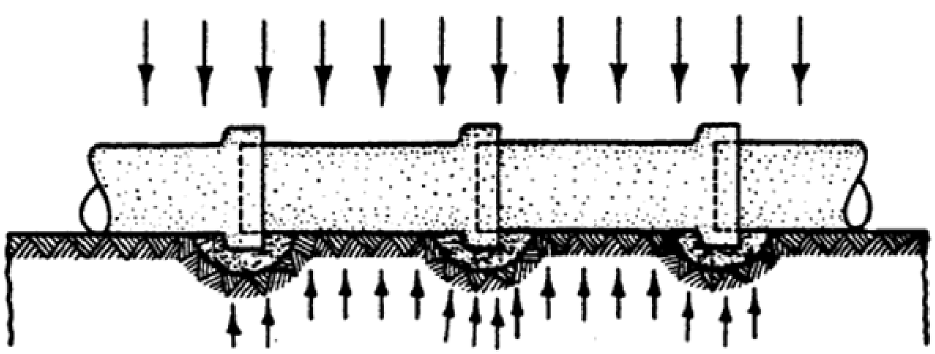


Figure 15.5.5 Properly Prepared Bedding

Cracks can occur across the bottom of the pipe 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 the pipe 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.

Other signs of distress include efflorescence, spalling, or rust stains.

Spalls

In concrete culverts, spalls often occur along the edges of longitudinal or transverse cracks, when the crack is due to overloading or poor support rather than simple tension cracking. Spalling and delamination may also be caused by the corrosion of the steel reinforcement when water is able to reach the steel through cracks or shallow cover (see Figure 15.5.6). Spalling may be detected by visual examination of the concrete along the edges of cracks

Figure 15.5.6 Spalls and Delaminations on Top Slab of Concrete Box Culvert

Distortion

The delamination of the inner concrete cover within the culvert, sometimes referred to as shear-slabbing or slab shear, refers to a radial failure of the concrete that occurs from straightening of the reinforcement cage due to excessive deflection. This is typically characterized by large slabs of concrete "peeling" away from the sides of the pipe and a straightening of the reinforcing steel. Slabbing may be a severe problem that can occur under high fills. Reference the *Manual for Bridge Element Inspection, 2nd Edition* for more information.

15.5.3 Bearing Areas

Bearing zones for rigid culverts are typically where the footing is supported by the earth. For concrete and masonry culverts, inspectors should look for cracking and spalling. Metal culverts should be examined for buckling and distortion. In timber culverts, inspectors should look for crushing.

15.5.4 Shear Zones

Horizontal and vertical forces can cause high shear zones at the intersection of culvert walls and slabs. For concrete and masonry culverts, inspectors should look for diagonal cracking. Metal culverts should be examined for buckling and distortion. In timber culverts, inspectors should examine for splitting.

15.5.5 Flexural Zones

High flexural moments are typically caused by horizontal and vertical forces that occur at the tops or sides of culverts. These moments cause compression and tension depending on the load type and location of the neutral axis. Inspectors should look for deficiencies caused by overstress due to compression or tension caused by flexural moments. Compression areas should be checked for splitting, crushing, or buckling. Tension areas should be inspected for cracking or distortion. High embankments may impose very high permanent loads on all sides of a culvert and can cause bending failure (see Figure 15.5.7).

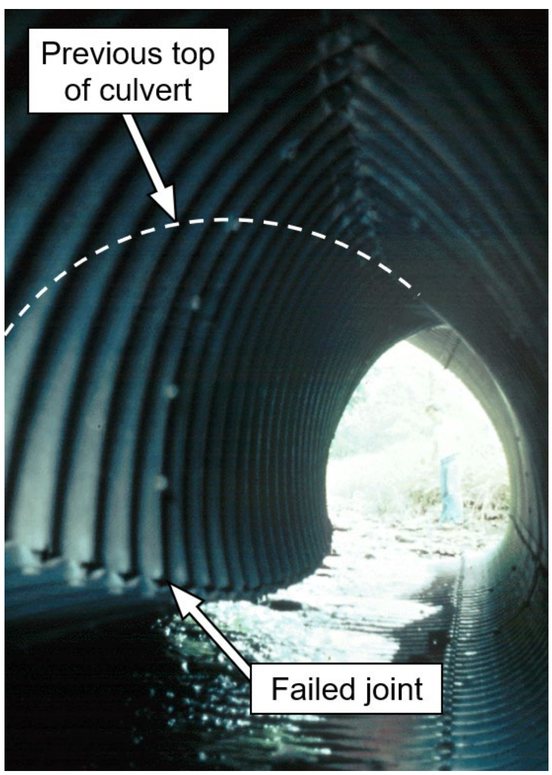


Figure 15.5.7 **Flexible Pipe Culvert Bending Failure**

15.5.6 Areas Exposed to Drainage

For concrete culverts, inspectors should examine for spalling, delamination, and exposed rebar. Concrete culvert headwalls and wingwalls should also be inspected, since these areas are often exposed to surface drainage carrying road salts, which chemically attack and can damage the walls. In metal culverts, the inspectors should look for signs of corrosion, especially in the vicinity of the joints. In masonry culverts, the inspectors should 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 15.5.8). Areas that are exposed to drainage should be examined for decay on timber culverts.

Figure 15.5.8 Drainage Through Missing Stones in Masonry Culvert

15.5.7 Areas Exposed to Traffic

Collision damage from vehicles passing adjacent to the culvert should be checked, typically when tall headwalls also function as curbs or parapets.

Damage to concrete culverts and headwalls may include spalls and exposed reinforcement and possibly steel reinforcement section loss. Distortion is the typical deficiency experienced in metal members struck by traffic. Damage to timber culverts includes split or broken members.

15.5.8 Areas Previously Repaired

It is typically common for flexible culverts to be repaired or rehabilitated with structural plate sections or with structural invert paving by using reinforced concrete or asphalt. The invert paving should be inspected for deficiencies such as surface cracks, spalls, abrasion, and other deficiencies. Structural plates can be visually inspected for deficiencies to those discussed previously for metal. Old pipe culverts may be entirely lined with concrete or similar material. Inspectors should check to ensure the lining is still adhering to the original pipe. Any concrete spall repairs within concrete box culverts or on headwalls, wingwalls, and aprons should be inspected for cracking or delamination.

15.5.9 Roadway and Embankment

Inspection of the roadway and embankment should include an evaluation of the functional adequacy.

Deficiencies in the roadway and embankment may be indicators of possible structural or hydraulic problems in the culvert. Inspect the 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.

Roadways should be examined 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 roadway pavement may be an indication of current problems such as excessive differential settlement, culvert movement, scour causing loss of backfill material, or other erosion (see Figure 15.5.9). It is generally advisable for the inspection team to have previous inspection reports that may indicate the age of the present overlay and the roadway condition before the placement of new roadway material.

Figure 15.5.9 Repaired Roadway Over a Culvert

It is typically important to note that not all deficiencies in approach roadways have an adverse effect on the culvert. Deterioration of the approach roadway pavement may be due to excessive traffic and not caused by problems at the culvert.

15.5.10 Scour and Undermining

Scour can cause undermining or the removal of supporting foundation material from beneath the culvert (see Figure 15.5.10). Refer to Chapter 16 for a more detailed description of scour and undermining.

Figure 15.5.10 Scour and Undermining at Culvert Inlet

Inspection for scour includes probing around the culvert inlet and outlet for signs of undermining or piping. Sometimes silt loosely fills in a scour hole and offers no protection or bearing capacity for the culvert inlet and outlet. Inspectors should also check culvert frame footings for these conditions along the length of the culvert.

15.5.11 Joints

Expansion joints should be inspected to verify that the filler material or joint sealant is in place and that the joint is not filled with incompressible material which would prohibit expansion.

When inspecting a joint in a culvert, inspectors should be sure to check for the following deficiencies:

- Separated joint.
- Cracks.
- Exfiltration.
- Infiltration.

Joint inspection also identifies any joints that are opened widely or are not open to uniform width. Joint separations are significant because they indicate signs of exfiltration and infiltration. They are typically noted when misalignment is observed. Longitudinal movement of the soil in the general direction of the culvert's centerline could cause the sections to pull apart at the joints. The slippage of the embankment may also cause a joint separation to occur.

Exfiltration occurs when leaking joints enable 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, a serious misalignment of the culvert or failure may occur. When the leakage is severe enough, it can create piping, which occurs when the water creates its own path to the culvert outlet. Leaking joints may be detected during low flows visually and by checking around the ends of the culvert for piping. Serious piping can result in the collapse of the approach roadway or the culvert itself.

Infiltration occurs when water is flowing or seeping into the culvert through open or loose joints (see Figure 15.5.11), which may enable supporting soil into the culvert. Infiltration occurs when the water is higher than the culvert inlet, with the water seeping through the fill material and into the culvert joints. 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.

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.

Figure 15.5.11 Precast Concrete Box Culvert Joint with Infiltration

15.5.12 End Treatments

Rigid Culverts

The most common types of box culvert end treatments are skewed ends and headwalls.

Both end treatment types may utilize wingwalls to retain the embankment around the opening (see Figure 15.5.12 and Figure 15.5.13).

Wingwalls should be inspected to ensure they are in proper vertical alignment (see Figure 15.5.12). Wingwalls may be tilted due to settlement, slides, scour, or undermining (see Figure 15.5.13). Cracking, tipping of wingwalls or headwalls or separation of culvert barrel from the headwall and wingwalls can be evidence of undermining (see Figure 15.5.14). Refer to Chapter 14 for a detailed description of deficiencies and inspection procedures of wingwalls.

Figure 15.5.12 Tilted Wingwall

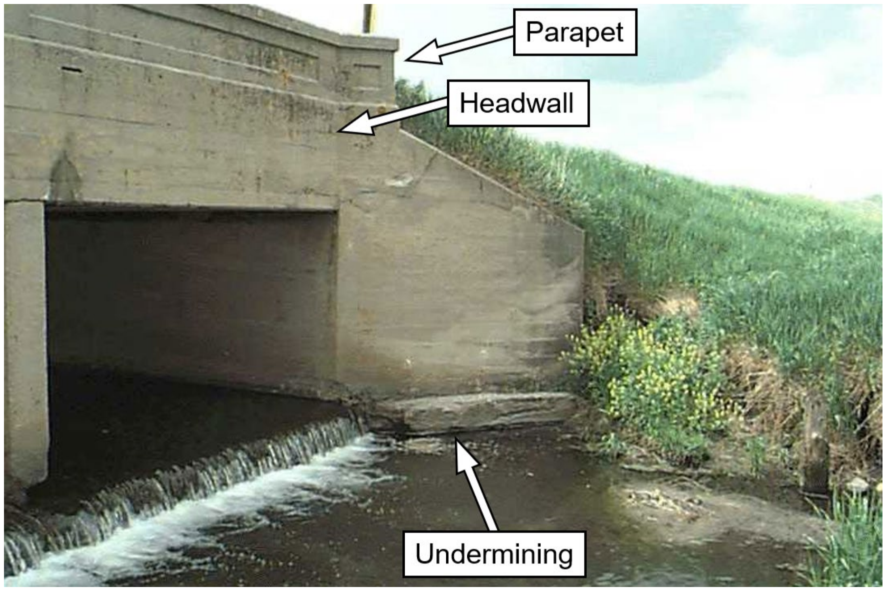


Figure 15.5.13 Cast-in-Place Concrete Headwall with Undermined Wingwall

Figure 15.5.14 Culvert Wingwalls with Exposed Footings

Skewed headwalls help support the top slab of a box culvert (see Figure 15.5.15). Stresses increase because a full box shape is not present at the end. The reinforcement is often designed to account for the increased stress due to the skew, but it is good practice to look for signs of distress caused by excessive flexure or bending in both the end wall and top slab of the culvert.

Figure 15.5.15 Skewed End

The embankment around the culvert entrance and exit should be inspected for damage from erosion or scour, including slide failure (see Figure 15.5.16).

Figure 15.5.16 Slope Failure

Other typical appurtenance structures include aprons (see Figure 15.5.17) and energy dissipators.

Aprons should be checked for undermining or settlement. Inspectors should also check the joints between the apron and wingwalls to observe if they are watertight. Piping may occur if water flows under the apron.

Figure 15.5.17 Concrete Apron

Energy dissipators may include stilling basins, rip-rap, or other devices (see Figure 15.5.18). Energy dissipaters should be inspected for material deficiencies, settlement, undermining, and overall effectiveness.

Figure 15.5.18 Energy Dissipater System

Flexible Culverts

The ends of flexible culverts are unstable, both the culvert itself and the embankment surrounding it. For this reason, end treatments are necessary. The most common types of end treatments for flexible culverts include:

- Projections (see Figure 15.5.19 and Figure 15.5.20).
- Miters (see Figure 15.5.19).
- Pipe end sections.
- Headwalls (see Figure 15.5.21).

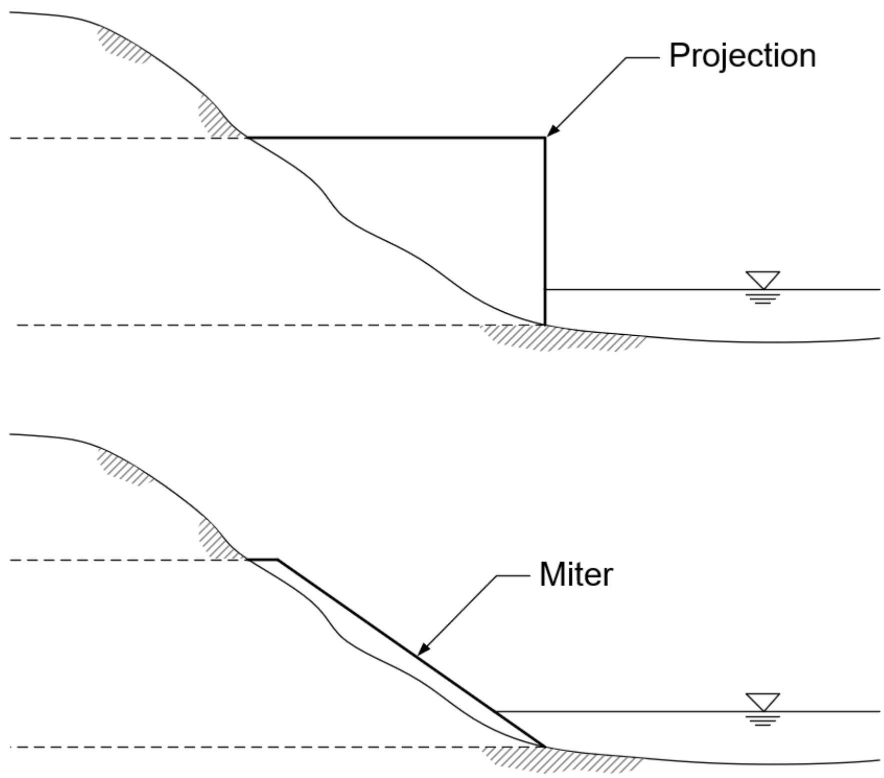


Figure 15.5.19 Projected and Mitered End Treatments

Figure 15.5.20 Flexible Culvert Projection

Figure 15.5.21 Flexible Culvert with Concrete Headwall

For projections, the location and extent of any scour or undermining around the culvert ends should be indicated. In low flow conditions scour holes tend to fill up with debris or sediment.

End treatments should be inspected 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 piping can lead to the culvert end being unsupported (see Figure 15.5.22). If not repaired in time, piping can cause cantilevered end portions of the culvert to bend down or break.

Figure 15.5.22 Unsupported Mitered Culvert End

Inspection items for mitered ends are the same as for projecting ends. Mitering the end of corrugated pipe culvert reduces its structural capacity.

Pipe end sections are typically used on relatively smaller culverts. For inspection purposes, the pipe end section should be treated similar to a projection.

Debris and Vegetation

The relatively small diameters of pipe culverts tend to produce debris accumulation. Branches, sediment, and trash can often be trapped at the culvert inlet restricting the channel flow and potentially causing scour (see Figure 15.5.23). Debris or vegetation may also build up at the outlet or within the culvert.

Figure 15.5.23 Debris and Sediment Build-up

Section 15.6 Evaluation

In evaluating inspection findings, part of the evaluation process is the determination of condition ratings for the bridge components and elements. Rating systems have been developed to aid in the inspection of culverts. The two major rating systems, that are used to collect data for the National Bridge Inventory are:

- *FHWA's Specifications for the National Bridge Inventory (SNBI)* for component **condition level** rating (23 CFR 650.317(b)(1)).
- *AASHTO Manual for Bridge Element Inspection (MBEI)* for **element level** condition state assessment (23 CFR 650.317(a)(1-4)).

The FHWA *SNBI* can be found at:
https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf.

Types of culverts are coded in the *SNBI* under Item B.SP.06 Span Type. When using Item B.C.04 Culvert Condition Rating, Item B.SP.06 should be coded for the applicable culvert type.

As per FHWA guidance, inspectors should consider the deficiencies in integral wingwalls and headwalls to the first construction or expansion joint when determining the component condition rating or the element condition state of the culvert.

Refer to Chapters 3 and 18 for detailed information on component and element level evaluation.

This page intentionally left blank.

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 16 TABLE OF CONTENTS

CHAPTER 16 LIST OF FIGURES

This page intentionally left blank.

Chapter 16 Inspection and Evaluation of Waterways

Section 16.1 Inspection of Waterways

16.1.1 Introduction

It is generally important for bridge inspectors to correctly identify and assess waterway deficiencies when performing a bridge waterway inspection. Accurate bridge waterway inspections can be vital for the safety of the motoring public. For this to happen, a thorough understanding of the different types of waterway members and deficiencies, as well as the various inspection techniques, is necessary. Refer to Chapter 4 for detailed descriptions of basic waterway characteristics. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

16.1.2 Waterway Performance Factors

There are various typical factors which determine the performance of the waterway and its potential effect on the bridge. These include, but are not limited to:

- Waterway alignment.
- Streamflow velocity.
- Hydraulic opening.
- Streambed material.
- Substructure shape.
- Foundation type.

Waterway Alignment

In general, bridges are typically designed so that the flow passes through the waterway parallel to the axis 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 potential will most likely 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 members supporting the structure.

Any change in direction of the approach of the flow to the bridge and any change in the angle at which the flow hits or impinges on the abutments and piers should be carefully noted; both under normal flow, and under flooding conditions when observed. Also, inspectors should make observations of local change in flow directions and surveys of changes in bed and bank elevations. Evaluation of aerial photographs over time can be extremely useful in assessing changes in waterway alignment. This information may be utilized to rate the severity of increasing misalignment in the flow on bridge safety.

If, for example, 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 larger floods that will most likely determine the magnitude of local scour.

Streamflow Velocity

Streamflow velocity is a major factor in the rate and depth of scour at bridges. During flood events, the streamflow velocity is increased, which may produce accelerated scour rates and depths. At high streamflow velocities, bridge foundations have the greatest chance to become undermined.

The streamflow velocity depends on many variables. One of these variables is the stream grade. A steep stream grade will likely produce high streamflow velocities. 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

For a complete bridge site inspection, the adequacy of the hydraulic opening (the cross-sectional area under the bridge) should be considered 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 collecting periodic channel cross-sections, measured along the bridge rails, and compared to cross-sections taken during subsequent bridge inspections. A weighted tape measure is typically used to accomplish this, and the lateral location of these railing points is documented so that as subsequent inspections are conducted, the points can be repeated to maintain consistency. It is helpful to display these multi-year results on a chart or graph for easy reference and tracking of any streambed movement. Photographs from key locations should also be used to document alignment changes, 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 erodibility of streambed material involves both the hydraulic conditions that create erosive forces, and the properties of the geomaterials to resist erosion when exposed to those conditions. One of the most important characteristics of geomaterials is the erosion threshold (or critical shear stress). Below this threshold, hydraulic conditions are typically such that erosion does not occur, whereas above this threshold, erosion occurs at rates that increase as the hydraulic conditions become more and more severe. The resistance provided by the geomaterials generally increase from sands to clays to rock. However, there may be many varying degrees of resistance within types of clays. The same is true for varying degrees of resistance provided by rock as affected by their material type, weathering, discontinuities, and other characteristics.

The size, gradation, cohesion, and configuration of the streambed material all affect scour rates. When comparing sands and cohesive soils, such as clays, sandy soils are typically eroded particle by particle. Scour occurs relatively rapidly such that the maximum (equilibrium) scour depth is reached within a time period of a few hours or days, often within the duration of a single flood event. Cohesive soils can erode particle by particle but also block by block of particles due to electromagnetic forces within the soils. One major difference is that the erosion of cohesive materials is typically measured in millimeters per hour, and includes the effects of many flood events over many years to reach the ultimate scour depth. For these reasons, the streambed type is important to be correctly evaluated by the bridge inspection team. Streambed rates of scour for different types of material are described in Section 16.1.8.

Substructure Shape

Substructure members on old bridges were not necessarily designed to consider their effects on scour. Wide piers and piers that might not align with the flow of the stream can contribute to an increase in the depth of scour. Due to increased awareness of bridge scour, new bridge substructure members are typically designed to enable the stream to pass through with as little resistance as possible. Many newer piers have rounded or pointed noses, which may decrease the scour depth by up to 20 percent.

Foundation Type

Footings that are undermined but founded on piles are typically not as critical as spread footings on soils that are undermined. Determination of the substructure foundation type is important 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 necessary to determine the foundation type and the bottom of the footing elevation.

16.1.3 Effects of Waterway Deficiencies

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.

Material Deficiencies

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, consider the identification of material defects of any hydraulic countermeasures including revetments, spur dikes, weirs, etc. A loss of quality and quantity of materials necessary to provide bridge safety may occur in a variety of ways. The changes in condition and integrity of materials should be recorded in the inspection report. Changes over time can be compared and any decision concerning maintenance requirements or replacement becomes more straightforward with historic information available.

Refer to Chapter 7 for further information on material deficiencies.

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 16.1.1 and Figure 16.1.2). Excessive scour often produces undermining of both piers and abutments. Such undermining is a serious condition, which may require immediate correction to assure the stability of the substructure unit. Undermining is especially serious for spread footings on soils, 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 complete failure of the substructure unit.

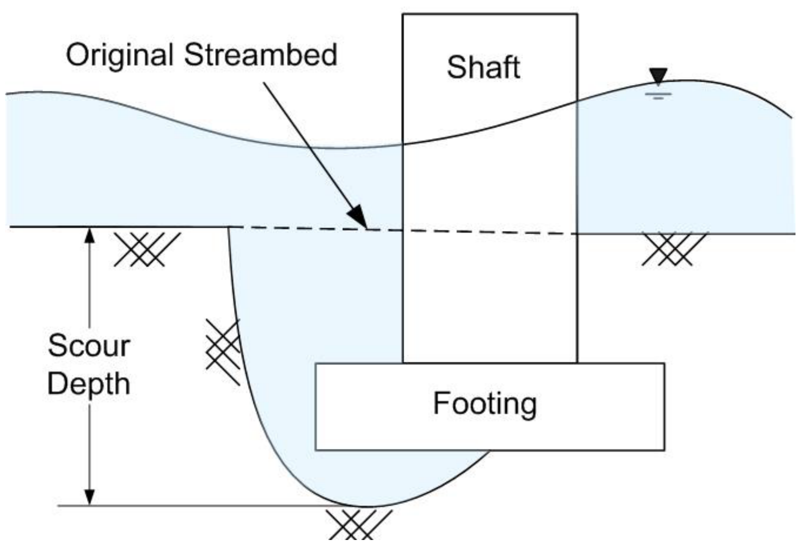


Figure 16.1.1 End View of Scour and Undermining

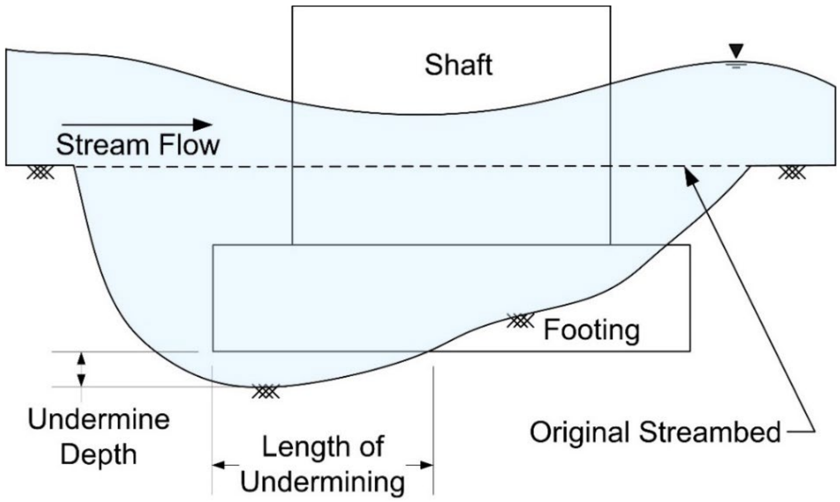


Figure 16.1.2 Side View of Scour and Undermining

The undermining of structural members is an advanced progression of scour. It is essential to determine whether undermining has already occurred. Address discovery of undermining immediately as a critical finding (if applicable) 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 in the receding leg of the flood event after undermining has occurred. In channels with low sediment supply and a bed formed of coarse rock, it may be possible to inspect the footings. During periods of low flow, the fine sediments generally do not backfill the coarse rock and access to the footings is maintained.

For areas not accessible to effective probing from above water, it is essential to employ underwater inspection techniques utilizing divers. Whenever possible, inspectors should take detailed measurements, showing the height, width, and penetration depth of the undermined cavities. Refer to Section 16.2 for a more detailed description of underwater inspections.

Settlement

Local scour and undermining are typically most severe at the upstream end of the substructure and, if not corrected, may result in differential settlement (see Figure 16.1.3).

Figure 16.1.3 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 slowly over a period of time, or it may be a very rapid process without warning during a flood event. Hydraulic countermeasures may be installed to mitigate erosion and scour of the stream channel or along the bridge piers and abutments. Refer to Chapter 4 for more details on hydraulic countermeasures.

16.1.4 Waterway Deficiencies

Total Scour

The most common bridge waterway deficiency is scour, which may adversely impact bridge substructure units. Scour is the erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges.

There are three forms of scour considered in determining total scour:

- Long-term degradation.
- Contraction scour.
- Local scour.

Long-Term Degradation

Aggradation and degradation are typically long-term streambed elevation changes. Aggradation is the general and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition (see Figure 16.1.4). It is important to report aggradation in the inspection report as it often results in increased overtopping of the roadway approaches. However, aggradation is not considered one of the components of 'total scour'. 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 16.1.5).

Degradation may be a result of the natural erosion and downcutting process which rivers experience through the years. This scour type may be accelerated by natural cutoffs in a meandering river, which steepens the channel profile, 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.

Figure 16.1.4 Streambed Aggradation

Figure 16.1.5 Streambed Degradation

Headcut migration (see Figure 16.1.6) is the degradation of the channel that is associated with abrupt changes in the bed elevation and then migrates upstream. Headcutting can occur in cohesive and non-cohesive streambed materials but is most visually evident in cohesive materials.

Figure 16.1.6 Headcut Migration

Contraction Scour

Contraction scour results from the acceleration of flow due to a natural contraction, a bridge contraction, or both (see Figure 16.1.7 and Figure 16.1.8). Many substructure units within the waterway can also restrict the stream flow (see Figure 16.1.9). When the available area for stream flow at the bridge is reduced compared with the available area upstream from the bridge, velocity will most likely 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.

<u>Continuity Equation:</u>

$$Q_1 = Q_2$$

$$A_1V_1 = A_2V_2$$

Q = flow

A = area

V = velocity

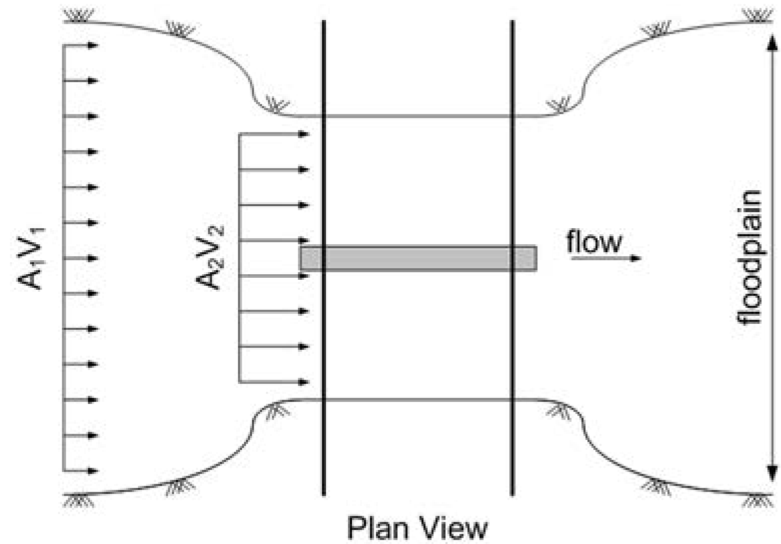


Figure 16.1.7 Stream Contraction Schematic

Figure 16.1.8 Contraction Scour

Figure 16.1.9 Large Number of Piers Reducing the Hydraulic Opening

The effects of contraction scour can be very severe. 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 16.1.9).
- Heavy vegetation in the waterway or floodplain (see Figure 16.1.10).
- 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.
- Ice formation or ice jams that temporarily reduce the waterway opening and produce contraction (see Figure 16.1.11).
- Flow under an ice sheet or flow contracting under the superstructure.
- Debris buildup, which often reduces the waterway opening (see Figure 16.1.12).

Figure 16.1.10 Vegetation Constricting the Waterway

Figure 16.1.11 Ice in Stream Resulting in Possible Contraction Scour

Figure 16.1.12 Debris Build-up in the Waterway

Local Scour

Local scour occurs around an obstruction that has been placed within a stream, such as a pier or an abutment which causes an acceleration of the flow and resulting vortices induced by obstructions to the flow. Local scour can 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.

Some common obstructions that may cause local scour (see Figure 16.1.13 and Figure 16.1.14) include the following:

- Abutments - floodplain overbank flow is collected along and forced around abutments at high velocities.
- Wide or Long Piers - scour depth is proportional to width. Different shapes can produce multiple vortices and greater scour depth if the pier is at an angle to the flow direction.
- Unusually Shaped Piers - can increase vortex magnitude. A square-nosed pier will most likely 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.

- Streamflow Velocity - as streamflow velocity increases vortex action can be magnified considerably.
- 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.

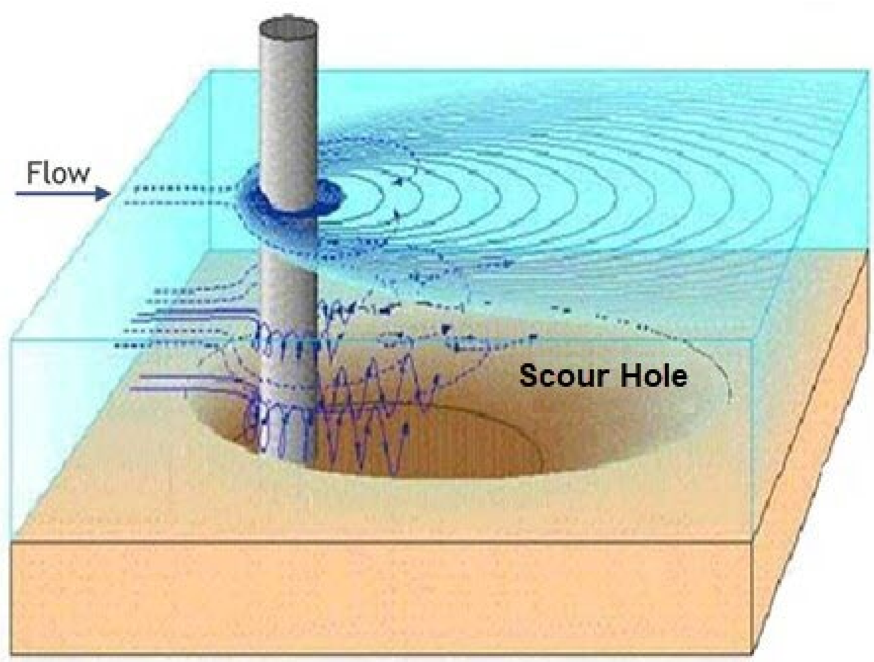


Figure 16.1.13 Local Scour at a Pier Schematic

Figure 16.1.14 Local Scour at a Pier

Scour depths resulting from local scour are typically deeper than those from contraction 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, degradation can be the leading contributor to 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 may have 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 the bridge inspection file. During each inspection, refer to past cross-section measurements to determine the degree of local scour that has developed over time. and whether the maximum local scour is a threat to bridge safety. The existence of scour is only a problem if it was not designed for. For example, if a substructure was designed to be able to have 10 ft of scour and there is 5 ft of scour observed, it should be noted but not rated as major scour. It may take some records review to have that design scour information available.

Channel depth measurements or soundings 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 by wading and probing. Scour measurements can also be taken by probing from a boat, by the use of divers, or by sonic methods when water velocity, depth, and other conditions dictate. The pattern of survey is established and will most likely remain the same during the life of the bridge, following 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. When possible, the scour and channel rating should be based upon maximum scour that occurred during floods and not upon examination of bed levels around abutments and piers during low-flow periods. Hence, it may be 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.

Inspectors should consider utilizing straight metal probing rods to probe loose sediments deposited along abutments and around footings; if sediment is finer than average 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 useful to incorporate underwater surveys using divers, or perhaps to even divert water away from critical members to enable removal of loose backfill material.

Based on risk, it may be appropriate to install permanent scour monitoring devices to measure scour development during flooding events, or the use of portable scour monitoring devices to monitor scour development at multiple piers (and bridges) as part of an owner's Plan of Action for scour critical bridges.

Scour can occur in a short time under certain conditions (see Figure 16.1.15 and Figure 16.1.16). Changes in downstream elevation, such as at the confluence with another river which is undergoing scour of its own, can cause scour in the upstream river (i.e., headcuts). Weather events such as hurricanes can accelerate scour development.

Figure 16.1.15 Micro-piles Exposed by Scour

Figure 16.1.16 Pier Foundation Exposed by Scour

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, the bridge inspection file should be used to study historical changes that have occurred in the bed elevation through the waterway. It may be possible to assess that these changes are related to specific causes. This information can be used to assess the present safety of the bridge, as well as provide insight for future conditions that may be imposed by changed flow conditions, watershed development, or other conditions affecting the safety of the bridge.

The challenge 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.

Lateral Stream Migration

Lateral stream migration or horizontal change in the waterway alignment is another type of erosion action 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 many bridge collapses in the country. Bridge approach roadways, abutments and piers are often threatened by this type of channel movement (see Figure 16.1.17).

Figure 16.1.17 Lateral Stream Migration Endangering an Abutment

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 exhibits lateral scour and the streambank protection may be failing (see Figure 16.1.18).
- 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 16.1.19).
- 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 can support itself without the underlying streambank material (see Figure 16.1.20).
- Channel misalignment - an adverse channel offset where the stream flow now impacts one of the bridge substructure units 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 can advance unchecked and leads to local scour conditions that result in undermining and substructure distress.

Figure 16.1.18 Streambank Damage

Figure 16.1.19 Sloughing Streambank

Figure 16.1.20 Undermined Streambank

Lateral stream migration is 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 16.1.21).
- Channel widening (see Figure 16.1.22).

Series of aerial photographs over time can be a way to check for lateral stream migration. Aerial images may be obtained using Unmanned Aerial Systems (UAS) or overhead maps data.

Figure 16.1.21 Stream Changes due to Lateral Migration

Figure 16.1.22 Channel Degradation due to Lateral Migration

When inspecting for lateral stream instability, some visual indicators may be:

- 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 (see Figure 16.1.23).
- Alternating 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.

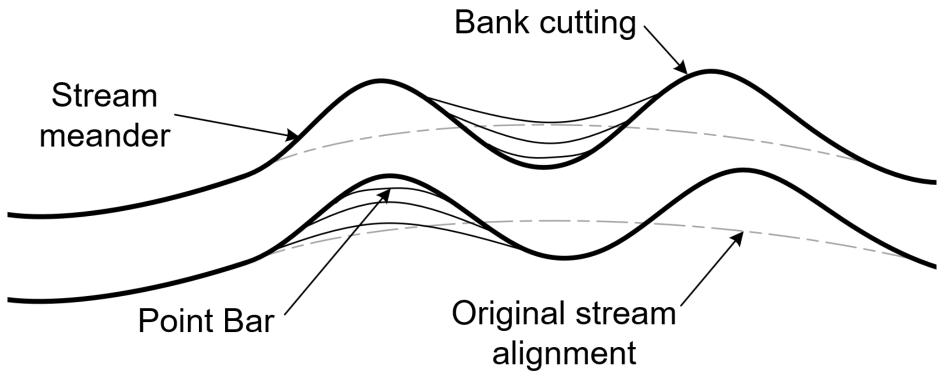


Figure 16.1.23 Stream Meandering with Point Bars

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 non-cohesive, cohesive, or composite bank material.

Non-cohesive bank material can be removed grain by grain from the streambank. The rate of the streambank erosion is 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 16.1.24).

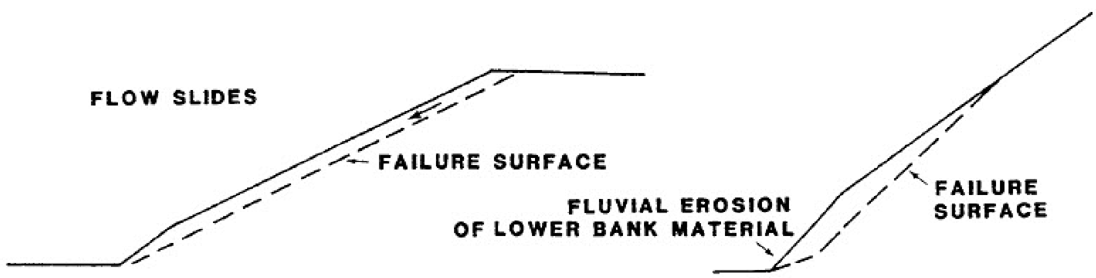


Figure 16.1.24 Schematic of Non-Cohesive Bank Material

Cohesive bank material is typically more resistant to erosion than non-cohesive bank material. It has low permeability which will likely 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, it is more likely to fail due to the mass wasting processes (see Figure 16.1.25).

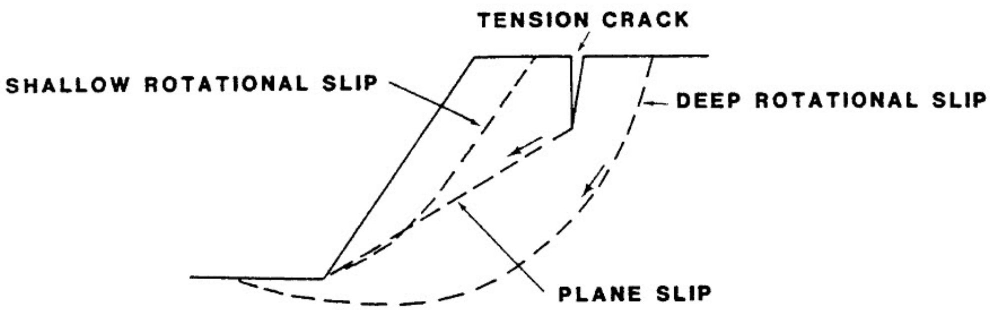


Figure 16.1.25 Schematic of Cohesive Bank Material

Composite bank material consists of layers of various sizes, permeability, and cohesive material. The non-cohesive layers will likely 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 16.1.26).

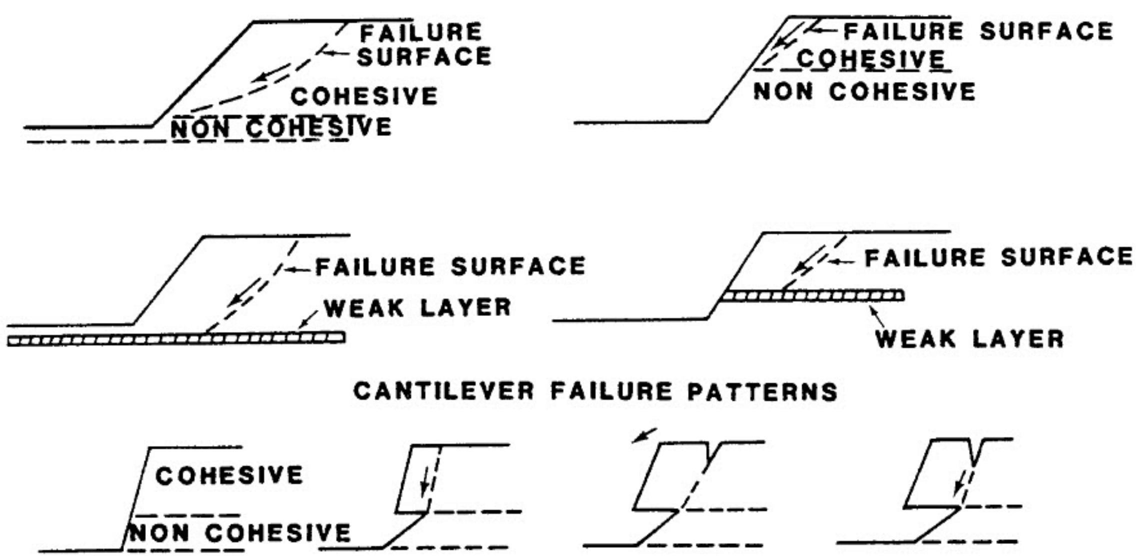


Figure 16.1.26 Schematic of Cohesive Bank Material

16.1.5 Purpose of Waterway Inspections

There are generally three major purposes for conducting waterway inspections.

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

Identify Damage

Waterway inspections are necessary to identify conditions that can severely damage or 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 a method used to detect loss of foundation support. When water depths prevent probing, divers may perform underwater inspections.

Record Existing Channel Conditions

Waterway inspections are conducted to create a record of the existing channel conditions adjacent to and under the bridge. Conditions such as channel opening width, depth at substructure members, 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, vegetation, debris, or pollution. These factors may require the bridge inspection team to return to the site during a period of low water flow. Alternatively, the inspection team should consider using an alternate means of waterway access, such as a boat, or an alternative inspection technique, such as underwater diving inspection.

Monitoring Channel Changes

Current waterway inspection data is compared to previous inspection data in order to identify channel changes. This "tracking" of channel changes over time is an important step in ensuring the safety of the bridge. Over time, vertical changes, due to 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 significant changes are found, a formal scour analysis of the site, involving a multi-disciplinary team of engineers, may be necessary 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 occurs.

16.1.6 Inspection Preparation

Inspectors should identify and assemble the documentation and equipment necessary to conduct the waterway inspection. The necessary equipment may depend upon the characteristics of the channel, the characteristics of the bridge, and the accessibility of the site.

Necessary Information

Inspectors should gather pertinent information for a comprehensive, well-organized inspection of waterways. Any previous hydraulic engineering scour evaluation or scour assessment studies on the bridge should be examined. These studies provide theoretical ultimate scour depths for the bridge substructure members. Inspectors should review original drawings and previous inspection report data to determine the foundation type and streambed material. Through regular comparisons of current and previous channel cross-sections, inspectors should establish whether the waterway is stable, degrading, or aggrading.

The inspection team should become familiar with site conditions and channel protection installations. Changes in the hydraulic opening should be verified by reviewing previous channel cross sections and any stream profile data. Also, inspectors should examine any photographs to determine any changes in the channel alignment.

Inspection Methods

Before beginning the inspection, the bridge inspection team should understand the type and extent of the inspection necessary. Waterway inspections are typically accomplished by surface inspection or underwater diving inspection.

Surface or "wading" inspection is conducted on shallow depth foundations when conditions provide. Submerged substructures, streambed, and embankments are often accessible by inspectors using hip boots or chest waders and probing rods (see Figure 16.1.27). Additionally, boats are often used as a surface platform from which to gather waterway data, including channel cross-sections, pier soundings, etc. Under certain conditions it can be appropriate to use other techniques.

Underwater diving inspection is used when the foundations are in deep water or in conditions adverse to wading. 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 members is suspected.

Equipment generally necessary to inspect bridges is listed and described in Chapter 2. Additional equipment may be used for the inspection of waterways. The type of equipment used 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 16.1.27).
- Sounding line (lead line to measure depths of scour).
- Fathometer to determine water depth.
- Diving equipment (see Figure 16.1.28).
- Boat, oars, motor, and anchor.
- 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 Section 16.2.10 for additional information on underwater inspection equipment.

Figure 16.1.27 Probing Rod and Waders

Figure 16.1.28 Surface Supplied Air Diving Equipment

Special Considerations

Special consideration should be given to the site conditions and the navigational controls that may adversely affect the safety of the bridge inspection team and others. Site conditions such as rapid stream flow velocity, pollution levels, safety concerns, and conditions requiring special observation should be accounted for during a waterway inspection (see Figure 16.1.29).

Navigational control may be necessary when inspecting large waterways. The Coast Guard should be notified in advance of inspections when and where navigational controls are necessary, for instance when evaluating the condition of dolphins and fenders. Navigational considerations include boat traffic, operational status, and dam release plans (see Figure 16.1.30).

Figure 16.1.29 Rapid Flow Velocity

Figure 16.1.30 Navigable Waterway

16.1.7 Inspection Methods and Areas

Methods

Visual & Physical

The primary method used to inspect waterways is visual. The inspection team should look at the site in the vicinity of the bridge as well as the floodplain. This observation may have to be done during periods of high flow. In addition, the inspection team should sight up and down the channel from the bridge deck.

The determination as to whether wading is a safe strategy for each site can be made by reviewing previous inspection reports, particularly channel cross section information.

After the inspection team assesses the general condition when visually inspecting the bridge site, the next step is to probe for any scour or undermining. To adequately determine the depth of an original scour hole, the probing rod should be pressed into the soil in the streambed. Sometimes scour holes are loosely filled with silt. This silt may be washed away during the next period of high stream flow velocity, permitting additional scour and possibly undermining.

Cross section measurements (see Figure 16.1.31) are generally taken under the bridge and can be used to record the changes in the streambed between inspections. The cross section under the bridge can be measured with a weighted tape or rod. These measurements can be used to find the area of the hydraulic opening and help determine a need for and design of mitigation measures.

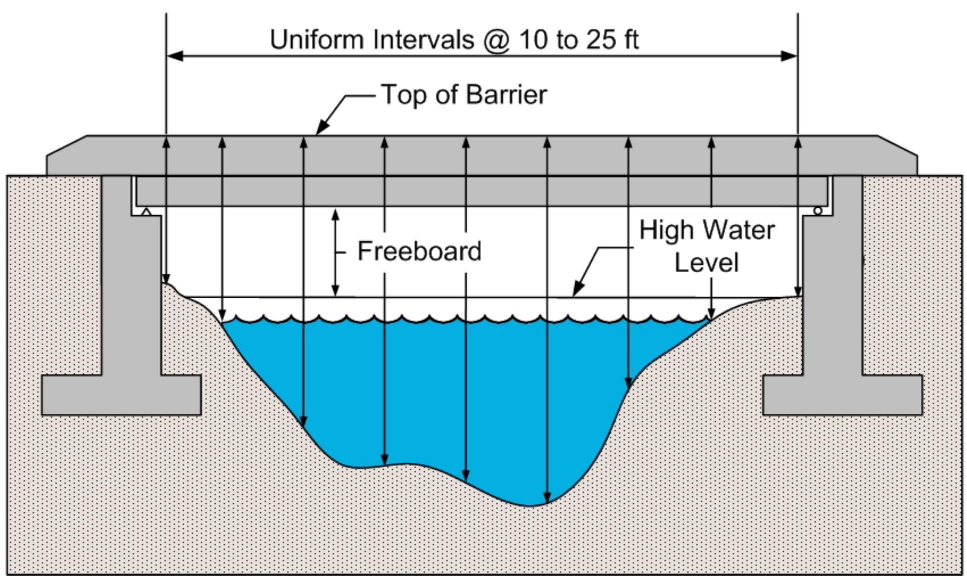


Figure 16.1.31 Streambed Cross-Section

Advanced Inspection Methods

Sometimes advanced methods can be used to take measurements to obtain the channel cross section and profile. The stream profile can be measured with a hand level, survey tape, surveying rod, and laser distance meters (see Figure 16.1.32). This data can also be used to compile channel soundings. Comparison of the streambed profile and hydraulic opening to previous inspections (including the baseline cross section) can indicate lateral or vertical channel movement.

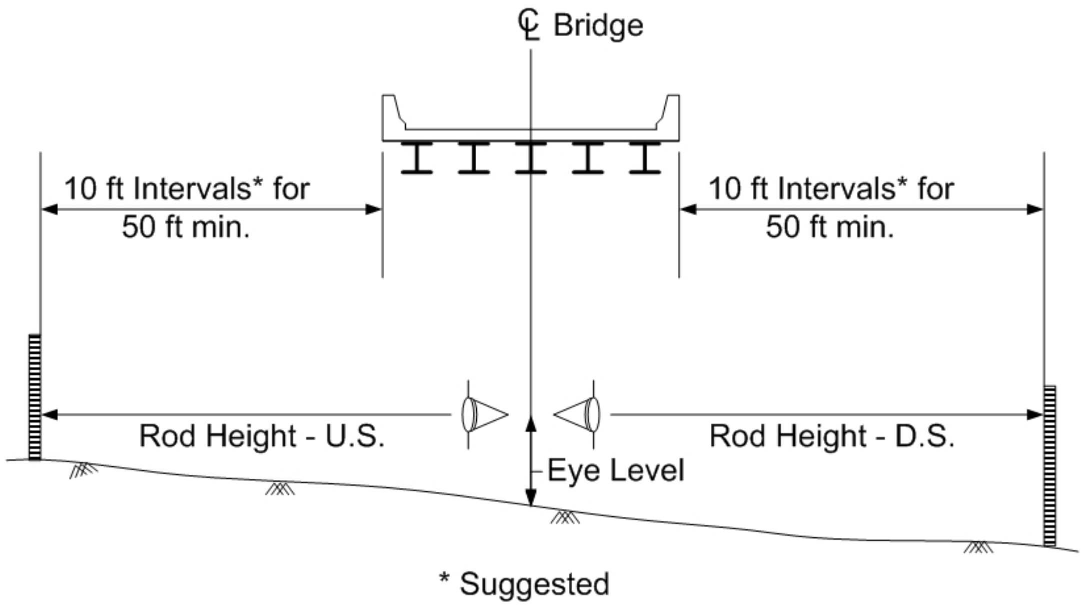


Figure 16.1.32 Streambed Profile

As an alternative to soundings, there are four common real-time scour monitoring devices used by inspection teams. These instruments include scour sensing devices, tilt meters, magnetic sliding collars, and float-out devices. Owners have elected to install fixed instrumentation directly on the substructure of bridges deemed to be higher risk or as a research project. With fixed instrumentation, local scour is continuously monitored and recorded as it occurs, unaffected by the refilling 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 off-site office).

One such instrument is the magnetic sliding collar which consists of a steel rod driven into the riverbed and the steel collar to slide along the rod, resting on the streambed at installation. As the streambed erodes, and the bed elevation lowers, so does the collar. Magnetic triggers in the rod determine the collar's elevation based on its relative location to the rod (see Figure 16.1.33). There are typical challenges associated with placement of mechanical devices in environments susceptible to debris and barnacle growth where fouling can occur.

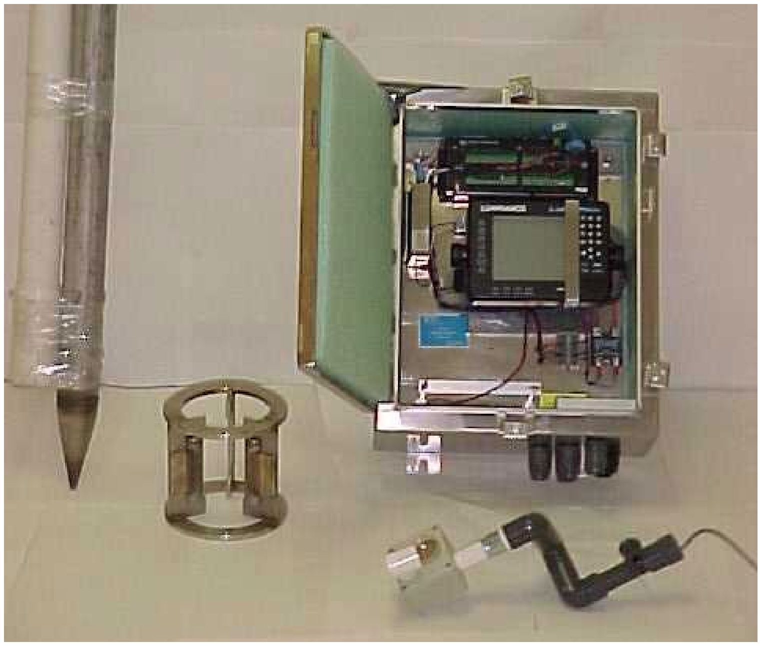


Figure 16.1.33 Scour Monitoring Collar Instrument

Other fixed instrumentation includes fixed sonar units, and buried "float-out" buoys. These buoys may be found at varying depths adjacent to the substructure, which float to the water surface after being uncovered by local scour, activating an electronic alarm system.

Sonar devices continue to be the state of practice at this time. Sonar devices may be permanently fixed to bridge foundations, may be portable sonar devices which can be moved from bridge site to bridge site, or can be deployed via survey vessels (see Figure 16.1.34).

Figure 16.1.34 Inspector with AUV Device

Researchers are continuing to find new ways to refine the devices used for data collection, like incorporating sonar, GPS, and other features into Autonomous Underwater Vehicles (AUV) and Unmanned Surface Vehicles (USV). These vehicles have the advantage of being deployed without a crew and continue to be refined to operate in fast flowing water and provide improved bathymetric surveys for use in monitoring scour, in near-real-time, in the vicinity of bridges.

Some agencies have implemented an automated remote flood event monitoring notification system. The goal is to provide bridge owners with a tool to predict and prepare for destructive weather events. These monitoring software programs may include a mapping feature to locate bridge assets and assess their potential risks during storm events. Bridges that are particularly susceptible to scour issues can be closely monitored.

Inspection Area: Channel Under the Bridge

When inspecting the bridge waterway, there are typically three main areas of concern. These areas include the channel under the bridge, the upstream channel, and the downstream channel.

Substructure

Refer to Chapter 14 for details on substructure members. The following list provides guidance with regards to inspection of the waterway at bridge substructures.

- Substructure units should be inspected below water level for defects, damage, and foundation condition (see Figure 16.1.35).
- Heights and lengths of exposed foundation members, and dimensions of foundation undermining (opening height, width, and penetration depth) should be measured, as applicable. The inspection team should document with sketches and photos.
- The inspection team should note the location of the high-water mark on abutments and piers.
- The face of abutments and piers/bents should be plumbed for local settlement (see Figure 16.1.36).
- Abutments and piers should be checked for accumulations of debris (drift).
- In case of damage to scour countermeasures, inspectors should check the condition and function of channel protection devices adjacent to substructure units.
- The inspection team should check upstream and downstream for a change of condition such as dredging, channel widening, or development that may have caused changes near the structure.
- The streambed cross-sections should be routinely updated along the upstream and downstream bridge rails, and compared to the baseline cross-section and previous cross-section.
- Sketches with dimensions of large scour holes at the substructure should be provided.
- A grid system for depth soundings at substructure members should be established, which can be repeated in subsequent inspections.
- Inspectors should take photographs to document conditions of abutments, piers, and channel features.
- Inspectors should check bridge seats and bearings for transverse movement.

Figure 16.1.35 Pile Bent Deterioration Typically Hidden Underwater

Figure 16.1.36 Out of Plumb Bent

Superstructure

During a waterway inspection, the superstructure and surrounding landscape can be an indicator of existing waterway deficiencies. Inspectors should check for the following potential issues.

- Superstructure is tied to the substructure to prevent washout.
- Inspectors should sight along the superstructure or bridge railing to reveal irregularity in grade or horizontal alignment caused by settlement (see Figure 16.1.37).
- Debris lodged in superstructure members or tree limbs overhead (see Figure 16.1.38).
- High watermarks or ice scars on trees.
- Inspectors should talk to local residents about high water during previous flood events.
- Overtopping flow elevation and frequency as per scour evaluation analyses.
- A large surface of resistance created by the superstructure during floods.
- Superstructure vulnerability to collapse in the event of excessive foundation movement (i.e., simple span vs. continuous).

Figure 16.1.37 Superstructure Misalignment

Figure 16.1.38 Drift Lodged in a Superstructure

Channel Protection and Scour Countermeasures

When examining any channel protection devices or scour countermeasures that may be present at the bridge, the inspection team should check for the following potential issues.

- Stability and condition of river training and bank protection devices.
- Gaps or spreading that have occurred in the protective devices.
- Separation of slope pavement joints.
- Exposure of underlying erodible material.
- Steepening of the protective material and the surface upon which these materials are placed.
- Condition and function of existing riprap, or newly placed riprap.
- Evidence of failed riprap in the stream (see Figure 16.1.39).
- Improper movement, condition, and function of guidebanks, or spurs.
- Evidence of scouring in the streambed near or under the channel protection device.
- Streamflow encroachment behind the protective devices.

Figure 16.1.39 Failed Riprap

It is important to identify any change that is observable, including changes to the riprap, as this can be indicative of larger issues. 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. Inspectors should 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 may be necessary to probe or sound for physical evidence to identify whether failure or partial failure exists.

Waterway Area

In examining the waterway, specifically under the bridge, inspectors should check for the following potential issues.

- If the width of the hydraulic opening is small compared to the floodplain, there could be high potential for contraction scour at the abutments.
- Determine the type of streambed material.
- Degradation (see Figure 16.1.40).
- Local scour around piers and abutments.
- Inspectors should inspect during drought conditions when applicable.
- Contraction scour due to abutment placement, sediment build-up, and vegetation.
- Debris underwater, which may constrict flow or create local scour conditions.
- Approach roadways in the floodplain.
- Approaches for signs of overtopping.
- Inspectors should determine if the hydraulic opening is causing or has the potential to cause scour under the bridge.

Figure 16.1.40 Severe Streambed Scour Evident at Low Water Flow

Inspection Area: Upstream and Downstream of the Bridge

In addition to the channel under the bridge itself, there are several waterway features upstream and downstream that should be inspected.

Streambanks

Inspectors should examine the streambanks both upstream and downstream of the bridge, and note whether the banks are stable or unstable based on the following descriptions.

- 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 16.1.41).
- Unstable **-** streambank is sloughing due to scour, evidence of lateral movement or erosion, damage to slope stabilization measures (see Figure 16.1.19).

Figure 16.1.41 Stable Streambanks

Main Channel

The streambanks should be examined both upstream and downstream of the bridge. Inspectors should record the information and check for the potential issues as described below.

- Flow conditions (e.g., low or high).
- Velocities should be estimated using floats.
- Sediment buildup and debris, which may alter the direction of stream flow (see Figure 16.1.42).
- 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 16.1.43).
- Streambed material type.
- Aggradation or degradation (check several hundred ft upstream and downstream of the bridge).
- Basic alignment of the waterway with respect to the structure and compare it to its original alignment (lateral stream migration) (see Figure 16.1.44).
- Direction and distribution of flow between piers and abutments.
- Inspectors should make sketches and take pictures as necessary to document stream alignment, conditions of bank protection, and anything that appears unusual at each inspection.

Figure 16.1.42 Sediment Accumulation Redirecting Streamflow

Figure 16.1.43 Fence in Stream at Bridge

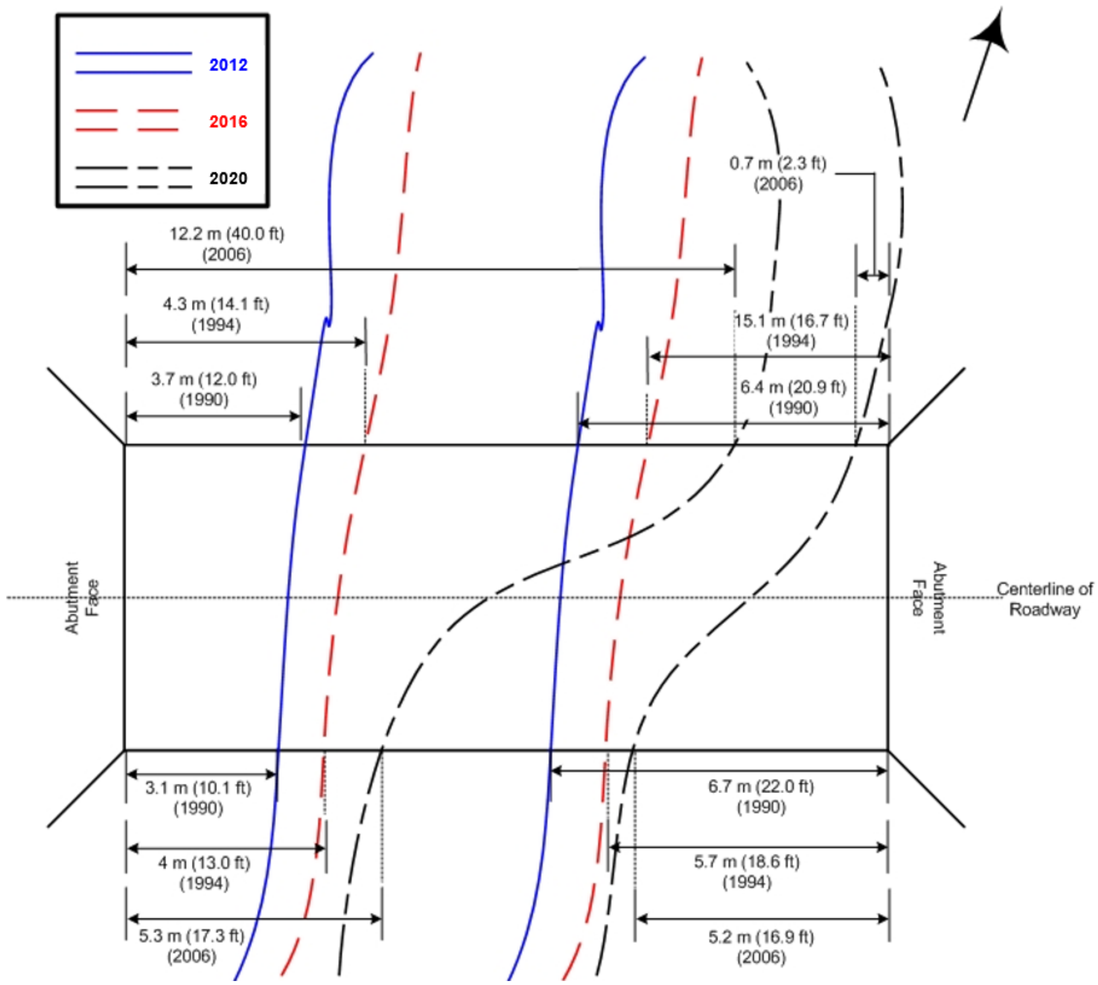


Figure 16.1.44 Waterway Alignment Sketch from 2012 - 2020

Floodplain

Inspectors should examine the floodplain in the area of the bridge. Inspectors should check for the following potential issues.

- Evidence of embankment sloughing, undermining, and lateral stream migration resulting from significant stream flow.
- Amounts and locations of debris, sediment accumulations, tree scaring, and amounts of vegetation growth, all of which may indicate the frequency of stream flow in the floodplain.
- 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.
- 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 (see Figure 16.1.45).
- Extent of structures, trees, and other obstructions that could impact the stream and adversely affect the bridge site (see Figure 16.1.46).

Figure 16.1.45 Approach Spans in the Floodplain

Figure 16.1.46 Debris and Sediment in the Channel

Other Features

In addition to the channel locations previously described, inspectors should check for streamflow impact of any other features such as tributaries, confluence of another waterway, dams, and substructure units from other bridges (see Figure 16.1.47). This may affect stream flow velocity through the bridge.

Any recent construction activity (e.g., causeways, fishing piers, and stranded vessels) that may affect stream flow under the bridge, should be reported.

Figure 16.1.47 Upstream Dam Impacts Streamflow on Bridge

16.1.8 Scour Appraisal

By regulation, all bridges over water should be appraised to determine whether or not they are scour critical. In order to accomplish this, a scour appraisal should be performed. A scour appraisal is a risk-based and data-driven determination of a bridge's vulnerability to scour and instability, resulting from scour that is observed, or estimated using a scour evaluation or a scour assessment (23 CFR 650.305). The appraisal determination is based upon the least stable of these methods.

Scour evaluation is the application of hydraulic analysis as described in HEC-18 and HEC-20 to estimate scour depths and determine bridge and substructure stability considering potential scour (23 CFR 650.305). HEC-18 is the Hydraulic Engineering Circular No. 18, *Evaluating Scour at Bridges*, and HEC-20 is Circular No. 20, *Stream Stability at Highway Structures*.

A scour assessment is the determination of an existing bridge's vulnerability to scour which considers stream stability and scour potential as described in HEC-20 and other scour-related data sources (23 CFR 650.305).

Purpose and Objective

When performing scour appraisals, structural, hydraulic, and geotechnical engineers make decisions on:

- The scope of the scour appraisals to be performed in the office and in the field; observed or estimated.
- Once the total anticipated scour depth is determined, make the decision on whether a bridge is considered scour critical considering the bridge's foundations.
- 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 channel protection and scour countermeasures.
- Monitoring and inspecting scour critical bridges.

Bridge inspectors should gather on-site data that may be used to ensure that assumptions accurately represent the present condition of the bridge and the stream (see Figure 16.1.48). The data may also be used to identify conditions that are indicative of potential problems with scour and stream stability. These identified conditions should be used to ensure that the assumptions made in the scour appraisal are consistent with that appraisal and that the current code for scour vulnerability is appropriate. In addition, the on-site data provides the most recent channel cross-section measurements.

Recognition of Scour Potential

To accurately assess the scour, the inspection team should 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 inspection team, further evaluation of the bridge is completed by an interdisciplinary team made up of structural, geotechnical, and hydraulic engineers.

Figure 16.1.48 Bridge Abutment Affected by Scour

Recognition of Scour Potential: Waterways

- Inspectors should identify and record waterway conditions at the bridge or culvert, as well as both upstream and downstream of the structure. Inspectors should look for the following indicators of scour potential in the waterway. Stream flow velocity is a major factor in the rate of scour. High velocities produce accelerated scour rates (see Figure 16.1.49 and Figure 16.1.50).
- Streambed materials such as soft cohesive soils, sand, or gravel material, are highly susceptible to accelerated scour rates from high stream velocities (see Figure 16.1.50).

- Orientation of waterway opening such as misaligned or skewed structure foundations, which can frequently generate adverse streamflow conditions, can lead to scouring of the streambed especially during high flood flows (see Figure 16.1.51).
- 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.
- Banks that are sloughing, undermined, or moving laterally are signs of potential scour at a bridge (see Figure 16.1.52).

Figure 16.1.49 Fast Flowing Stream

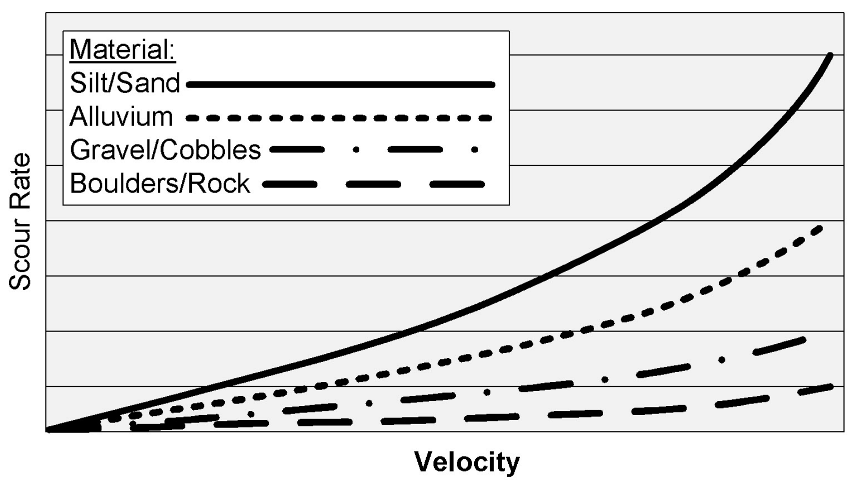


Figure 16.1.50 Scour Rates vs. Velocity for Common Streambed Materials

Figure 16.1.51 Misaligned Waterway

Figure 16.1.52 Lateral Stream Migration

Recognition of Scour Potential: Substructure

Consider the following condition of bridge foundations and substructure units when reporting potential scour conditions:

- 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 16.1.53).
- Rotational, horizontal, or vertical movement of piers and abutments can be evidence of undermining (see Figure 16.1.54).
- Spread footing foundation levels above maximum calculated scour depth determined (scour critical) for a particular bridge is 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 16.1.55).
- 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 16.1.56).

Figure 16.1.53 Stream Alignment Not Parallel with Abutments

Figure 16.1.54 Rotational Movement and Failure Due to Undermining

Figure 16.1.55 Exposed Piling Due to Scour

Figure 16.1.56 Accelerated Flow Due to Constricted Waterway

Recognition of Scour Potential: Superstructure

Inspectors should consider the following conditions associated with the superstructure when reporting potential scour conditions:

- 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.

Specifications for the National Bridge Inventory (SNBI) Condition Items

In assessing the adequacy of the bridge to resist scour, the inspection team and engineer should understand and recognize the interrelationships between several items:

- Substructure Condition Rating (B.C.03).
- Channel and Channel Protection Condition Rating (B.C.09 and B.C.10).
- Scour Condition Rating (B.C.11).
- Underwater Inspection Condition (B.C.15).

Refer to Chapters 3 and 18 for a detailed description of NBI Items.

Substructure Condition Rating

The substructure condition rating is a key item for rating the bridge foundations for any effect from scour damage. When observed conditions are not consistent with the scour design or assumptions used in the scour appraisal, scour is to be considered in the condition rating of the substructure. If the bridge is determined to be scour vulnerable, further evaluate the condition rating for the substructure to ensure that any existing problems have been properly considered in the scour appraisal. Scour is considered for coding this item when observed conditions are not consistent with the assumptions used in scour appraisal or the scour design. In this case, Scour Vulnerability will need to be reevaluated per the *SNBI*.

Channel Condition Rating

The channel condition rating is used to describe the extent that defects negatively affect the channel at the bridge. Inspectors should consider the channel upstream and downstream only insofar as it threatens the bridge and approach roadway. For concrete lined channels, all channel defects except for aggradation and debris will typically be not applicable.

Channel Protection Condition Rating

The channel protection condition rating is used to provide a condition rating for channel protection devices. The condition and effectiveness of channel protection devices installed on banks or in the stream to mitigate channel issues that may impact the bridge should be evaluated. When reporting this item, inspectors should consider erosion and scour, damage, and material defects.

Channel protection devices are considered countermeasures that control, inhibit, delay, or minimize stream instability and scour problems, including river training and armoring countermeasures.

Scour Condition Rating

Scour condition rating is used to describe the extent that observed scour depths in the field negatively affects the stability of a bridge. This rating also describes those actions that should be taken to address the safety conditions resulting from the observed scour condition.

When observed conditions are not consistent with the scour design or the assumptions used in the scour appraisal, then the observed conditions may indicate a need to reevaluate potential scour depths, performed by an interdisciplinary team comprised of structural, hydraulic, and geotechnical engineers.

Underwater Inspection Condition

The underwater inspection condition item represents the condition of underwater members identified to be inspected in the underwater inspection procedures, and incorporated into the substructure condition rating.

SNBI Appraisal Items

There are several appraisal items in the *SNBI* that relate to the waterway features at a bridge, including:

- Overtopping Likelihood (B.AP.02).
- Scour Vulnerability (B.AP.03).
- Scour Plan of Action (B.AP.04).

Overtopping Likelihood

This item depicts the likelihood of the waterway overtopping the riding surface carried on the bridge. Bridge overtopping likelihood, since the year built, is typically determined from historical bridge inspection or maintenance records, hydraulic studies, local residents/landowners, and/or site indicators including highwater marks on the bridge or its surroundings, debris remains on bridge upper members, etc.

For newer bridges with limited historical inspection or maintenance information, hydraulic design information can be used to establish an overtopping likelihood.

This item does not apply to the likelihood of the waterway overtopping approach roadways.

Scour Vulnerability

The intent of this item is to record the status and scour vulnerability determination from scour appraisals required by the NBIS.

The codes for this item are based on the appraised scour vulnerability as described in HEC-18 *Evaluating Scour at Bridges*, HEC-20 *Stream Stability at Highway Structures*, and HEC-23 *Bridge Scour and Stream Instability Countermeasures*. Additionally, there are codes that should be used for bridges with unknown foundations as well as conditions where non-designed countermeasures have been installed.

Scour appraisals are typically performed by a multidisciplinary team of hydraulic, geotechnical, and structural engineers (Scour Appraisal Team) (*SNBI* Subsection 7.4, Item B.AP.03 Commentary).

When observed conditions are not consistent with the scour design or the assumptions used in the scour appraisal, then scour is considered in the coding of the scour condition rating. In this case, observed conditions also indicate a need to reevaluate the scour appraisal code.

A scour critical bridge is a bridge with a foundation member that has been determined to be unstable by the scour appraisal.

Scour Plan of Action

For scour critical bridges, bridges with unknown foundations, and bridges with non-designed countermeasures, the NBIS (23 CFR 650.313 (o) (2)) requires that a Plan of Action (POA) is developed for mitigating the scour problem or potential problem. The Scour Plan of Action is a list of procedures for bridge inspectors and engineers use in managing each bridge determined to be scour critical or that has unknown foundations (23 CFR 650.305). Such a plan would address the type and interval of future monitoring or inspections to be made, and for higher risk bridges should also include a schedule of timely design and construction actions for appropriate countermeasures to protect the bridge. Monitoring is considered a type of countermeasure, and for low risk bridges may be tolerable as the only component of a POA. During each inspection, the inspection team verifies that the information in the Scour POA is current with the bridge and site conditions.

Section 16.2 Underwater Inspection

16.2.1 Introduction

Underwater inspections are an important part of a bridge owner's management system. Over 80 percent of the bridges in the National Bridge Inventory (NBI) are built over waterways (which can be obtained from the National Bridge Inventory at https://www.fhwa.dot.gov/bridge/nbi/ascii.cfm). Many of these bridges have foundation members in water. Bridge failures occur because of underwater and channel instability issues. Underwater members should be inspected, to the extent necessary, to determine with as much certainty as possible that their condition has not been compromised and negatively affect the structural safety of the bridge.

Several bridge collapses during the 1980s, traceable to underwater deficiencies (see Figure 16.2.1), led to the underwater inspection provisions of the National Bridge Inspection Standards (NBIS). As a result, requirements for underwater inspections are outlined in the NBIS (23 CFR 650.313 (e)) . Underwater inspection procedures developed as per the requirements under 23 CFR 650.313 (g) are to be followed for each Underwater inspection.

Figure 16.2.1 Schoharie Creek Bridge Failure

As defined in 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 (23 CFR 650.305).

The expense of such inspections necessitates careful consideration of the bridge candidates described above, since underwater inspection is sometimes a hands-on inspection requiring an underwater breathing apparatus and related diving equipment.

Bridge Applicability Criteria

Bridges that cross waterways often have substructure foundations in water to provide the most economical total design. Where these members are continuously submerged (see Figure 16.2.2), underwater inspections are performed, when wading and probing is not done, to establish their condition and degree of any scour.

In many cases, a multi-disciplinary team including structural, hydraulic, and geotechnical engineers evaluate a bridge over water that is a candidate for underwater inspection.

Figure 16.2.2 Liberty Bridge over Monongahela River

Factors to Consider

Various factors influence the underwater bridge inspection selection criteria. In accordance with the NBIS (23 CFR 650.311) and the AASHTO *MBE*, all structures receive routinely scheduled Underwater inspections at intervals not exceeding 60 months, unless requirements are met that qualify the bridge for extended or reduced intervals.

The maximum interval permitted between Underwater inspections for bridges, that are in satisfactory condition underwater and that are in passive, nonthreatening environments, is not to exceed 72 months. More frequently scheduled and detailed inspections may be desirable for many structures and necessary for critical structures. Refer to Section 16.2.3 for further details relating to extended and reduced Underwater inspection intervals.

Factors to consider in establishing the agency extended or reduced inspection interval policy and levels of inspection include:

- Structure type.
- Design.
- Materials.
- Age.
- Condition ratings.
- Scour.
- Environment.
- Traffic.
- History of vehicle/vessel impact damage.
- Loads and safe load capacity.
- Oher known deficiencies.

The MBE also provides a list of recommended items to consider when determining inspection intervals for Underwater inspections. These include:

- Past inspection findings.
- Age.
- Condition.
- Scour/Scour history.

Designated Bridges

Note those bridges that require Underwater inspection on the bridges' individual inspection reports and inventory files. For each bridge requiring Underwater inspection, the following information should be included as a minimum:

- Type and location of the bridge.
- Type and interval of necessary 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.
- A master list of applicable bridges for the Agency.

16.2.2 Diving Levels of Inspection

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 levels of inspection have evolved as follows:

- Level I - Visual, tactile inspection.
- Level II - Detailed inspection with partial cleaning.
- Level III - Highly detailed inspection with nondestructive testing (NDT) or partially destructive testing (PDT).

Routine Underwater inspections typically 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 portions that may be exposed that would typically be buried. The Level I inspection should be detailed enough to detect obvious major damage or deterioration. A Level I inspection is typically 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 necessary. If the water clarity is poor enough that the inspection team 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 more detailed inspection where portions of the structure should be cleaned of marine or aquatic growth (see Figure 16.2.3). In some cases, cleaning is time consuming, particularly in saltwater, and should be restricted to critical areas of the structure. However, in fresh water, aquatic coatings can often be removed by just wiping the structural member with a glove.

Generally, the critical issues and defects commonly occur adjacent to the low waterline and near the mud line. It is also good practice to inspect the entire area between the low waterline and the mud line. On pile structures, horizontal bands, approximately 6 to 12 inches in height should 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 16.2.3 Level II Cleaning of a Steel Pile

On large substructure units, such as piers and abutments, areas at least 1 square foot in size at three or more levels on each face of the member should be cleaned (see Figure 16.2.4). For a structure that is greater than 50 ft in length, it is a good practice to 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. Deficient areas should be measured and documented, 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 members. The thoroughness of cleaning should be governed by what is necessary to determine the condition of the underlying material. Complete removal of all growth is generally not necessary.

Figure 16.2.4 Diver Cleaning Pier Face for Inspection

Level III

A Level III inspection is a highly detailed inspection of a structure or structural member of high concern, or a member where extensive repair or possible replacement is contemplated. This type of inspection can 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 ultrasonic, 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 may be suspect; or areas that may be representative of the entire bridge member in question.

16.2.3 Types of Underwater Inspections

A comprehensive review of bridges contained in an agency's inventory should 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. Any of these inspection types apply to above waterway and Underwater inspections:

- Routine.
- Initial.
- Damage.
- In-Depth.
- Special.
- Scour Monitoring.

Underwater inspections are typically Routine, or In-Depth inspections. Team leaders are required to be present at each Underwater inspection (23 CFR 650.313 (j)).

Routine Underwater Inspections

Most Underwater inspections are regularly scheduled, intermediate level inspections 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 should incorporate Level I, Level II, and a review of scour condition.

The summary recommendations for a Routine Underwater inspection include:

- A Level I inspection should be conducted on 100% of the underwater portion of the structure to determine obvious problems.
- A Level II inspection should be conducted on at least 10% of underwater units selected as determined by the Level I inspection.
- Scour evaluations that help give the cross-section of the channel by sounding and probing near the underwater sections of the substructure.

The dive team also conducts a review of scour condition at the bridge site. They should inspect the channel bottom and sides for scour, take cross sections of the channel, sound substructures, and determine undermining.

Cross sections of the channel bottom routinely taken should be compared with as-built plans or previously taken cross sections to detect lateral channel movement or deepening (see Figure 16.2.5). Cross sections are typically taken at the upstream and downstream faces of the bridge.

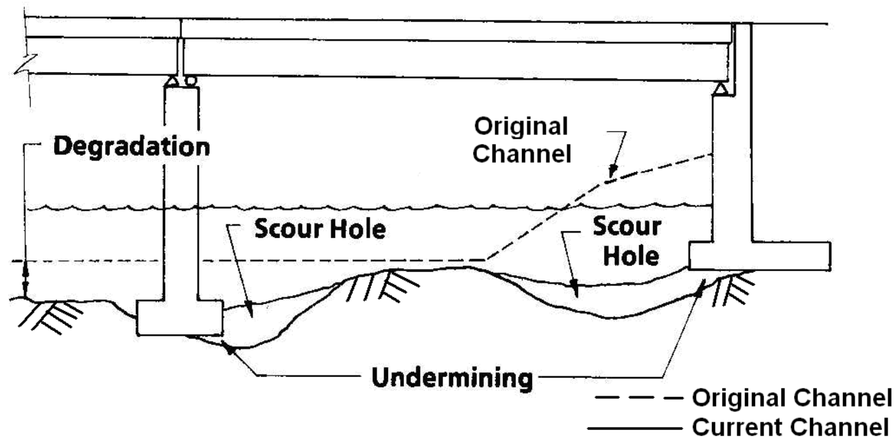


Figure 16.2.5 Channel Cross-Section (Current Inspection Versus Original Channel)

Soundings should be made in a grid pattern (see Figure 16.2.6) about each pier and upstream and downstream of the bridge, developing contour elevations of the channel bottom, to detect areas of scour.

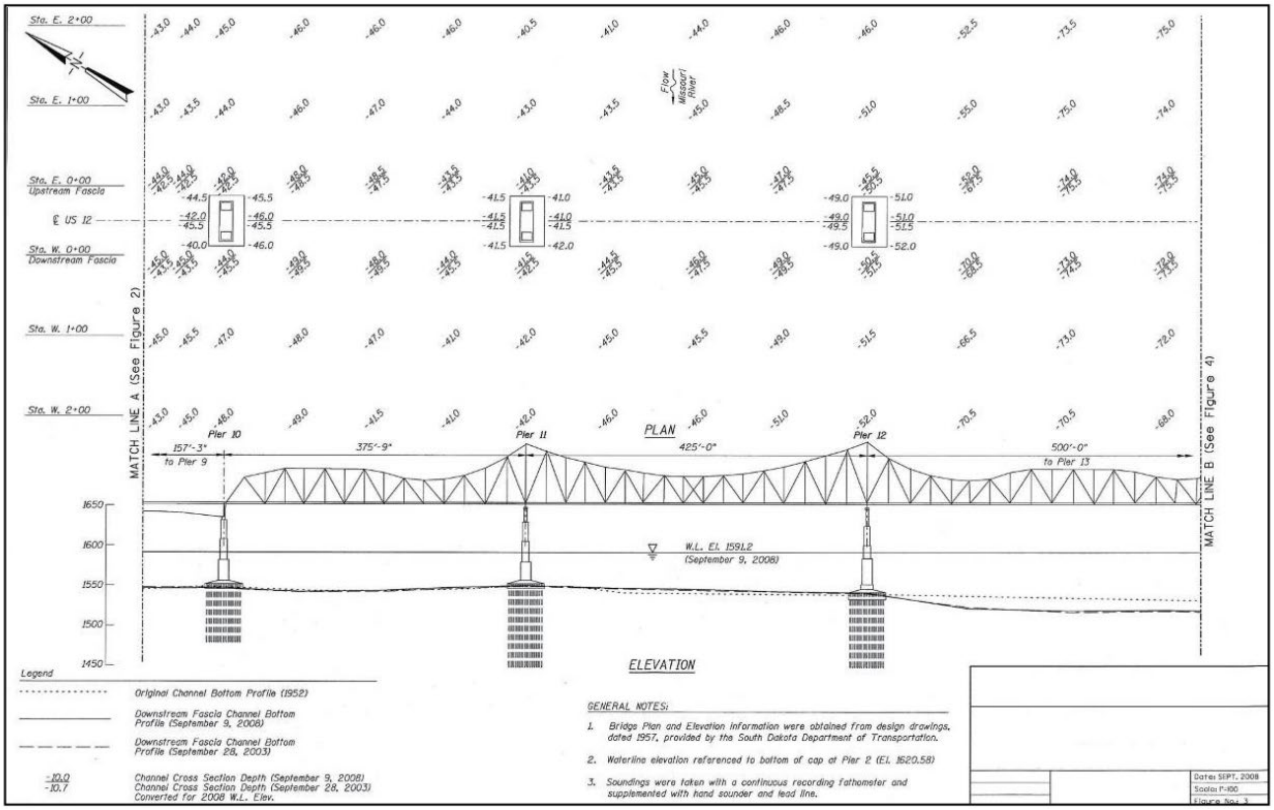


Figure 16.2.6 Pier Sounding Grid

Permanent reference point markers can be placed on each abutment/pier (see Figure 16.2.7). Data obtained from the soundings should be correlated with the original plans (if available) of the bridge foundations and tied to these markers for reference during future Underwater inspections.

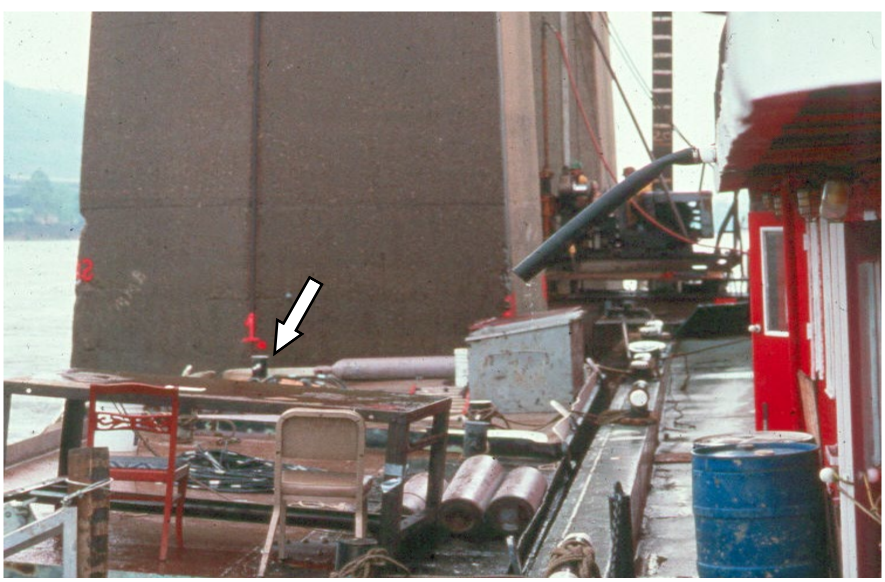


Figure 16.2.7 Permanent Reference Point (Bolt Anchored to the Pier)

Local scour and undermining can be determined with probes or rulers in the vicinity of piers and abutments (see Figure 16.2.8). In streams carrying large amounts of sediment, reliable scour depth measurements may be difficult at low flow due to scour hole backfilling with a fine material.

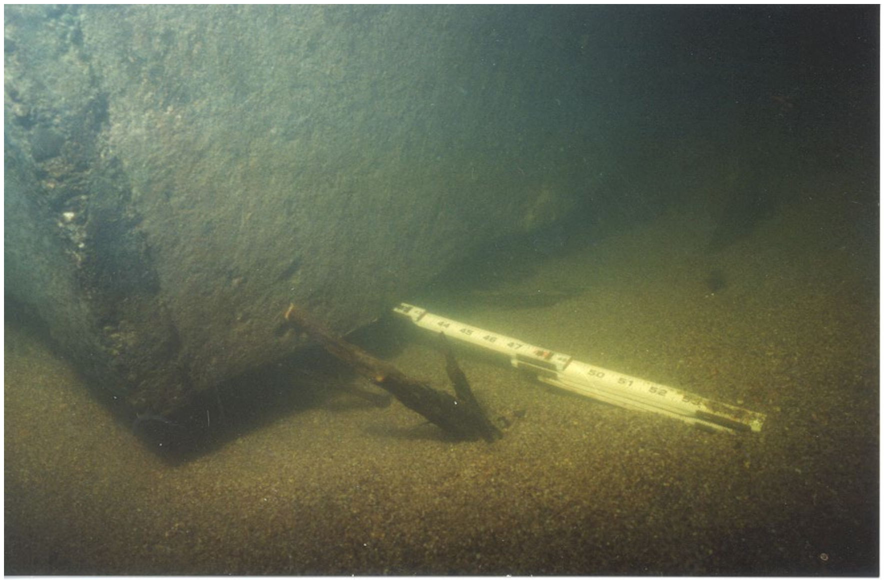


Figure 16.2.8 Local Scour - Causing Undermining of a Pier Footing

Initial Underwater Inspections

An Initial Underwater inspection is the first Underwater inspection of a bridge as it becomes a part of the bridge inventory. An Initial Underwater inspection is a fully documented investigation that will typically incorporate Level I and Level II inspections and a review of scour conditions as necessary for the next scheduled Underwater inspection. In addition, this type of inspection will likely provide the relevant data to determine the baseline structural conditions. It also identifies and lists existing problems and locations, 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) should be noted for subsequent focus and special observation.

An Initial Underwater inspection may also be necessary when there has been a change in the configuration of the structure such as widening, lengthening, bridge replacement, or change in ownership. The first Underwater inspection for each bridge and for each bridge with portions underwater that have been rehabilitated should be performed as soon as practical, but within 12 months of the bridge opening to traffic (23 CFR 650.313 (e)).

Damage Underwater Inspections

Certain conditions and events affecting the underwater portion of a bridge may require an unscheduled Underwater inspection to assess structural damage resulting from environmental or accident-related causes. Examples may include flooding or barge collision.

The scope of the Damage Underwater inspection is to be sufficient to determine the need for emergency load restrictions or closure of the bridge to highway or marine traffic and to assess the level of effort necessary to repair the damage. The amount of effort expended on this type of inspection may vary significantly depending upon the extent of the damage. If major damage has occurred, the inspection team should evaluate section loss, make measurements for misalignment of members, and check for any loss of foundation support.

In-Depth Underwater Inspections

An In-depth Underwater 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 standard Underwater inspection procedures. When appropriate or necessary to fully ascertain the existence of or the extent of any deficiencies, Level III, nondestructive tests should be performed.

The In-depth Underwater inspection typically includes Level II inspection over extensive areas and Level III inspection of limited areas. Nondestructive testing is typically 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 Underwater 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 Underwater 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 Underwater inspections.

Special Underwater Inspections

Sometimes additional Underwater inspections are scheduled at the discretion of the individual in charge of bridge inspection activities. These may be referred to as Special inspections. A Special Underwater inspection is used to monitor a particular known or suspected deficiency (e.g., foundation settlement or undermining).

Scour Monitoring Inspections

According to the FHWA *SNBI*, a Scour Monitoring inspection is an inspection performed during or after a triggering storm event as required by a Scour Plan of Action (POA), by personnel with qualifications required by the agency. Underwater inspection portions of a Scour Monitoring inspection should be completed after a triggering storm event, wherever possible, for the safety of the inspection team. After the storm event, Underwater inspection may be necessary to monitor suspected scour issues and to record any changes in condition.

Conditions for Inspection

Situations that may warrant a Damage, In-depth, Special, or Scour Monitoring Underwater inspection include:

- Unusual floods.
- Vessel impact.
- Unusual ice flows.
- Prop wash from vessels.
- Adverse environmental conditions.
- Floating and build-up of debris.
- Evidence of deterioration or movement above the waterline.

Bridge members should be inspected after floods. Bridge members situated in streams, rivers, and other waterways with known or suspected scour potential after every major runoff event should be inspected to the extent necessary to ensure bridge foundation integrity.

Underwater inspection should be performed if there is visible damage above water from vessel impact or settlement from scour. This should be done in order to determine the extent of the damage or deficiencies.

Unusual ice flows can damage substructure members, and accumulations of ice on the members can cause scouring currents or increase the depth of scour.

Turbulence caused by the propellers of marine vessels can cause scouring currents and may propel coarse-grained bottom materials against substructure members in a manner similar to that of blast cleaning operations.

Rapid and severe deterioration of substructure materials may be caused by environmental conditions such as brackish water, polluted water, and water with high concentrations of chemicals. Some waterways may promote microbial induced corrosion (MIC) on submerged steel substructure members.

A build-up of debris at piers or abutments effectively widens the substructure unit and may cause scouring currents and increase the depth of scour (see Figure 16.2.9).

Figure 16.2.9 Buildup of Debris at Pier

Any evidence of deterioration or movement above the water line should 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. Susceptible bridges that have substructures underwater should be inspected following significant earthquake events.

Inspection Intervals

Inspections of the underwater portions of substructures must be inspected at regular intervals not to exceed 60 months unless the bridge meets the requirements for reduced or extended intervals (23 CFR 650.311 (b)(1)(i)).

Federal agencies, State, or Tribal governments must develop or document criteria to determine when the inspection interval should be reduced below 60 months. Factors to consider in this determination should include the following (23 CFR 650.311 (b)(1)(ii)(A)):

- Structure type.
- Structure design.
- Type of construction materials.
- Structure age.
- Condition ratings.
- Scour.
- Environment.
- Annual average daily traffic (AADT).
- Annual average daily truck traffic (AADTT).
- History of vehicle/vessel impact damage.
- Loads and safe load capacity.
- Other known deficiencies.

Bridges requiring Underwater inspection that meet any of the following criteria must be inspected at intervals no greater than 24 months (23 CFR 650.311 (b)(1)(ii)(B)):

- Underwater portions of the bridge are rated in serious or worse condition.
- Channel or channel protection is rated in serious or worse condition.
- The observed scour condition is three (3) or less.

It should be noted, where condition ratings are three (3) or less as a result of localized deficiencies, a Special inspection, as described in 23 CFR 650.313(h), limited to the deficient underwater portions of the structure may take place at the reduced interval in lieu of a complete Underwater inspection. A complete Underwater inspection must still be completed at regular 60 month intervals (23 CFR 650.311 (b)(1)(ii)(C)).

Certain underwater structural members may be inspected at greater than 60-month intervals, not to exceed 72 months, with FHWA approval (23 CFR 650.311 (b)(1)(iii)). This may be appropriate when past inspection findings and analysis justifies the increased inspection interval. If underwater portions of the bridge, channel, channel protection, and Scour Condition Rating are all coded "satisfactory" or better and if the Scour Vulnerability item is coded A or B, then the bridge is eligible for an extended Underwater inspection interval.

Agencies that decide to implement a 72 month extended Underwater inspection interval must develop an underwater extended interval policy and FHWA must be notified in writing prior to implementation. Factors to consider in this determination should include the following (23 CFR 650.311 (b)(1)(iii)(B)):

- Structure type.
- Structure design.
- Type of construction materials.
- Structure age.
- Condition ratings.
- Scour.
- Environment.
- Annual average daily traffic (AADT).
- Annual average daily truck traffic (AADTT).
- History of vehicle/vessel impact damage.
- Loads and safe load capacity.

Inspection intervals may also be determined by a rigorous risk assessment. If agencies decide to utilize this method, policies and criteria for inspection intervals based on risk assessment must be developed and submitted to FHWA for approval. The developed criteria must classify a bridge such that the Underwater inspection interval would not exceed 24, 60, or 72 months.

16.2.4 Qualifications of Diver-Inspectors

An underwater bridge inspection diver must complete FHWA-approved underwater bridge inspection training and score 70 percent or greater on an end-of-course assessment. Completion of FHWA-approved comprehensive bridge inspection training or FHWA-approved underwater bridge inspection training under FHWA regulations in effect before June 6, 2022, satisfies the intent of the requirement (23 CFR 650.309 (e)).

The Underwater inspection team should have knowledge and experience in bridge inspection. Conduct 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 of a qualified bridge inspection team leader.

The ability of the Underwater inspection team to safely access and remain at the underwater work site is paramount to a quality inspection. The individual should possess a combination of commercial diving training and experience as a working diver. This enables the inspection team 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. Refer to Chapter 1 for responsibilities of the bridge inspection team and 23 CFR Part 650, Subpart C, Section 650.309.

Federal Commercial Diving Regulations

Underwater bridge inspection, using 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 commercial diving operations performed both 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 diving and related support operations that are conducted in connection to diving. OSHA delineates the diving personnel requirements, including general qualifications of dive team members (OSHA 1910.410). The standard also provides general and specific procedures for diving operations and provides requirements and procedures for diving equipment and recordkeeping.

U.S Army Corp 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 necessary 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.

Divers should be adequately trained, which may include dive physiology, first aid, and cardiopulmonary resuscitation (CPR).

Employers should develop a safe diving practices manual consistent for OSHA commercial diving operations. Designate a person to oversee the operation who is qualified by training and experience. Minimum size dive team for both SCUBA and surface-supplied diving operations should be three, but more personnel may be necessary.

SCUBA gear is not recommended to be used for depths greater than 130 ft sea water. SCUBA gear is not recommended to be used in currents that are exceeding 1 knot unless diver is line tended. Surface-supplied air is not recommended for depths greater than 220 ft sea water. A recompression chamber should be on-site and ready for use for dives that exceed 100 ft sea water and for dives that are outside the no-decompression limits.

Minimum equipment requirements are specified for diver and diving equipment, and equipment testing. OSHA 1910.424 covers SCUBA diving and OSHA 1910.425 covers surface-supplied air diving.

Record keeping 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 (OSHA 1910.440).

Visit www.osha.gov and https://www.usace.army.mil/ 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 distinctions between recreational and commercial diver training. Though 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 (ANSI/ACDE-01-2009).

The American National Standard for Divers - Commercial Diver Training - Minimum Standard (ANSI/ACDE-01-2009) 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 before 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 Standards

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 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 goal is that the diver has knowledge and experience in underwater bridge inspection. The individual employers and bridge owners are in the best position to determine the specific requirements of their dive teams.

16.2.5 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:

- 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 typically most influential for first-time (Initial) Underwater inspections which set a benchmark for future inspections. Therefore, it is important to distinguish a plan between the Initial and follow-up Routine inspections.

When planning, the most helpful information will most likely be found in previously documented specialized procedures. Plans for future inspections can generally be developed after a thorough review of previous procedural needs. Refer to Chapter 2 for further details.

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 diving inspection intensity level. Depending on conditions and the type of damage found, a higher diving inspection intensity level may then be necessary to determine the actual bridge condition. It is also possible that different diving inspection intensity level efforts may be necessary at various locations on the same bridge.

The steps in planning an Underwater inspection include:

- Preliminary planning.
- Data collection and research.
- Hazard analysis.
- Develop a dive inspection operations plan.
- Risk assessment.
- Quality Control and Quality Assurance.

Preliminary Planning

Preliminary planning determines the goal of the inspection and decides what information is to be gathered and the amount of detail.

Data Collection and Research

Data collection and research is the next step taken 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 should be reviewed.

Hazard Analysis

Hazard analysis is recommended when planning an inspection. Refer to Section 16.2.8 for special considerations to be accounted for when planning Underwater inspections. The site should be examined to identify all potential hazards and identify ways to work around these hazards. Planning may include the avoidance or removal of the hazard, the selection of appropriate operational methods, scheduling during slack tides, the choice of the appropriate inspection and diving equipment and the use of special protective equipment. Common hazards may include swift current and tides, 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

A dive inspection operations plan should 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 should be used.

Risk Assessment

Risk assessment 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 should 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, checklists have been developed to aid the bridge owner and the dive team.

Checklists generally include the following items:

- Bridge identification.
- Marine information.
- Scour information.
- Structure information (e.g., type of substructure).
- Inspection information such as dates and condition ratings.
- Dive team certification requirements.
- Dive inspection level.
- Data to be included in the report.

16.2.6 Underwater Inspection Areas

The underwater portions of bridge structures to be inspected shall be included in the Underwater inspection procedures and can be classified into the following categories: bents, piers, abutments, cofferdams, protection devices, and culverts. Proper identification is important since various members may require different inspection procedures, levels of inspection, or inspection tools.

Bents, Piers, and Abutments

In most cases, the abutments are dry during low water periods and do not require a diving inspection. However, for some bridges the abutments remain continually submerged and an Underwater inspection should be necessary.

Piers are typically the most common substructure unit that will rely on Underwater inspections for important condition information. Protection devices are usually at least partially underwater, so it may be prudent to conduct a diving inspection in concert with the substructure unit inspection.

Important items that should be noted by the inspection team during Underwater inspections are collision damage to the substructure, and material deficiencies. Scour of the river bottom material at the bottom of the piles/footing can result in instability of the foundation. The Underwater inspection team compares present scour and resultant pile length/footing depth with that observed in previous inspections.

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.

Refer to Chapter 14 for detailed characteristics of abutments, wingwalls, piers and bents, multiple foundation types, and various substructure protection devices.

Cofferdams and Foundation Seals

Cofferdams and foundation seals are typically used to maintain a dry work area when constructing piers and abutments in water. Cofferdams are usually 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 is placed below the water on top of the soil and to prevent any uplift and flooding of the dewatered cofferdam.

Culverts

Some culverts may be filled with water year-round and do not provide an opportunity for a typical inspection. These would typically be inspected by divers. 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 completely submerged, or exhibit limited freeboard. In northern environments, winter inspections may also include ice as a contributing factor (see Figure 16.2.10). Diving operations in physically confined space should be conducted 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. Refer to Chapter 2 for confined space entry safety concerns.

Figure 16.2.10 Inspection of Culvert with Limited Freeboard and Ice cover

Submerged drift and debris can be 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 impacted. Use surface-supplied air diving equipment when conducting diving operations in physically confined and/or debris-laden culverts.

Refer to Chapter 15 for the characteristics of culverts.

16.2.7 Underwater Inspection for Material Deficiencies

This section outlines deficiencies specific to underwater conditions. Refer to Chapter 7 for detailed descriptions of material deficiencies.

Concrete

Plain, reinforced, and prestressed concrete are generally used in underwater members. Since the majority of substructures are basically compression units, concrete is a desirable 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 16.2.11).

Figure 16.2.11 Concrete Pile Deterioration

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 and abrasion occurs in the area near the water surface.

Masonry

Masonry can be used in substructure units but is seldom used as a material in newer bridges. Masonry substructures can experience splitting and delamination of the stones. Cracking of mortar joints at the normal waterline is a result of freeze-thaw damage.

Timber

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 untreated 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 can be particularly vulnerable if the treatment leaches out (which can happen 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 16.2.12).

Figure 16.2.12 Timber Pile Deterioration

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. Inspectors should take special care in differentiating between treated and untreated piles to ensure a thorough inspection of any exposed, untreated piles. With each inspection, the diameter or circumference for each timber pile should be noted. As a minimum, these measurements should be taken at the waterline and mud line. Inspectors should 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. Some sources recommend hammer soundings to detect internal damage. However, this method is unreliable in the underwater environment. One way to inspect for such damage is to take core samples. All bore holes should be plugged. Ultrasonic techniques for timber piling are also available.

Steel

Underwater steel structures may be highly sensitive to corrosion, particularly in the low to high water zone (see Figure 16.2.13). Whenever possible, inspect during periods of low water and measure steel to determine if section loss has occurred. Ultrasonic devices can be particularly useful to determine remaining steel thicknesses.

Figure 16.2.13 Steel Pile Deterioration Visible at Low Water Flow

Connections such as bolts, rivets, and welds should be examined for corrosion. If the steel members have a coating, the inspection team should 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 common in the future. Inspectors should check to see if the system appears to be working and should check the connections and power source.

Vessel Damage

Bridges in water may be susceptible to damage from vessels. Damage that happens from a vessel collision may be visible only above normal water elevation, but the extent of the underwater damage cannot be properly assessed without an In-depth 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 deterioration 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 scraper, knife, or ice pick, can be particularly useful for detecting small cracks. With limited visibility, the diver feels with their hands for damage. Because orientation and location are often difficult to maintain, the diver should be systematic in the inspection. Regular patterns from well-defined reference points should be established.

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. Measurements should be made from a consistent reference point on the bridge from inspection cycle to inspection cycle.

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

Any damage encountered should be measured. 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 before the dive. The waterline is not a good reference if its location is not known in respect to the fixed reference point.
- For undermining of foundations, inspectors should take enough measurements to define the zone no longer providing foundation support.
- 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).
- Inspectors should check for displacements or distortions of major members and whether they are plumb.
- The beginning and ends of cracks and intermediate points should be located as necessary to define the pattern of cracking.
- Inspectors should measure the maximum crack width and penetration depth.
- The length, width, and penetration of spalls or voids should be measured, making note of exposure and condition of any reinforcing steel.
- The degree of scaling on concrete should be noted.
- Inspectors should measure the thicknesses of all four flange tips on steel H-piles at distressed areas and specify the vertical location.

- Inspectors should locate buckles, bulges, and significant loss of section in steel members. Inspectors should accurately measure the thickness of remaining sound material when significant section loss is found.
- Damage at connections should be noted.
- The diameter of timber piles should be measured – the extent and width of checks, and extent of any decay should be noted.

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. Sufficient detail should be provided, as 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 members enable the inspection team to track any scour or streambed movement. A profile of the streambed can also provide information for tracking the development of scour and undermining.

In addition to sketches, the inspection team should 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, the inspection team should verify the repairs were made and that they have adequately addressed the previous deficiencies.

The results should be included in an inspection form or report. Drawings and text may be used to describe aspects of the inspection and any damage found. Recommendations on condition assessment, repairs, and time interval for the next inspection should be included in the report. An Underwater inspection form or report generally includes the following information:

- Bridge identification information.
- Date of inspection.
- Body of water.
- Dive techniques.
- Diving conditions.
- Unit inspected (e.g., pier columns, footings, culvert walls).
- Condition ratings.
- General inspection notes.

Condition ratings should be assigned for each substructure unit. These individual ratings should then be evaluated to assign the overall substructure rating code for the underwater portions of the bridge. The recommended substructure condition rating should be included in a summary portion of the report.

Refer to Chapter 2 for detailed descriptions of record keeping and documentation.

16.2.8 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 necessary. There may be times when tactile (by feel) inspections are all that can be accomplished, significantly compromising the condition evaluation of the member(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 self-contained or surface-supplied equipment, the diver may find it useful to adjust his/her underwater weight to near buoyancy and use swim fins for propulsion.

Dealing with Current

Most waterways have low flow periods when current should not hinder an inspection. Diving inspections should be planned 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 bridge substructure units or cofferdams (see Figure 16.2.14).

Figure 16.2.14 Diving Inside a Cofferdam

Waterway conditions may sometimes be too swift to provide safe diving operations (see Figure 16.2.15). For these conditions, other appropriate procedures should be used to evaluate the condition of underwater members or decide to return when the current is calm.

Figure 16.2.15 High Velocity Current

Dealing with Drift and Debris

The drift and debris that often collects at bridge substructures can be extensive (see Figure 16.2.16). This type of buildup typically consists of logs and limbs from trees which are usually matted or woven against or within the substructure members. Often this debris is found 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 significant underwater portions of the substructure.

Address concerns such as removal, past history, and safety when dealing with the presence of drift and debris when preparing for the Underwater inspection.

Figure 16.2.16 Debris Collection and Diver at Pier

Since drift and debris are often under the water surface, it is difficult to estimate the time and cost necessary to remove and gain access. The removal of the drift and debris is necessary if a hands-on inspection of the underwater members is to proceed. In some cases, debris can be removed by the inspection divers. However, heavy equipment, such as a hoist or underwater cutting devices, are often necessary. This debris removal can be considered when preparing for the inspection.

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, but others might 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 typically approach debris from the downstream side to avoid entanglements (see Figure 16.2.16).

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 member and may reach several inches or more in thickness (see Figure 16.2.17). The cost of cleaning heavily infested substructures may be completely impractical. In such cases, spot cleaning and inspection may be the only practical alternative.

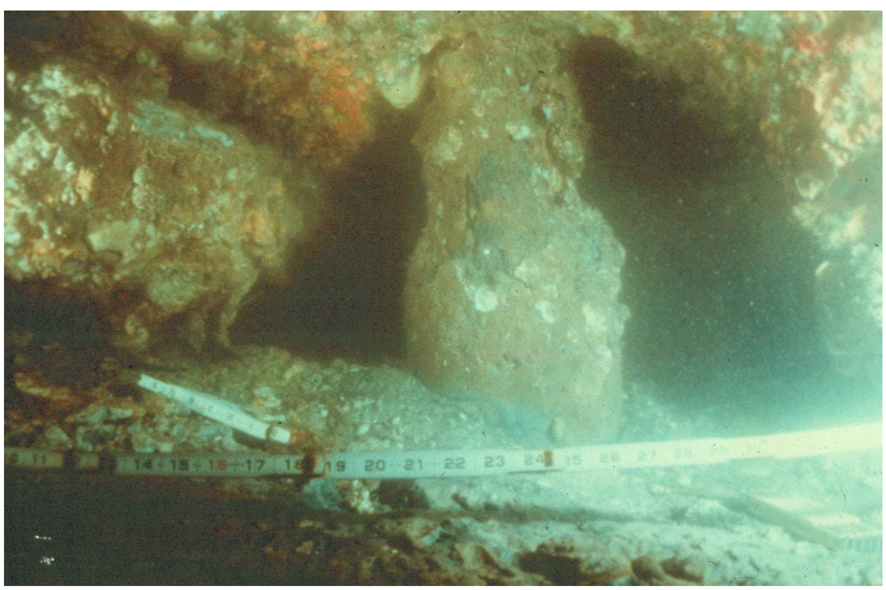


Figure 16.2.17 Marine Growth on a Timber Pile

Physical Limitations

Underwater conditions can create a cold, dark, hostile underwater environment that could 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 observation. Typical distractions include living organisms such as fish, snakes, 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 ft. 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 ft sea water (OSHA 1910.425(b)(2)).

Marine Traffic

Another concern for divers is vessel traffic near the area to be inspected (see Figure 16.2.18). Someone should always be topside with the responsibility of watching boat traffic. In addition, display flags indicating that a diver is down in the water. 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 (OSHA 1910.421(h)). However, it is also prudent to display the sport diver flag (white stripe on red), since it is more likely that recreational boaters may recognize this flag (see Figure 16.2.19).

Figure 16.2.18 Commercial Marine Traffic

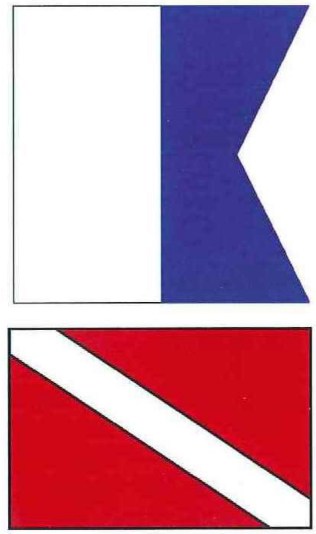


Figure 16.2.19 Diver in Water: International Flag (top) and Recreational Flag (bottom)

16.2.9 Requirements for Underwater Inspections

Bridge owners are responsible for identifying the location of underwater members including a description of the underwater members. They also verify if the inspection interval and procedures are adequate in accordance with the inspection record. Diving inspectors can make recommendations to improve the Underwater inspection procedures listed in the inspection record if conditions have changed.

The NBIS defines an Underwater inspection as the inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be visually inspected at low water by wading or probing, and generally requiring diving or other appropriate techniques (23 CFR 650.305). Though wading may be necessary and adequate to inspect bridge substructure units, wading should not be documented as an Underwater inspection. It should be completed during the Routine bridge inspection. Typical methods used to perform Underwater inspections include, but are not limited to:

- Commercial SCUBA.
- Surface-supplied diving.

Wading Inspection

Wading inspection is a basic method inspection to be used during a Routine inspection. Wading is used on structures over wadable streams. The substructure units and the waterway are typically 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 16.2.20). Again, wading inspection should not be recorded as an Underwater inspection.

Figure 16.2.20 Inspector Wading and Probing during Inspection

Commercial SCUBA

SCUBA, an acronym for Self-Contained Underwater Breathing Apparatus, is used for many Underwater inspections in this country (see Figure 16.2.21). 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.

Figure 16.2.21 SCUBA Inspection Diver

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.

Extreme care should be exercised 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 ft sea water and the time on the bottom should be limited by the amount of air the diver can carry and the amount of time based upon the no-decompression limits.

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. It also provides a safety tether line and hard-wire communications system connecting the diver and above water personnel (see Figure 16.2.22).

Figure 16.2.22 Surface-Supplied Diving Inspection

Depths of surface-supplied dives can be conducted down to 190 ft sea water or if the bottom times are less than 30 minutes, to a depth of 220 ft sea water. 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 limitations of this form of diving are that it requires more topside support than SCUBA and it is limited in mobility due to the connection to the surface.

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.
- Dives needing heavy physical exertion or of relatively long duration.

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 the sole criteria. Many factors, including the list below, 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.

16.2.10 Underwater Inspection Equipment

Reference OSHA directive *29 CFR Part 1910.430* and ADCI *Consensus Standards for Commercial Diving and Underwater Operations* for further information on the necessary equipment and proper use.

Diving Equipment

Personal diving equipment will likely include:

- Wet suit or dry suit (also known as an exposure suits) (see Figure 16.2.23).
- Face mask or helmet (see Figure 16.2.24).
- Buoyancy compensator (a flotation device).
- Breathing apparatus and/or reserve breathing air supply.
- Weight belt.
- Swim fins.
- Knife.
- Wristwatch.
- Depth gauge.
- Submersible pressure gauge.
- Flashlight or dive light.

Wet suits let a thin layer of water form 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 can be very effective in cold or polluted water.

Figure 16.2.23 Vulcanized Rubber Dry Suit

Figure 16.2.24 Full Face Lightweight Diving Mask with Communication System

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 Figure 16.2.25). 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 16.2.26).

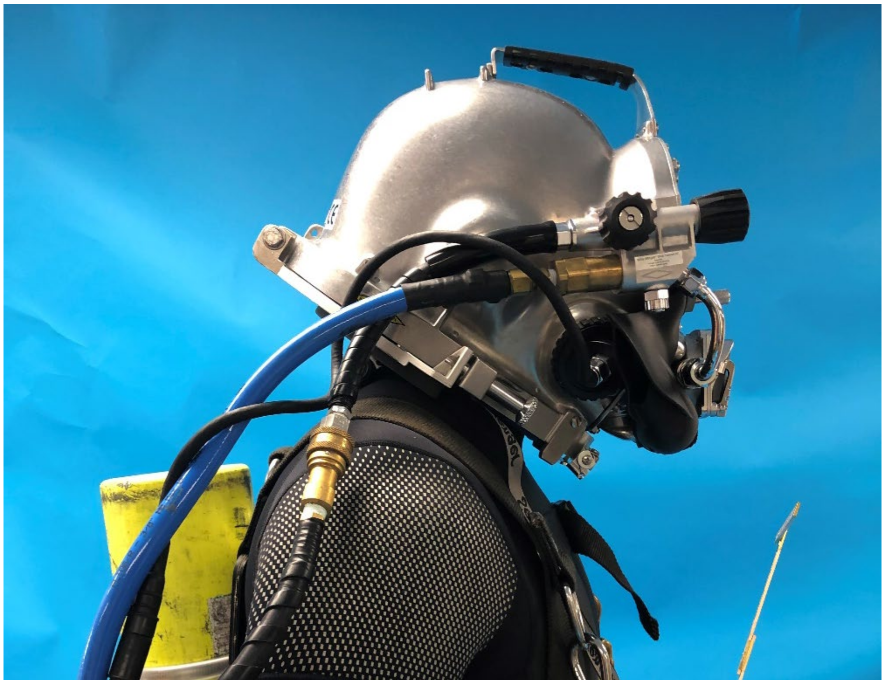


Figure 16.2.25 Surface-Supplied Diving Helmet

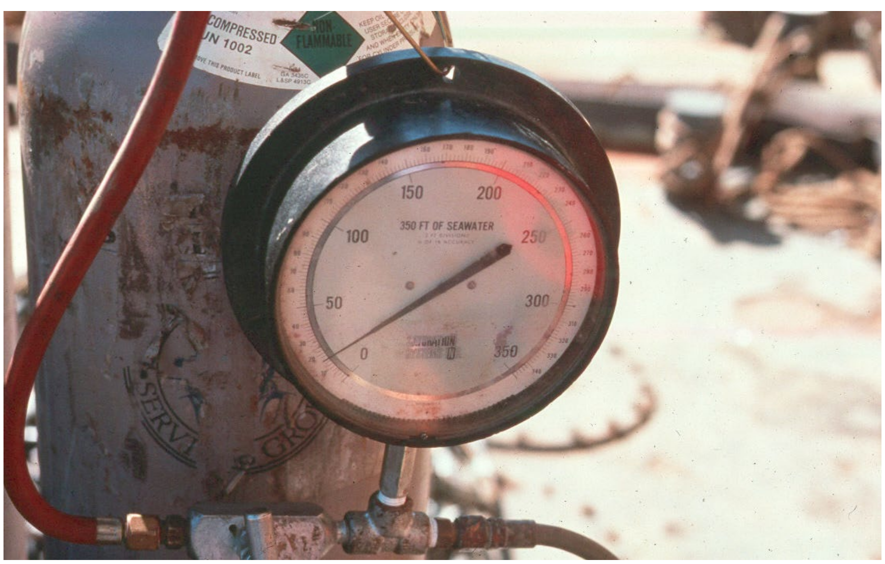


Figure 16.2.26 Pneumofathometer Gauge

For self-contained diving, the breathing gas supply is contained within a pressurized tank, which is carried by the diver.

Equipment malfunction leading to loss of air supply should be a constant concern to the dive team. Even in shallow water, submerged drift, and debris adjacent to a bridge can make an emergency ascent an arduous affair, for both the diver and the support team. As such, a reserve air supply should always be worn by the diver using surface supplied air (see Figure 16.2.27). Carbon monoxide poisoning can occur if the air intake of the surface supplied air compressor is near the exhaust of other motorized equipment.

Figure 16.2.27 Surface-Supplied Diver with a Reserve Air Tank

Surface Communications

A two-way communication system linking the diver(s) and topside personnel can greatly enhance the Underwater inspection, though this is not required in all situations. 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 enables the diver to have more mobility (see Figure 16.2.28) 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 enables an inspection team leader/engineer at the surface to discuss observations with a diver who is not yet an inspection team leader, to direct observation to specific zones, and to ensure that a satisfactory inspection is completed, according to the type and severity of damage found (see Figure 16.2.29).

Figure 16.2.28 Wireless Communication Box System

Figure 16.2.29 Surface Communication with Inspection Team Leader

Access Equipment

Inspection of short-span bridges can often be accessed from shore, but 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 Figure 16.2.30 and Figure 16.2.31). The boat should be equipped with an engine which should be dictated by the waterway conditions and the boat size.

Figure 16.2.30 Access Barge and Exit Ladder

Figure 16.2.31 Access from Dive Boat

Tools

A number of inspection tools are available. The dive team should 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

Most hand tools can be used underwater. The most useful can include the following:

- Rulers (see Figure 16.2.32).
- Calipers.
- Scrapers for removing build-up (see Figure 16.2.33).
- Probes (ice picks, dive knifes, and screwdrivers).
- Flashlight.
- Hammers (especially masonry and geologist's hammers) for sounding.
- Hand drills.
- Wire brushes for removing surface deposits or corrosion.
- Incremental borers.
- Hand saws or axes for removing debris.
- Pry bars.

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 16.2.32 Diver with a Pry Bar and 6-foot Ruler

Figure 16.2.33 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 underwater pneumatic tools that can be used include pneumatic drills, chippers, hammers, scalers, and saws. Pneumatic tools are typically limited to practical depths of 100 to 150 ft 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.

Pneumatic tools are sometimes used, but hydraulic tools tend to be favored for heavy or extensive work often necessary 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 pressure washing. This may be done during low flow to clean areas that are typically under water (see Figure 16.2.34). Particular care should be taken with such equipment to ensure that structural damage does not result from overzealous blasting.

Figure 16.2.34 Pressure Washing

Advanced Inspection Methods

When inspecting underwater members, nondestructive evaluations (NDE) and other methods may be necessary to determine the structural condition and may be used in Level III inspection. Refer to Chapter 17 for the principles of advanced inspection methods for various material types.

Steel

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 can be 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, and 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 generally 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, may cause a magnetic flux leakage field, which would 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 can also be difficult to implement due to the high currents and the poor clarity of inland water.

Concrete

An ultrasonic pulse velocity meter (or V-meter) is an ultrasonic device that requires two transducers and measures the distance necessary 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. The transducers should be on opposite sides of the member to provide the most accurate data when using direct transmission methods. Indirect transmission methods place the transducers on the same side of the member and utilize 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 uses a low frequency magnetic field to locate the steel.

Coring is classified as another advanced inspection method that provides a partially destructive evaluation method whose use is limited to critical areas. Cores can be taken in concrete or timber (see Figure 16.2.35).

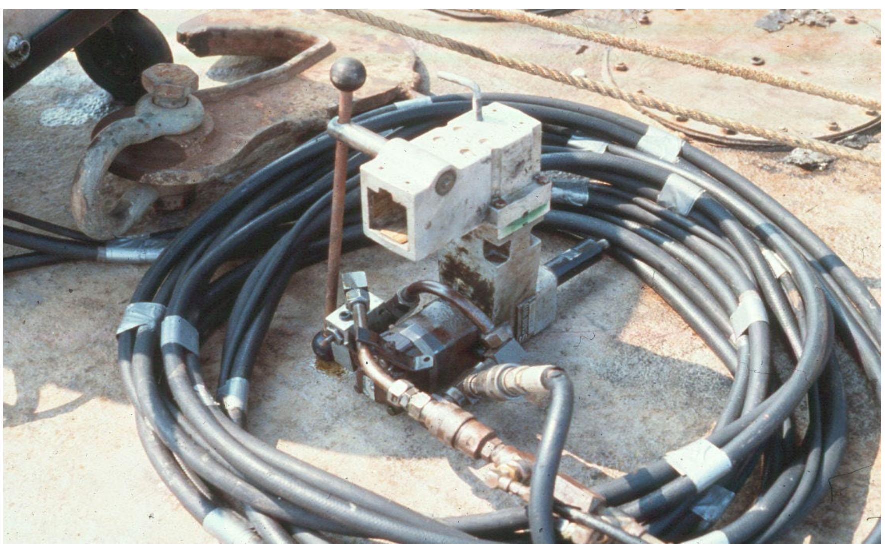


Figure 16.2.35 Underwater Coring Equipment

Concrete coring requires pneumatic or hydraulic equipment (see Figure 16.2.36). Deep cores, 3 ft or more in length, can be taken to provide an interior assessment of massive substructures (see Figure 16.2.37). Two-inch diameter cores are common, but coring tools are available in other sizes. 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. The core holes should be patched upon completion.

Figure 16.2.36 Concrete Coring Taking Place

Figure 16.2.37 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 16.2.38). Power tools are sometimes used, but the most effective procedure is usually 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. The hole should always be plugged with a treated hardwood dowel to prevent infestation.

Figure 16.2.38 Timber Core

Underwater Imaging

There are multiple options to provide underwater images of the bridge or substructure unit that is being inspected.

Photography

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 16.2.39 and Figure 16.2.40). There are also waterproof cameras that are designed specifically for underwater photography.

Figure 16.2.39 Various Waterproof Camera Housings

Figure 16.2.40 Diver Using a Camera in a Waterproof Housing

In some cases, visibility is limited, and the camera should 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 16.2.41). A clearwater box is a clear plastic box that is filled with clean water. The box can be placed adjacent to the subject, which displaces the dirty water and enabling the camera to focus on the member being photographed.

Figure 16.2.41 Diver Using a Clearwater Box

Video

Video equipment is available 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 16.2.42). The latter type enables a surface operator to direct shooting. At the same time, 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 soundtrack is linked to the communication equipment, a running audio commentary can also be created.

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 16.2.42 Underwater Video Inspections

Remote Operated Vehicle (ROV)

An extension of the video camera is a remotely operated vehicle (ROV) where the camera is mounted on a surface-controlled propulsion system (see Figure 16.2.43). 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 with controlling the vehicle due to the current or the umbilical being tangled. The ROV cannot perform cleaning operations before photos being taken.

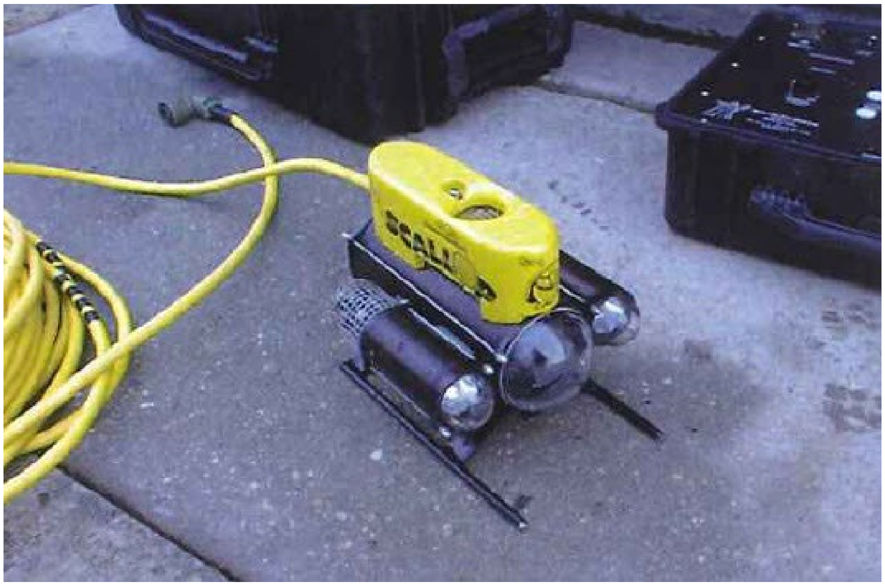


Figure 16.2.43 Remote Operated Vehicle (ROV)

ROVs may take the form of float-out devices, as discussed in Section 16.1.7, Advanced Inspection Methods. AUVs and USVs are both examples of vehicles that are remotely operated and used on the surface or underwater to gather imaging data without the necessity of human access.

Underwater Acoustic Imaging

Underwater acoustic imaging is a form of sonar can provide greatly improved images of the channel bottom conditions, undermining and submerged foundations (see Figure 16.2.44).

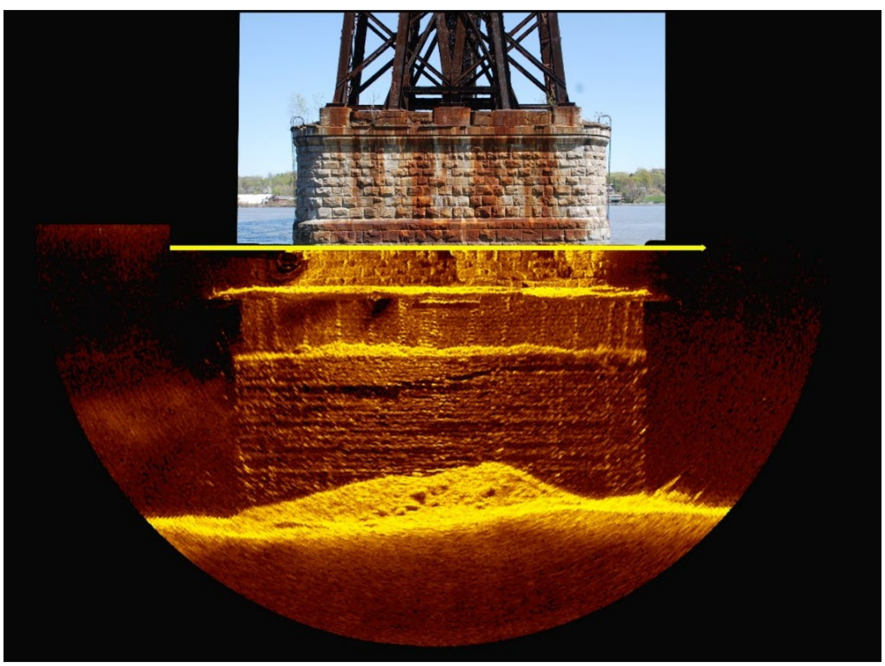


Figure 16.2.44 Acoustic Imaging of a Pier

This can aid in the planning of diving operations by detecting areas of possible damage. This may enable the divers to concentrate on these areas. It can also enhance diver safety by identifying potential dive hazards before divers enter the water. Acoustic imaging can also provide images of an underwater member that an underwater camera may not be able to take due to the turbidity of the water. Imaging can operate at distances of 200 ft. However, cameras, even in fairly clear water, have an effective range of only a few ft.

Underwater acoustic imaging is also useful when an emergency evaluation of a bridge may be necessary after a bridge is damaged by a collision, significant weather, or an earthquake, especially if the water conditions (e.g., high current, low visibility, debris) preclude the use of divers.

16.2.11 Underwater Instruments to Determine Scour

Divers may be able to note scour under certain conditions, but sometimes it may be necessary to use alternative methods when diving is not feasible. 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 may be necessary. 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. Refer to Advanced Inspection Methods in Section 16.1.6 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.

A fathometer consists of a transducer that is suspended in the water, a sending/receiving device, and a recording device that should display the depth on paper or a display. It can be 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.

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 is also used in scour surveys. GPR can be used to obtain high resolution, continuous subsurface profiles on land or in shallow water which is less than 25 ft deep. GPR transmits short electromagnetic pulses into the subsurface and measures 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 should be sent back to the surface and the rest should be sent into deeper layers. These are typically not as effective when encountering material that is highly conductive (e.g., clay), in saltwater, or water with heavy amounts of sediment.

Two-Dimensional Sonar Systems

In basic terms, sonar utilizes sound, or acoustic waves, to determine the distance to an object from the source of the wave. Generally, a transducer is used for this purpose. The transducer emits the sound through the water and then measures the amount of time it takes to return.

Two-dimensional (2D) sonar systems utilize a flat, fan-shaped array or beam (see Figure 16.2.45). The return measurements are generally used to plot a 2D drawing of the underwater surface. The orientation of the array is changed to obtain different views.

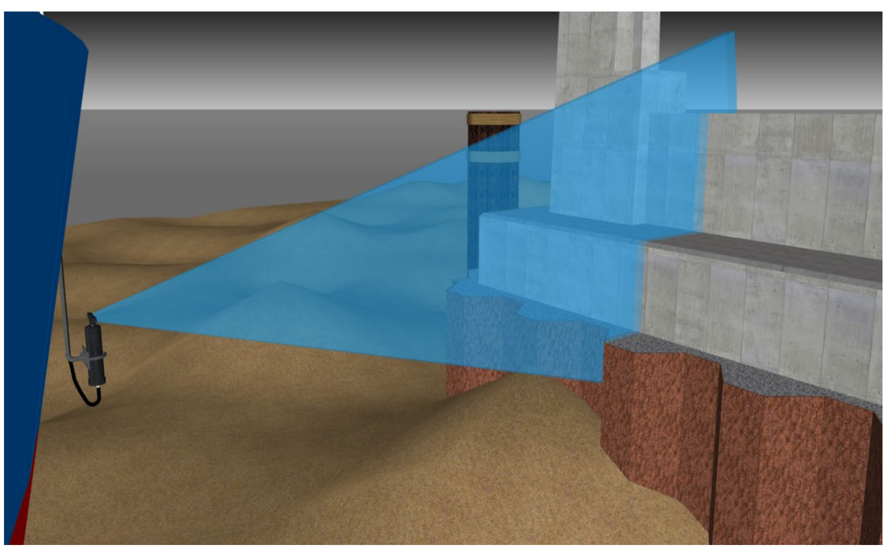


Figure 16.2.45 2D Sonar System Vertical Beam

The three types of 2D sonar systems are listed below:

- Side-scan.
- Sector-scan.
- Lens-based multibeam.

Reference *Underwater Inspection of Bridge Substructures Using Imaging Technology, FHWA-HIF-18-049* for detailed information.

Three-Dimensional Sonar Systems

Three-dimensional (3D) sonar systems are generally able to gather more data in a shorter time period, based on the coverage area. As compared to a singular plane of data provided with 2D sonar, 3D data consists of more data points that each have x, y, and z coordinates. This provides for a 3D rendering of the underwater surfaces.

Available 3D sonar systems include:

- Fathometers/Echosounders.
- Geophysical sub-bottom profilers.
- Multibeam swath sonar.
- Real-time multibeam sonar (see Figure 16.2.46).

Reference *Underwater Inspection of Bridge Substructures Using Imaging Technology, FHWA-HIF-18-049* for detailed information.

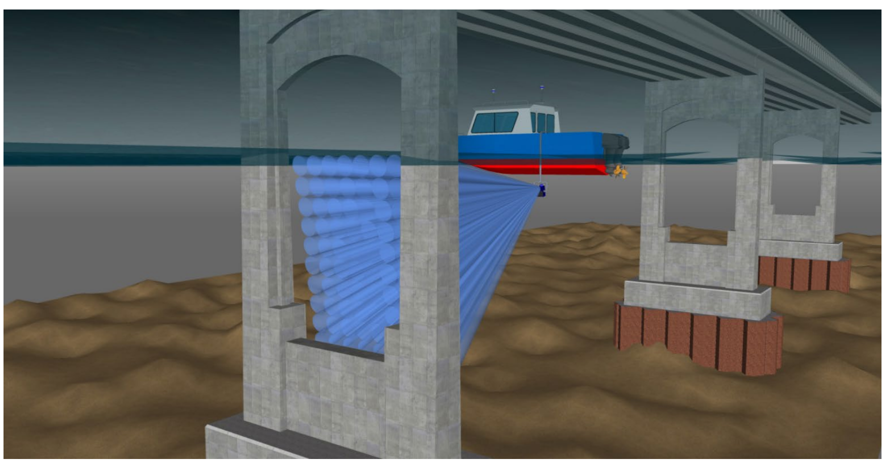


Figure 16.2.46 Real-time Multibeam Sonar Pattern

Underwater Inspections for Scour

When identifying scour vulnerability or monitoring 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. Underwater inspections of scour and undermining may include:

- Recording bottom conditions adjacent to submerged foundations.
- Detecting undermining and scour holes near the upstream end of the foundation (see Figure 16.2.47).
- Detecting soil build-up soil at downstream end.
- Noting any debris that may cause local scour.
- Noting type of bottom material.
- Noting the presence, location, and size of riprap.
- Detecting small diameter, deep scour holes that expose the footing or cause undermining.
- Recording dimensions of undermining, if it exists.
- Noting any piles to be examined if they are exposed.

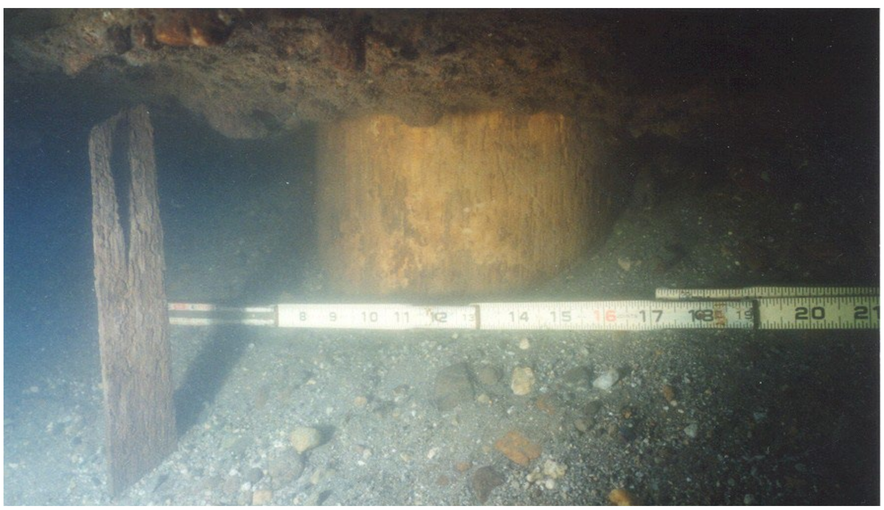


Figure 16.2.47 Pier Undermining, Exposing Timber Foundation Pile

PART III – BRIDGE, CULVERT, AND WATERWAY INSPECTION AND EVALUATION

CHAPTER 17 TABLE OF CONTENTS

CHAPTER 17 LIST OF FIGURES

Chapter 17 Advanced Inspection Methods

Section 17.1 Introduction

Advanced inspection methods give inspectors the ability to further evaluate suspected defects found during a visual or physical type of inspection. According to the NBIS, for a hands-on inspection which is an inspection within arm's length of the member, visual techniques are used that may be supplemented by nondestructive evaluation (NDE) techniques (23 CFR 650.305). For an in-depth inspection, which is a close-up, detailed inspection of one or more bridge members located above or below water, visual or nondestructive evaluation techniques are used as required to identify any deficiencies not readily detectable using routine inspection procedures (23 CFR 650.305). Advanced inspection methods usually demand the use of calibrated testing equipment, a professionally trained technician to perform the testing, and a professional with expertise in interpreting the results. Even though such expertise is necessary to perform the methods, it is beneficial for the bridge inspector to understand the typical applications and limitations of various advanced inspection methods.

New technology is making the use of these highly technical systems more economically feasible for bridge inspections. Advanced inspection methods are becoming necessary for the various types of bridge members. The *Specifications for the National Bridge Inventory (SNBI)* now incorporates the reporting of advanced inspection equipment. *SNBI* Item B.IE.12, Inspection Equipment is used to report the type of advanced inspection methods or testing used to perform the inspection (*SNBI* Subsection 6.2).

There are generally two main classifications of advanced inspection methods discussed, nondestructive and other evaluation methods. The following sections provide a description of the testing methods for each of the different bridge material types. Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

Section 17.2 Concrete

17.2.1 Nondestructive Evaluation Methods

Nondestructive evaluation or nondestructive testing (NDT) refers to advanced inspection methods that do not impair the functionality or serviceability of a structural member.

The FHWA InfoTechnology website provides further, more in-depth, information on many of the concrete NDE methods discussed in this section. The website can be found at https://infotechnology.fhwa.dot.gov/.

The following methods are detailed on the InfoTechnology website, but not included in this section. Reference the website for further information on:

- Active infrared thermography.
- Displacement gauge.
- Radiography.

Electrical Resistivity (ER)

The electrical resistivity method evaluates the presence of corrosive substances, such as chlorides and salt solutions, in concrete. High moisture level and chloride concentrations are generally due to cracking or otherwise damaged concrete surfaces that have enabled fluid to enter into the material and in turn, facilitates ion flow. Higher chloride concentrations lead to higher concrete electrical conductivity or lower resistivity.

ER utilizes an instrument called a Wenner probe to introduce an electric current into the concrete (see Figure 17.2.1). The voltage and current flow are measured, and the ER is calculated. ER can indicate a corrosive environment in the concrete and also be linked to a corrosion risk of the steel reinforcement. Typically, the lower the electrical resistivity, the more corrosive the environment.

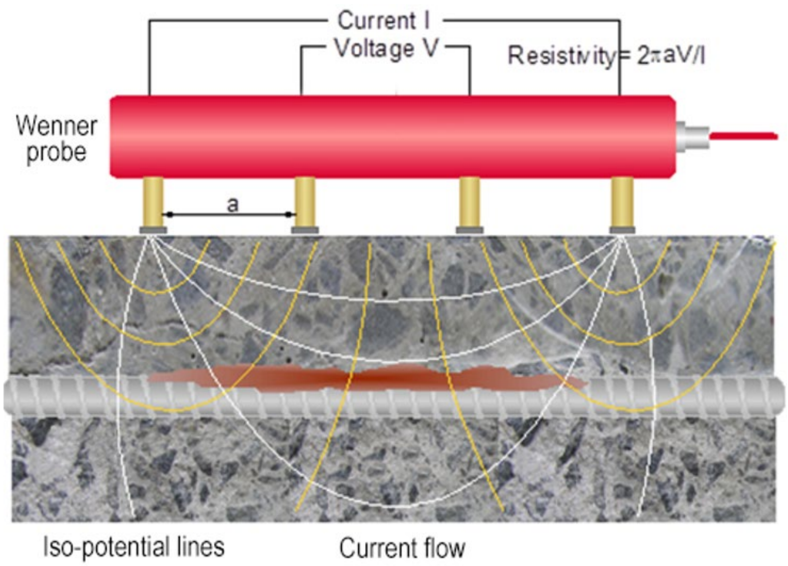


Figure 17.2.1 Schematic Representation of Electrical Resistivity

Galvanostatic Pulse Measurement (GPM)

Galvanostatic Pulse Measurement can be used to estimate the corrosion rate of reinforcing steel in concrete members. The method uses small currents to measure potential change in the polarization of rebar. The corrosion rate and resistance, along with the half-cell potential, can be determined by measuring the difference in currents between reference electrode with no corrosion and a section with suspected corrosion (see Figure 17.2.2). GPM uses an electrode device on the surface of the concrete and another reference electrode that is typically attached directly to the steel rebar.

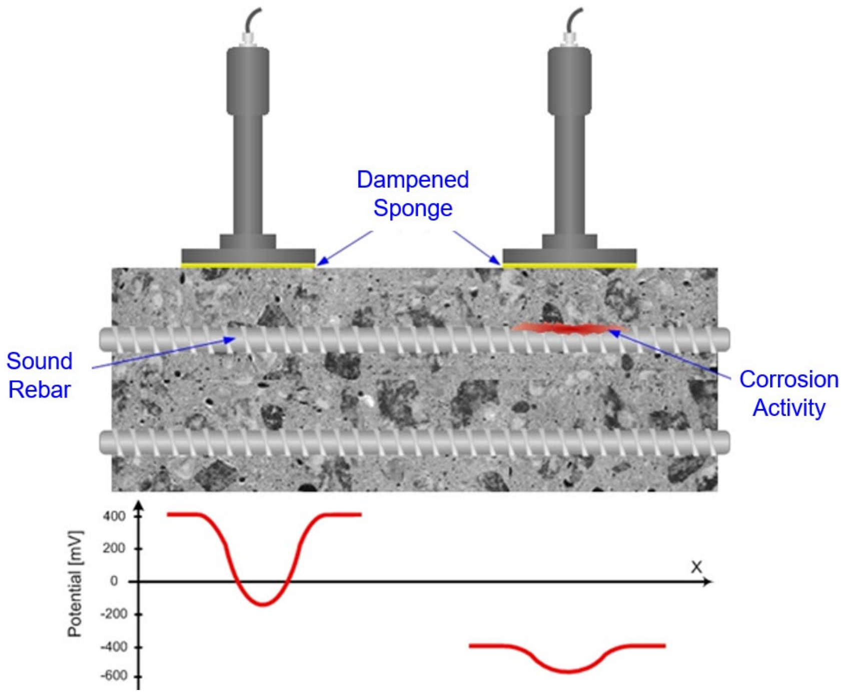


Figure 17.2.2 Schematic Representation of GPM of Corrosion Activity

Ground Penetrating Radar (GPR)

Ground-Penetrating Radar is an electromagnetic method that uses high-frequency electromagnetic waves to acquire subsurface information. GPR can be used to identify and map areas of potential corrosion-based deterioration, establish structural reinforcement layout, and estimate the thickness of the deck, overlays, or reinforcement cover. By assuming the speed of electromagnetic waves in asphalt, GPR results can be used to estimate the thickness of asphalt covering and examine the condition of the top flange of concrete box beams that may otherwise be inaccessible.

An electromagnetic wave is radiated from a transmitting antenna, and travels through material at a velocity which is determined primarily by the electrical properties of the material. The wave propagates downward; however, materials with different electrical properties can alter its path. Upon encountering an anomaly with different electrical properties, part of the wave energy is reflected or scattered back to the surface. At the same time, the rest of the energy continues its downward path (see Figure 17.2.3). The wave that is reflected back to the surface is captured by a receiving antenna and recorded on a digital storage device for postprocessing and interpretation. The most common display of GPR data is one showing amplitude versus time and is referred to as a trace. A GPR trace consists of the directly transmitted pulse energy followed by pulses that are received from reflecting objects or layers.

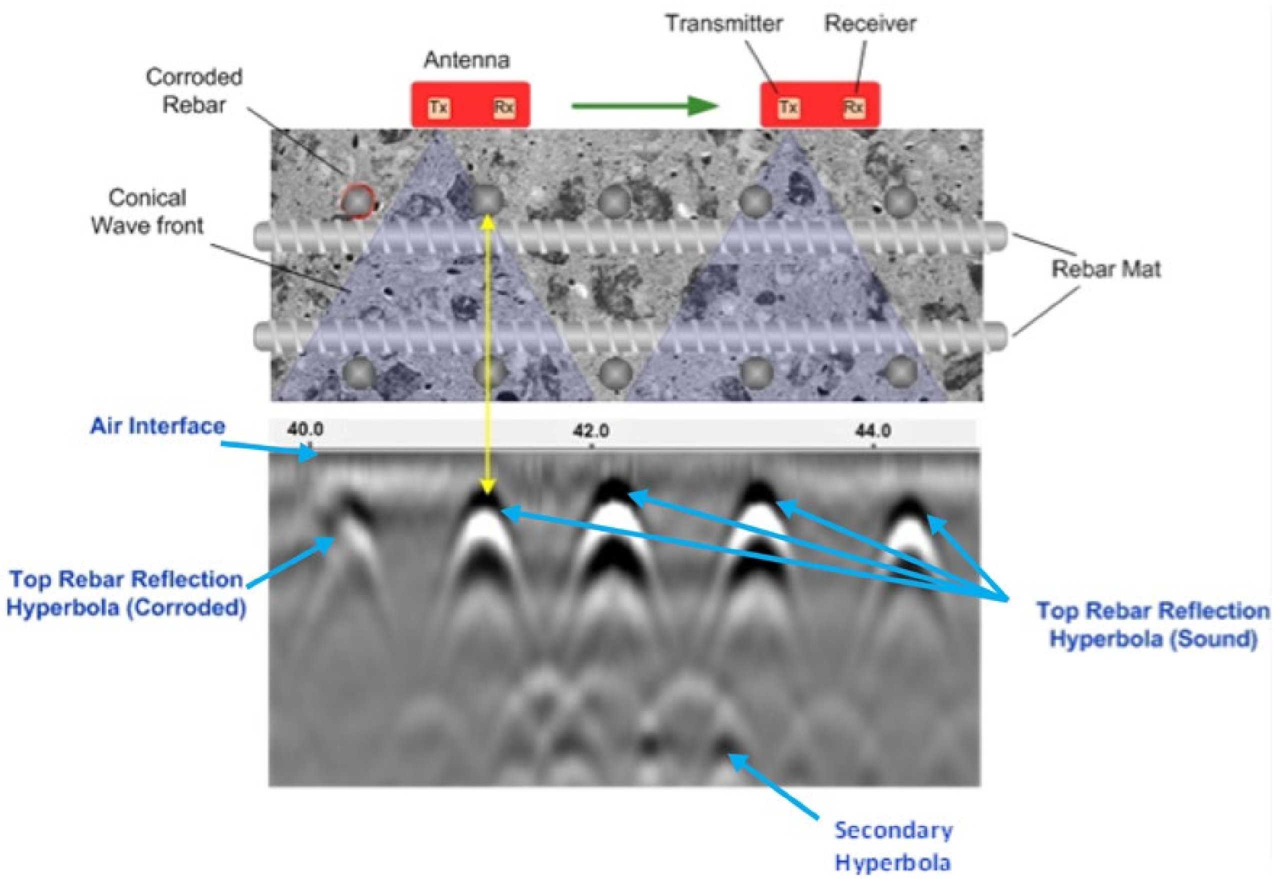


Figure 17.2.3 Schematic Representation of Ground-Penetrating Radar Method

GPR is typically used for the detection of delaminated areas and flaws in reinforced concrete bridge members; location and depth of reinforcing, and location of post-tensioning ducts in prestressed concrete members. Other applications not related to concrete inspection can include mapping geologic conditions that include depth to bedrock, depth to the water table, depth and thickness of soil and sediment strata on land and under freshwater bodies, and the location of subsurface cavities and fractures in bedrock. GPR can also be used in the location of objects such as pipes, drums, tanks, cables, and boulders. This technology has been also used for concrete bridge members and tunnel lining condition assessments.

Importantly, GPR is truly a nondestructive technology, as opposed to the destructive activity of 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.

Depending on the desired results of the GPR testing, the data acquisition procedure, and the processing and interpretation of that data, slightly differ. Detecting delamination, finding voids, and locating the reinforcement are typically the three main purposes of GPR. Further details about GPR can be found in the FHWA InfoTechnology website.

Half-Cell Potential (HCP)

Half-cell potential tests are a well-established and widely used electrochemical technique to evaluate active corrosion in steel reinforced and pre-stressed concrete structures. (see Figure 17.2.4). A copper copper-sulfate ($Cu/CuSO4$) electrode is typically used for the half-cell reference. Reinforcing bar networks can be physically accessed and wired for current detection. Half-cell electrical potentials of reinforcing steel are typically measured by moving the electrode over the concrete surface. As the electrode contacts concrete over an actively corroding rebar, voltage is registered. Measured potential values reflect levels of corrosion activity in the rebar. Lower potential measurements more negative than -350mV indicate higher likelihood of corrosion. The HCP survey can be used to determine the location where core sample should be taken.

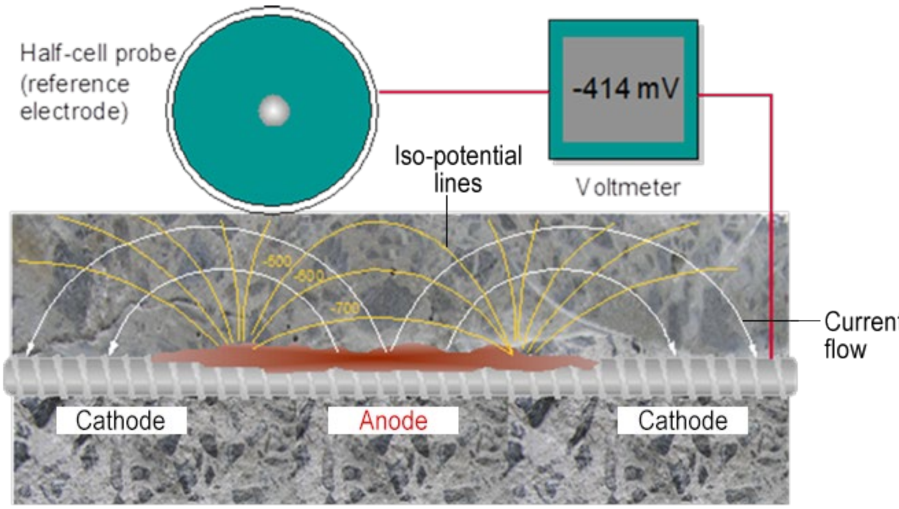


Figure 17.2.4 Schematic Representation of Half-Cell Potential Test

Impact Echo (IE)

Impact-echo evaluates the condition of concrete and masonry structures by applying an impact to generate stress (sound) waves. The resulting waves propagate through the structure and are reflected by internal flaws and external surfaces (see Figure 17.2.5).

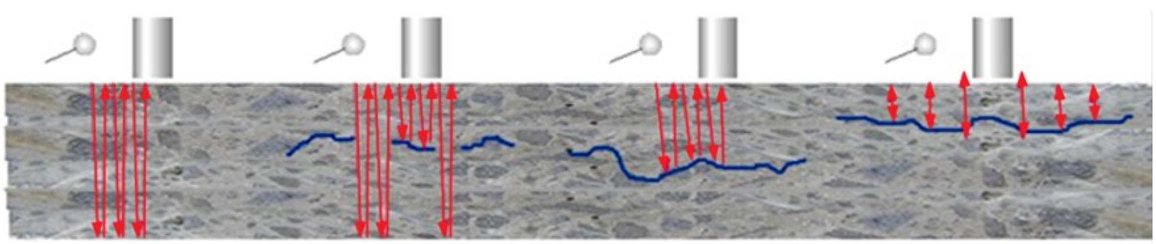


Figure 17.2.5 Schematic Representation of Impact Echo Test

This method can be used to determine the location and extent of flaws such as delaminated areas, voids, honeycombing and debonding in plain, reinforced, and post-tensioned concrete structures. It can find voids in the subgrade beneath slabs and pavements. It can be used to determine member thickness or find voids, and other defects in masonry structures where the brick or block units are bonded with mortar. This method is generally 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. Due to wavelengths substantially larger than the size of aggregates or reinforcement, 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 placed 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 into 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.

Infrared Thermography (IT)

NDE inspection using infrared thermography is generally based on imaging surface temperatures of a specimen to infer subsurface delaminated areas or defects (see Figure 17.2.6).

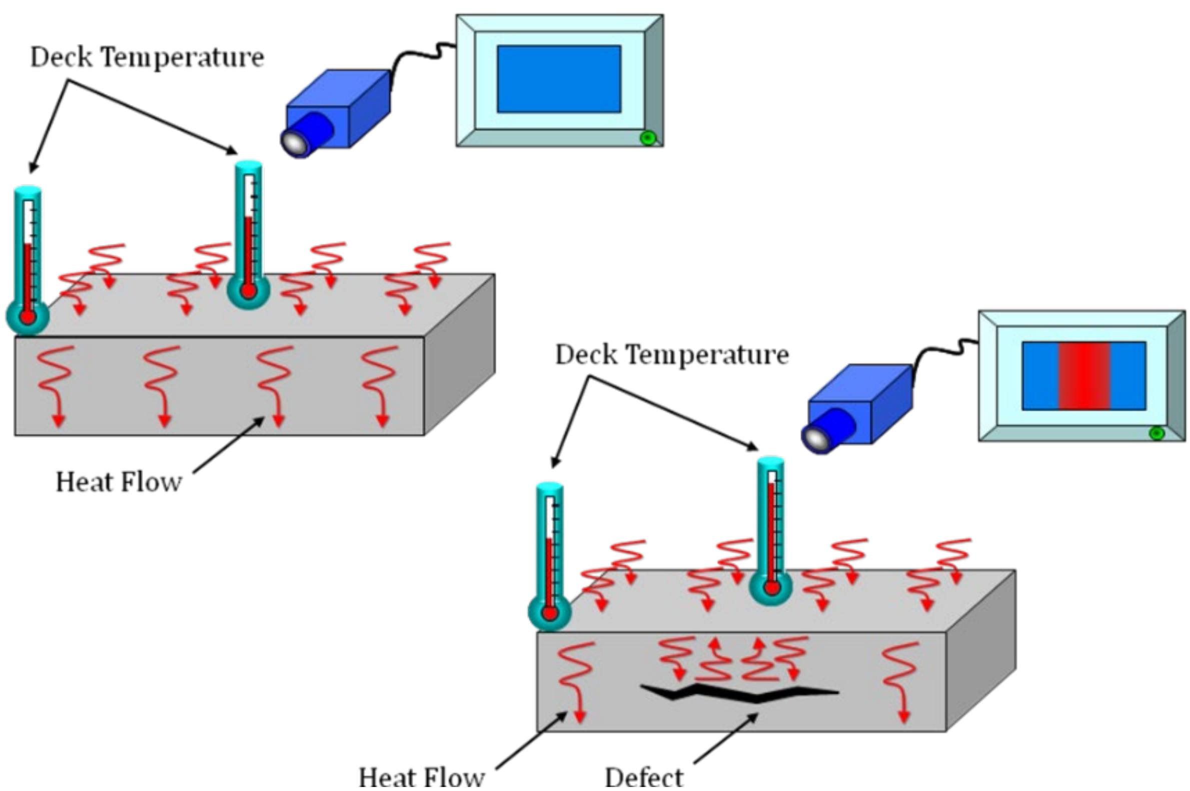


Figure 17.2.6 Schematic Representation of Thermal Imaging Technique

The basic theory is that heat conduction through a material is altered in the presence of defects. For example, the temperature of the deck is typically greater than the surrounding air. With no internal defect, heat flow through the deck, and subsequently an image of the surface temperature of the deck, should be relatively uniform. If a delamination is present, the heat flow can be altered. As shown below, the surface of the deck above the delamination appears to be higher in temperature than the remainder of the deck (see Figure 17.2.7).

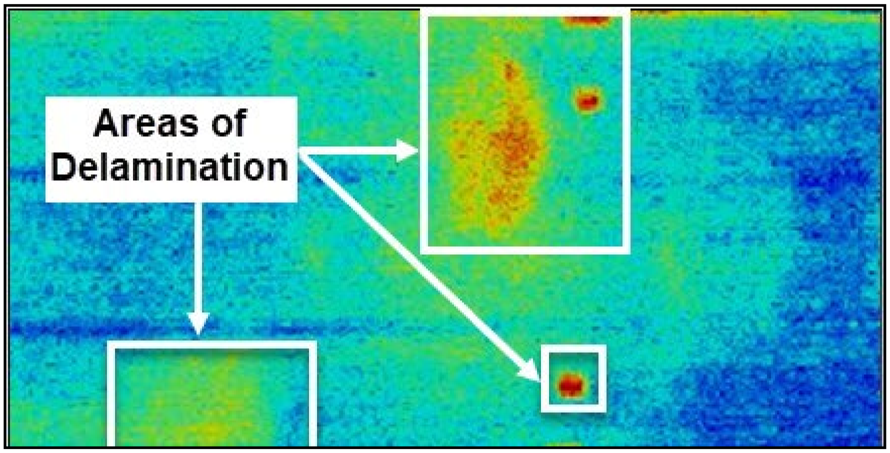


Figure 17.2.7 Deck with Areas of Delamination

Thermographic measurements can be complicated by a number of issues. Probably the most significant is that a thermal camera does not directly measure the temperature of a specimen. Instead, the camera measures a radiant flux that should 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 as thermal radiation. Two objects at the same temperature but with a different emissivity 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.

Linear Polarization (LPR)

The linear polarization method is generally used to measure the active rate of corrosion of the reinforcement. LPR uses an electrode on the concrete surface and an electrical connection to the steel reinforcing and measures the current between them (see Figure 17.2.8). The technique is semi nondestructive but necessitates localized damage to the concrete cover to enable an electrical connection to be made to the reinforcing steel. Monitoring the relationship between electrochemical potential and current generated between the electrically charged electrodes under the test enables the calculation of the corrosion rate. Because results may be dependent upon the environmental conditions at the time of testing, it is generally recommended to perform repeated testing over time. Results up to 0.2 $\mu A/cm^2$ indicate a passive/very low corrosion classification rate with values of >1.0 $\mu A/cm^2$ indicating a very high corrosion rate.

The results of the test are highly dependent on the degree of chloride contamination and therefore LPR is often used in combination with half-cell potential or electrical resistivity. If reinforcing steel is experiencing corrosion in a smaller, localized area, generally due to a high level of chlorides, the measured active rate will likely be skewed. The test is typically most accurate when rebar is corroding uniformly. Also, this test is not applicable for use with epoxy coated rebar.

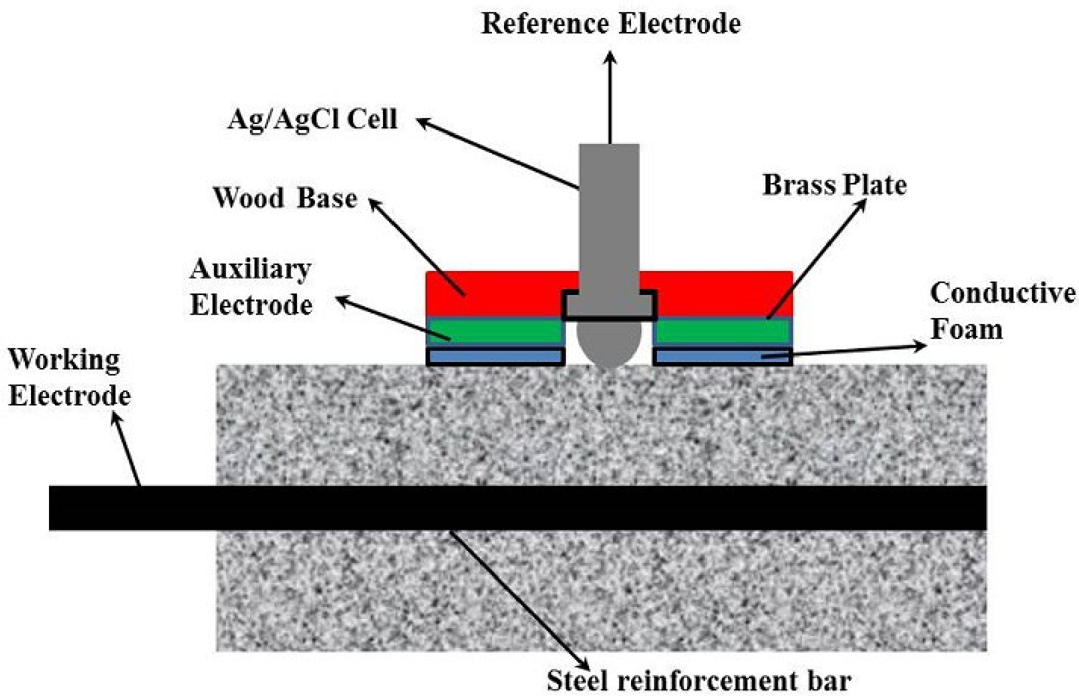


Figure 17.2.8 Schematic Representation of Linear Polarization Test

Magnetic Flux Leakage (MFL)

The MFL method can be used to detect the location and extent of corrosion in post-tensioned and precast prestressed strands in concrete girders, and breakage of wires and strands in post-tensioning tendons and prestressing strands. MFL is also commonly used to test the cables of suspension or cable stayed bridges. MFL units can be clamped onto a cable as part of a climbing module or rolled across a surface.

The magnetic flux leakage method uses a magnetic field to detect flaws in the steel embedded in concrete. The process is completed by magnetizing the rebar or prestressing strands with an external magnet. A magnetic field is then created with sensors on the surface of the concrete (see Figure 17.2.9). Any disturbance in the field reflects a flaw in the steel.

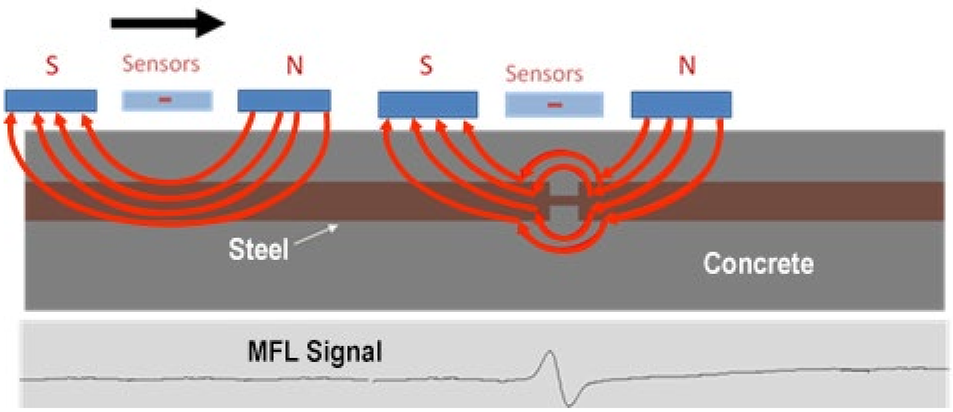


Figure 17.2.9 Schematic Representation of Magnetic Flux Leakage Technique

Magnetometer (MM)

Magnetometers or profometers (see Figure 17.2.10) are generally used to determine the approximate size and orientation of the reinforcing steel in bridge decks and other concrete members. They can also detect the concrete cover depth. MM methods do not detect concrete defects or rebar deterioration directly. However, they can detect regions of inadequate cover. Inadequate cover typically increases the ability for oxygen, moisture, and corrosive agents to reach the reinforcing steel, which can lead to corrosion-induced deterioration. MM is only effective within top four inches of the concrete surface and only the first layer of reinforcing steel may be found and sized.

Figure 17.2.10 Locating Rebar with a Magnetometer

Magnetometers were previously known as pachometers or covermeters. They generally work by creating a magnetic field that is uninterrupted by non-conductive materials (i.e., concrete). An eddy current is produced by any conductive material such as steel and can, therefore, be detected.

Rebound and Penetration Methods

Rebound and penetration methods measure the hardness of concrete and can be used to estimate the strength of concrete.

The rebound hammer (also known as the Schmidt or Swiss hammer) is probably the most commonly used device to provide a measure of surface hardness of concrete. A spring-loaded device (see Figure 17.2.11) strikes the surface of the concrete, and based on the response (rebound), the compressive strength of the concrete can be determined when compared to a rebound index. This inspection method can be used to compare the quality of the concrete in different parts of concrete bridge members. However, only the surface of the concrete is being tested, and the strength value is relative.

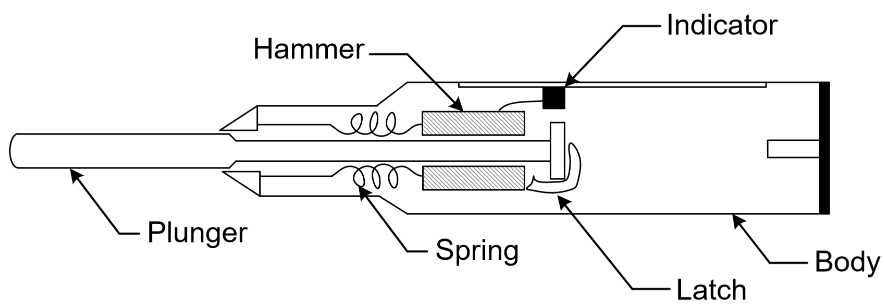


Figure 17.2.11 Schematic of a Rebound Hammer

A common penetration resistance test utilizes a pistol-like driving device, known as a Windsor Probe, that fires a probe into the surface of the concrete (see Figure 17.2.12). The probe is specifically designed to crack aggregate particles and to compress the concrete being tested. Penetration resistance test can be conducted to estimate the strength of concrete on-site for early form removal or to investigate the strength of concrete in place because of low cylinder test results.

Both tests are primarily applied to 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 the concrete strength of older bridges.

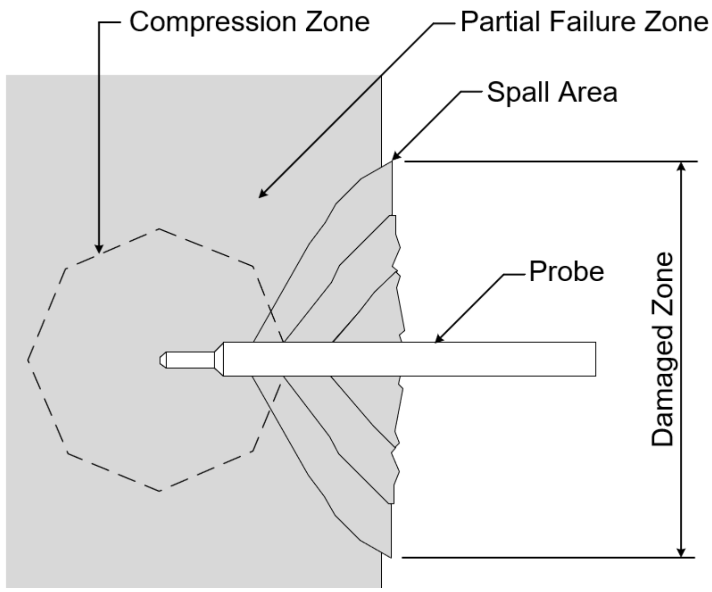


Figure 17.2.12 Schematic Representation of Windsor Probe Penetration Test

Ultrasonic Pulse Velocity (UPV)

Ultrasonic pulse velocity methods are generally used to evaluate the relative quality of concrete and estimate compressive strength. The pulses pass through the concrete and the transit time is then measured (see Figure 17.2.13). The pulse velocity is then interpreted to evaluate the quality of the concrete and to estimate in-place concrete compressive strength. Comparatively higher velocity is measured when concrete quality is good. The quality of results from the test can be affected by path length, lateral dimension of the specimen tested, existence of reinforcement steel and moisture content of the concrete.

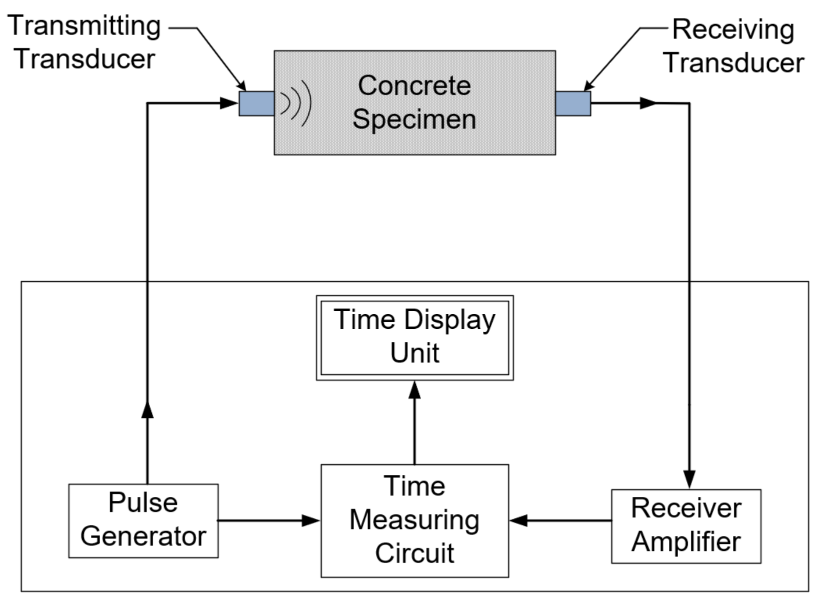


Figure 17.2.13 Schematic Representation of Ultrasonic Pulse Velocity

This equipment analyzes concrete in bridge decks by measuring the velocity of sound waves. Ultrasonic pulse velocity test consists of measuring travel of ultrasonic pulse produced by an electro-acoustical transducer. The time for the waves to return depends on the integrity of the concrete.

Ultrasonic Pulse Echo (UPE)

Ultrasonic Pulse Echo evaluation is a method in which a transducer emits a shear wave/pulse, and records the travel time/echo back to the device (see Figure 17.2.14). As the acoustic energy propagates through the concrete, depending on the frequency, a discontinuity (i.e., a crack) should return the signal to the receiver. If there are no cracks or voids in the concrete, the wave may be enabled to travel through the full depth of the material. The velocity of the waves is known and can be used in combination with the travel time to calculate depth.

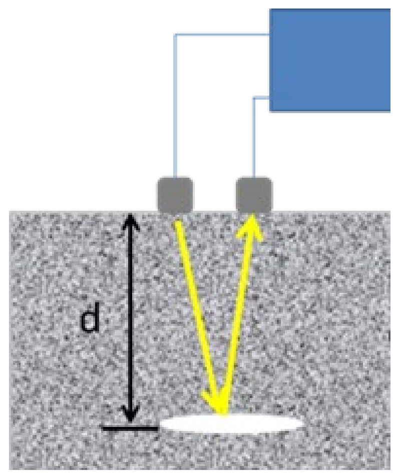


Figure 17.2.14 Schematic Representation of Ultrasonic Pulse Echo Methodology

UPE is commonly used for measuring unknown thicknesses and locating voids and delaminations in concrete members. It can also be used to find prestressing tendon ducts or find voids in grouted post-tensioning grouted ducts.

Ultrasonic Surface Waves (USW)

Evaluation of concrete decks can be accomplished with sonic or ultrasonic acoustic wave velocity measurements. This method delineates areas of internal cracking (including delaminated areas) 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 principal part of a computer-based monitoring and recording system for a detailed evaluation of bridge decks and other bridge members (see Figure 17.2.15). Bridge abutments and concrete support members are tested using the same recording system with a portable, hand-held sensor array. The system works directly on bare concrete or through wearing surfaces such as asphalt. It can distinguish between debonded asphalt and delaminated areas, and it may be effective for a detailed evaluation of large areas.

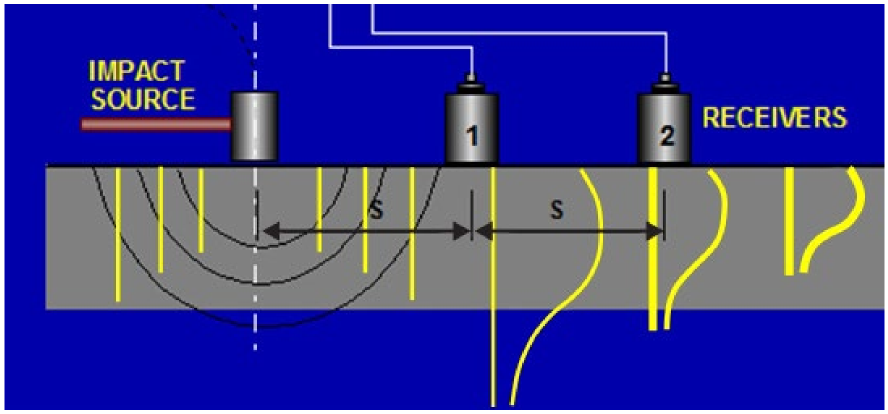


Figure 17.2.15 Schematic Representation of Ultrasonic Surface Waves Test

17.2.2 Destructive Evaluation Methods

Other evaluation methods, the second main classification, covers advanced inspection procedures that are typically invasive and damage localized areas of bridge members, that are generally repaired after testing.

Core samples can be used for many of the following other advanced inspection methods. Usable cores can typically be obtained only if the concrete is relatively sound. If possible, the core diameter should be at least three times larger than the maximum aggregate size. Core holes should be filled with non-shrink concrete grout. Since removing a concrete core may weaken the member, inspectors should exercise caution, and should not remove cores from high-stress areas.

Concrete Cores

Actual concrete strength and quality can be determined only by removing a concrete core (see Figure 17.2.16) and performing such laboratory tests as:

- Compressive strength.
- Cement content.
- Air voids.
- Static modulus of elasticity.
- Dynamic modulus of elasticity.
- Splitting tensile strength.

Figure 17.2.16 Concrete Core Sample

Borescopes

Borescopes, also known as endoscopes and videoscopes, are viewing tubes that can be inserted into holes drilled into a concrete bridge member (see Figure 17.2.17).

Figure 17.2.17 Remote Video Inspection Device

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 not generally considered to be an NDE method because some destruction is necessary for its proper use in concrete structures.

Moisture Content

Moisture content in concrete serves as a major factor in corrosion activity for steel reinforcing. Moisture content can be determined 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 and composition of hardened concrete, which can be useful in determining the existing quality of a representative specimen (see Figure 17.2.18). This advanced inspection method can detect Alkali-Silica Reaction (ASR) products.

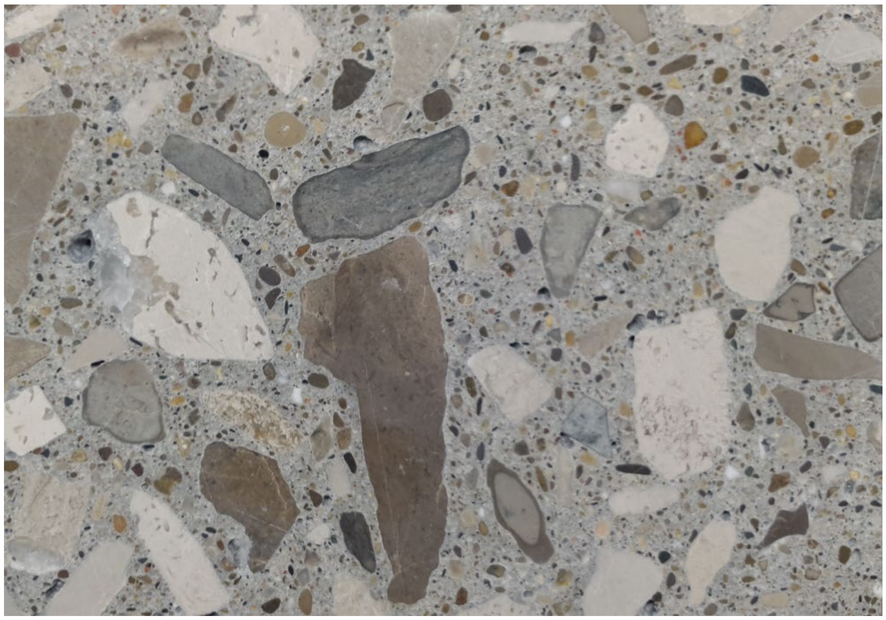


Figure 17.2.18 Petrographic Photo of a Concrete Specimen

Alkali-Silica Reaction (ASR) Evaluation

Another method is a qualitative ASR field test that utilizes colored dyes. This test should be 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.

The results of the ASR field test can be compared to results from a petrographic analysis to verify findings.

Reinforcing Steel Sample

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; therefore, it should be done preferably in low-stress areas and is only considered when such data is essential.

Rapid Chloride Permeability Testing

This method can be used to assess the resistance of concrete to the penetration of chloride ions is the rapid chloride permeability test. It is important in determining the extent to which aggressive substances can attack embedded reinforcement. Low permeable concrete generally is of superior strength and is resistant to water/chloride infiltration. Porous concrete enables water, oxygen, and chlorides to reach rebar, which accelerates corrosion of reinforcing.

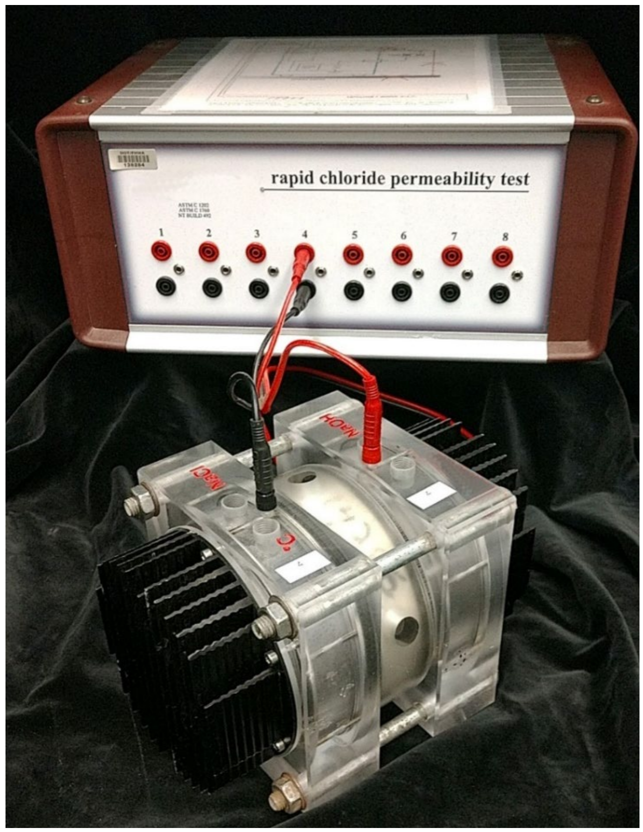


Figure 17.2.19 Rapid Chloride Permeability Testing Device

The test measures the charge passed through a concrete specimen subjected to sixty volts (direct current). 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 typically 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 may be applied for three different 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 into the specimen.

Section 17.3 Steel

17.3.1 Nondestructive Evaluation Methods

Nondestructive evaluation or testing (NDE or NDT) refers to inspection methods that do not compromise the functionality or serviceability of a structural member.

The FHWA InfoTechnology website provides further, more in-depth, information on many NDE methods for steel members discussed in this section. The website can be found at https://infotechnology.fhwa.dot.gov/.

The following methods are detailed on the InfoTechnology website, but not included in this section. Reference the website for further information on:

- Active infrared thermography.
- Dispacement gauge.
- Eddy current array testing.

Acoustic Emission (AE)

Acoustic emission (AE) evaluation detects elastic waves generated by the rapid release of energy from a test object due to mechanisms such as plastic deformation, and fracture. When a structure is under sufficient load levels, it can produce an acoustic wave that ranges between 20 kHz and 1 MHz. The wave that is generated is known as acoustic emissions. Acoustic emission testing uses an ultrasonic sensor to listen for sounds from active flaws and is very sensitive to flaw activity when a structure is highly loaded. This process can detect flaws and imperfections such as the initiation and growth cracks in steel structural members, corrosion products cracking, deformation, rubbing from cracks opening and closing, some weld discontinuities, and the failure of bonds. Most acoustic emissions 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 generally contain a large number of joints, welds, and connections that are potential initiation points for fatigue cracks. Acoustic emission monitoring may be used for early detection of fatigue cracks in critical bridge members and to monitor the relative activity of existing cracks. Advanced signal processing and correlations to parametric measurements are generally used to separate noises generated by dynamic loading, loose connections, rivets, and crack growth (see Figure 17.3.1).

When energy is released (for example high-tensile wire failures), waves propagate in the material. Acoustic sensors distributed along the structure can detect and record the signal. Arrays of sensors and computer processing of the signal then provides valuable information about the event including location, origin, energy, and frequency (see Figure 17.3.1 and Figure 17.3.2).

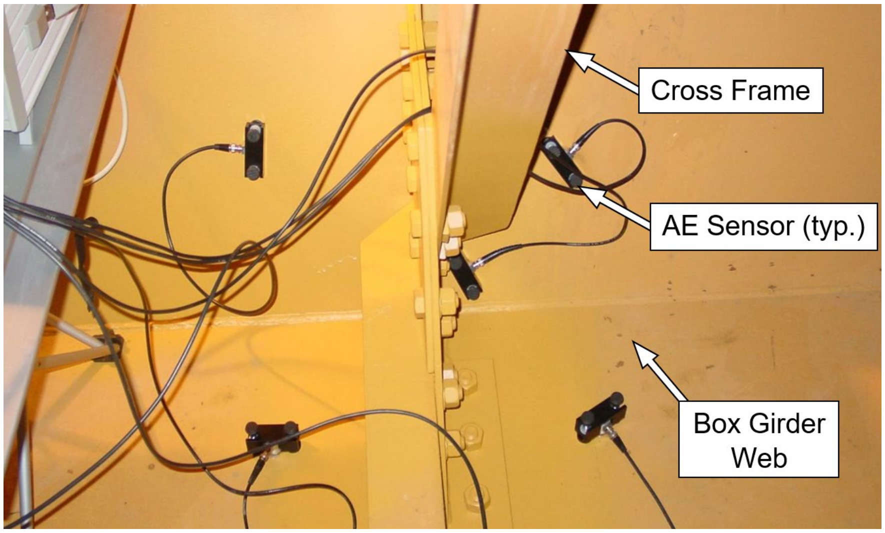


Figure 17.3.1 Acoustic Sensors Used to Determine Crack Propagation

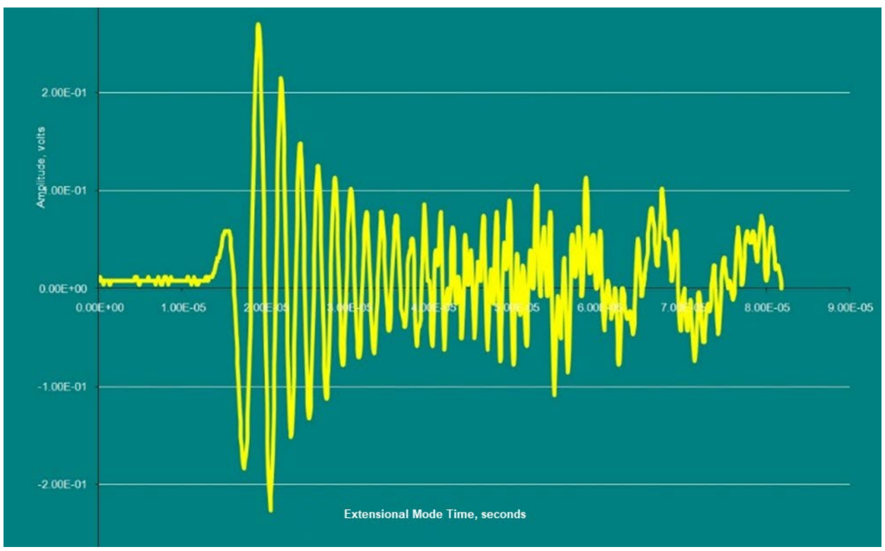


Figure 17.3.2 Acoustic Emission Reading

The main advantage of these systems is the recording and real-time analysis of the waves themselves, providing automatic filtering by the acquisition unit according to preset criteria. The events of interest are stored in the acquisition unit and automatically analyzed.

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 17.3.3). The system can be directly connected to a computer or it can be accessed through wired or wireless modems for data collection. Alarms can be set up to automatically notify engineers of unexpected changes in acoustic emission activity.

Figure 17.3.3 Inspector Using Acoustic Emissions to Determine Crack Propagation

One limitation of acoustic emission testing is that it can be a non-repeatable test. Once a test is completed to a certain load level, it typically cannot be repeated due to the irreversible process of flaw growth unless the structure is loaded to a higher stress, which is not always possible. Also, flaws are not detectable if they are not growing or have flaw surfaces rubbing. Therefore, if the flaw is not increasing in size or has enough flaw surface rubbing, acoustic emission testing may not be able to detect the crack. The bridge may also cause interference with testing. If background noise exists, that noise could be similar to the sound energy released by a flaw. For this reason, the sensors should be properly shielded and filtered against background noise. Lastly, the acoustic emission testing unit is relatively expensive and also necessitates the additional cost of an experienced operator to set up.

Crack Propagation Gage (CPG)

CPGs are generally used to monitor the length of an existing crack or monitor an area that is highly susceptible to cracking. The gage is installed on the surface near the tip of an existing crack and includes a parallel series of conductive wires that are perpendicular to the crack (see Figure 17.3.4). If a crack were to propagate, the individual wires would be severed one by one, thus enabling measurement of growth, without physically seeing the crack.

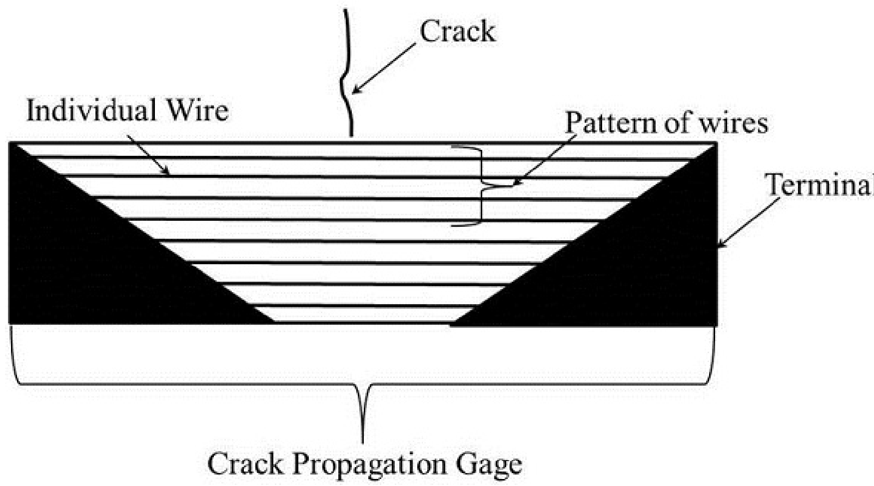


Figure 17.3.4 Schematic Representation of a Crack Propagation Gage

When used to observe a localized area for future cracking, the gage would be placed at a location of high stress, such as in the head of an eyebar. When the measured voltage changes due to the change in electrical resistance of the gage, it may be evident that the steel has cracked and fractured a portion of the gage.

Dye Penetrant Testing (DPT)

A dye penetrant test (DPT), also often called a liquid penetrant test, can be used to determine the extent and size of surface flaws in steel or other metal members (see Figure 17.3.5). The test area should be cleaned to bare metal to remove contaminants, a penetrant is applied to the surface by spray or brush, provided sufficient time for the penetrant to be pulled into the flaws, and then excess penetrant is removed. A white developer is then applied, which acts like a blotter, and draws the dye out of the irregularities and delineates the extent and size of surface flaws (see Figure 17.3.6). Bridge inspectors commonly use this method since it does not necessitate extensive training or expensive equipment.

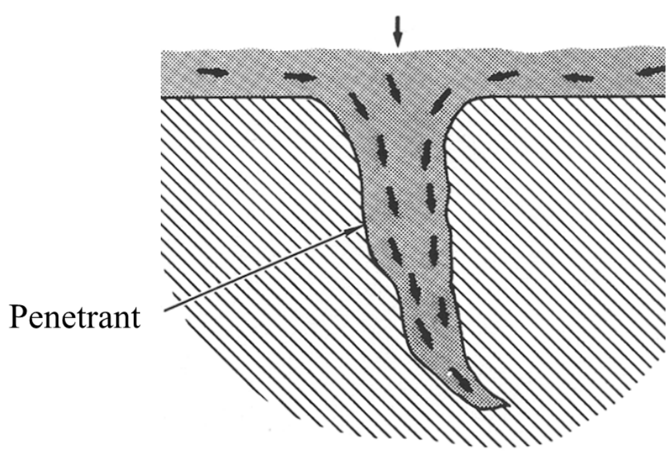


Figure 17.3.5 Penetrant Being Pulled into a Crack

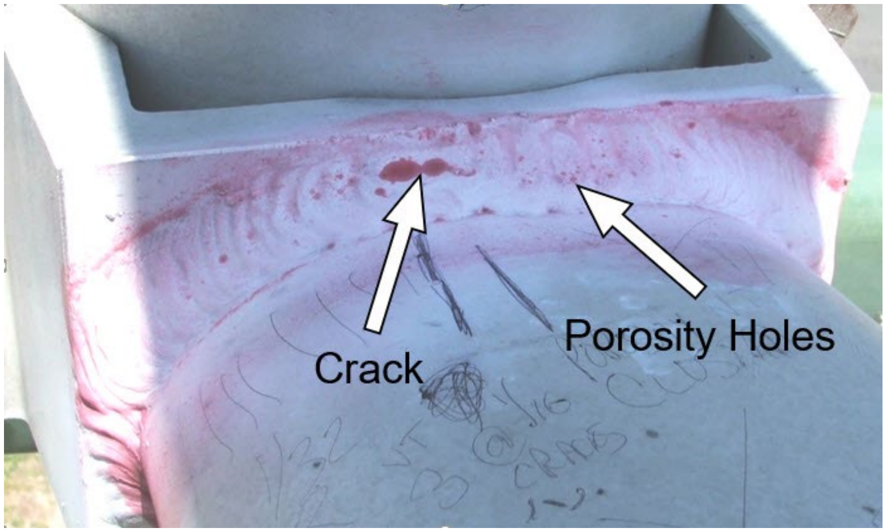


Figure 17.3.6 Detection of a Crack and Porosity Using Dye Penetrant

The primary limitation of DPT is it can only detect defects that are open to the surface and nothing inside the specimen. For a proper inspection, the surface should be clean, which includes paint removal. To investigate the presence of a crack, the inspector should have good visual acuity. In addition, a recommended temperature of over 40 degrees Fahrenheit is necessary to achieve reliable results using dye penetrant testing.

Eddy Current Testing (ECT)

Eddy current testing (ECT) monitors the electrical properties across a coil that has an AC current flowing through it. When the coil is placed near the conductive member, the electrical eddy currents are produced in the material that flow opposite to the direction of current flow from the coils. Flaws in the member will typically disturb the eddy currents, which, in turn, affect the induced current. The affected induced current is monitored through the electrical properties across the coil. Eddy current testing devices can be hand-held devices (see Figure 17.3.7). ET can only be performed on conductive materials and is primarily used to detect cracks. This method can be used on painted or untreated surfaces.

Figure 17.3.7 Hand-Held Eddy Current Testing (ECT) Instruments

Limitations in eddy current testing include the inability to determine the depth of the crack. This NDE method also does not work well with galvanized steel members. Eddy current testing needs operators with the proper training to correctly interpret the test results, which adds to the cost of this inspection method.

Magnetic Particle Testing (MT)

Magnetic particle testing may be useful in detecting cracks and similar weld flaws in ferromagnetic materials such as carbon and low alloy steels used in highway bridges (see Figure 17.3.8 and Figure 17.3.9). It can also detect near-surface deficiencies, such as voids, inclusions, lack of fusion, and cracks if certain MT equipment is used. Its effectiveness, however, diminishes quickly depending on the depth and type of flaw and MT is generally recognized as primarily a surface inspection method like PT.

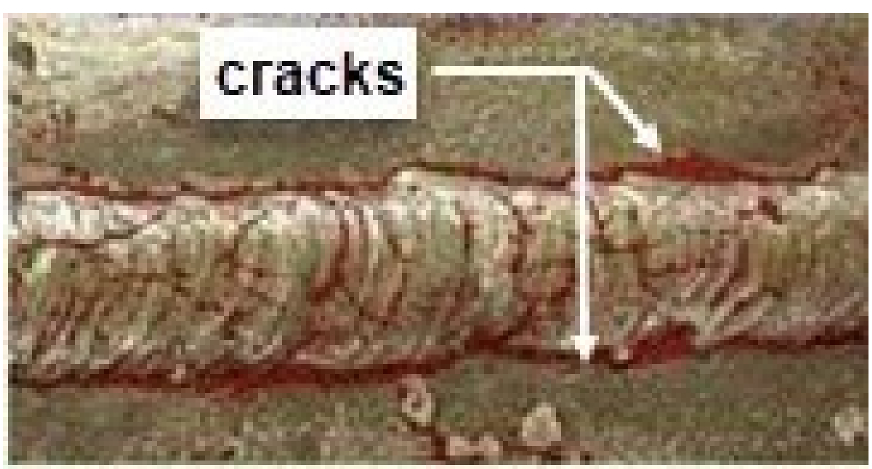


Figure 17.3.8 MT Detected Cracks in a Weld

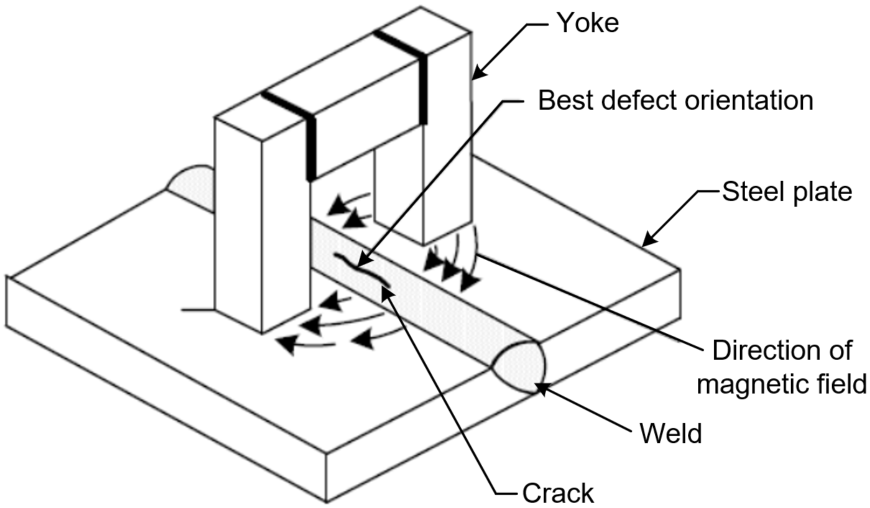


Figure 17.3.9 Magnetic Particle Testing Sketch

The method consists of magnetizing the member, applying iron filings dyed for color contrast, and then interpreting the pattern formed by the filings, which are attracted by the magnetic leakage field (see Figure 17.3.10).

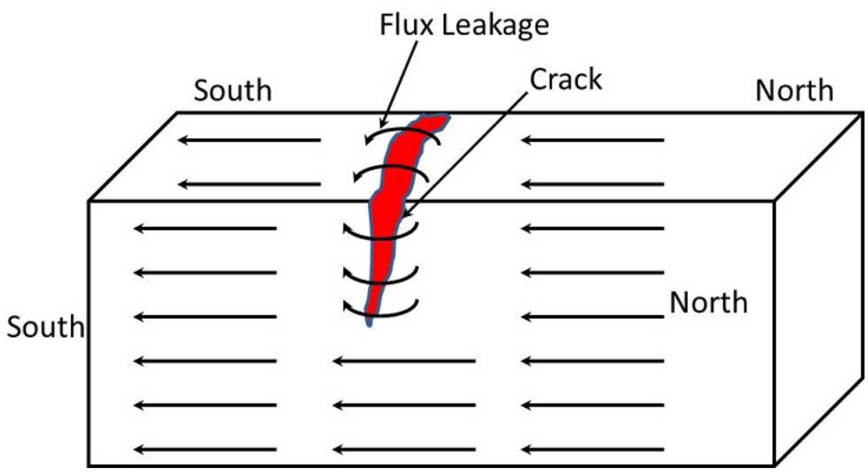


Figure 17.3.10 Schematic of Magnetic Field Disturbance Around a Flaw

Limitations of magnetic particle testing include applicability only for members composed of a ferromagnetic material. Painted surfaces can be inspected but unpainted surfaces help to ensure the maximum sensitivity of the inspection (see Figure 17.3.11).

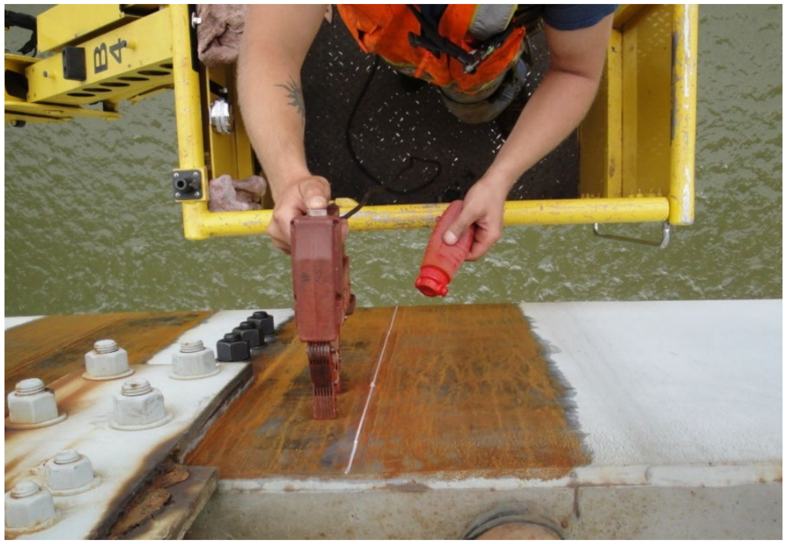


Figure 17.3.11 Magnetic Particle Testing (MT) on Member with Paint Removed

Magnetic Flux Leakage (MFL)

Magnetic flux leakage testing, is a form of nondestructive evaluation that has been used over the past few decades to detect the location and extent of corrosion in post-tensioned and precast prestressed strands in concrete girders, and breakage of wires and strands in post-tensioning tendons and prestressing strands (see Figure 17.3.12). MFL is also commonly used to test the cables of suspension or cable stayed bridges. MFL units can be clamped onto a cable as part of a climbing module or rolled across a surface. This method uses a magnet to magnetize the steel to help detect cracks, corrosion, and pitting in steel. Cracks can be detected by sensors that record changes in the magnetic field caused by flaws in the rope.

Figure 17.3.12 MFL Device for Testing Post-Tensioned Tendons in Deck

Ultrasonic Testing (UT)

Ultrasonic testing is frequently used in steel applications and can be used to detect flaws and cracks in welds and pins. UT units generate pulses of high-frequency sounds (range of 100 kHz to 25 MHz) into the material and produces signals to identify flawed regions (see Figure 17.3.13). It can also be used to measure the thickness of corroded steel members, providing detailed information concerning the 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 involves applying a couplant to the region of interest and then scanning the area with a transducer, which is attached to the UT machine.

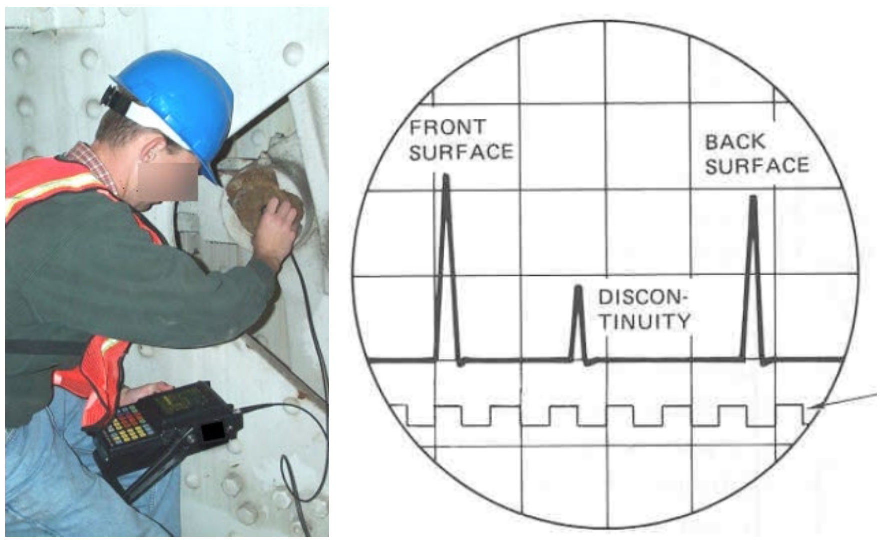


Figure 17.3.13 Ultrasonic Testing of a Pin in a Movable Bridge

Limitations of ultrasonic testing may include the inability in inspect through very rough surfaces or in complicated geometries. In addition, flaws that are parallel to the sound waves may not be reliably detected. Skilled operators are needed to perform ultrasonic testing, adding to the cost of this NDE method.

UT thickness depth meters (often referred to as D-meters) are miniature versions of an ultrasonic testing instrument and are very straightforward to use (see Figure 17.3.14). These instruments can only detect the thickness of the part being tested and cannot be used for the detection of internal flaws.

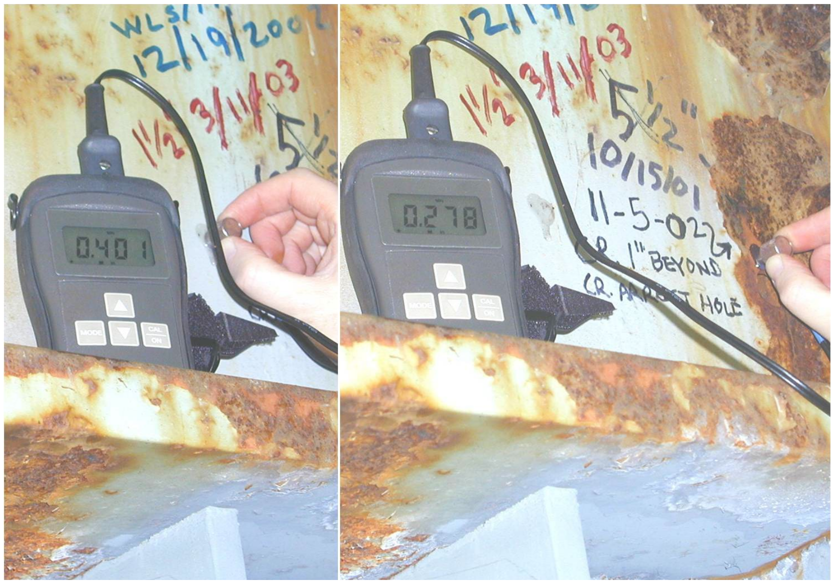


Figure 17.3.14 Ultrasonic Thickness Depth Meter (D-meter)

Phased Array Ultrasonic Testing (PAUT)

Phased array ultrasonic testing (PAUT) units are another form of ultrasonic testing that can be used to test for discontinuities in steel members (see Figure 17.3.15). They utilize an array that consists of a series of individual transducer elements that are separately pulsed, time-delayed, and processed. The software enables the operator to modify the beams pulse time delay or phasing which result in the ability electronically scan, sweep, steer, and focus the beam.

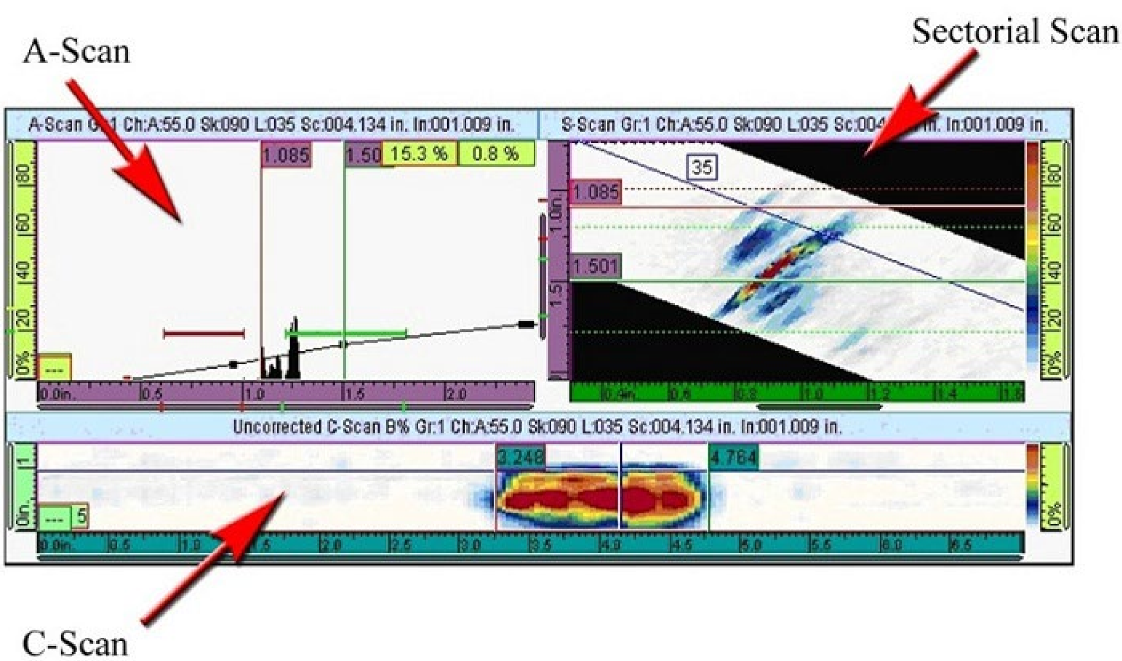


Figure 17.3.15 Phased Array Ultrasonic Testing Results

PAUT is executed using shear wave (angled beam) or longitudinal wave (straight beam) techniques. Shear wave methods are most commonly used when testing welds for flaws. The technology uses a transducer to emit acoustic waves. The waves propagate through the steel to measure the full thickness or may detect any discontinuities in the material.

In contrast to the conventional UT, PAUT uses multiple sensor elements to emanate and detect the acoustic waves. This generally results in improved coverage and more flexibility with complex geometries. Other 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 can also produce high resolution images, which results in less user-interpretation needed by the operator.

Limitations of this method is that the surface should be prepared before testing, and extensive training courses are necessary, due to the difficulty in using this equipment. Lastly, the units are typically expensive and necessitate an additional cost for a qualified operator.

17.3.2 Destructive Evaluation Methods

Other testing methods are available to measure the physical properties of the steel. These methods are typically invasive and damage localized areas of bridge members, that are generally repaired after testing. The following tests can be conducted only by the destructive method of removing a sample and evaluating it in a laboratory.

Material property 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 should be suitable for the planned tests. If a coupon is necessary, the inspector should consult the bridge engineer to determine the most suitable area of removal. For instance, an engineer should not remove a coupon from the web area over a bearing. An engineer should not recommend the removal of a coupon from a high-stress zone such as the bottom flange at midspan. Such tests may be necessary to determine the strength or other properties of a bridge member made from an unknown steel.

Brinell Hardness Test

The Brinell hardness test measures the resistance to penetration of steel. A hardened steel ball is pressed into the test coupon by an applied load. The surface area of the indentation is 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 can determine the amount of energy necessary to fracture a specimen, which provides a measurement of the material's fracture toughness (see Figure 17.3.16). A common impact test for steel coupons is the Charpy V-notch test (see Figure 17.3.17). A notched test coupon is placed into a slot, and a hammer on a pendulum is then released from a known elevated position, swinging down and hitting the coupon.

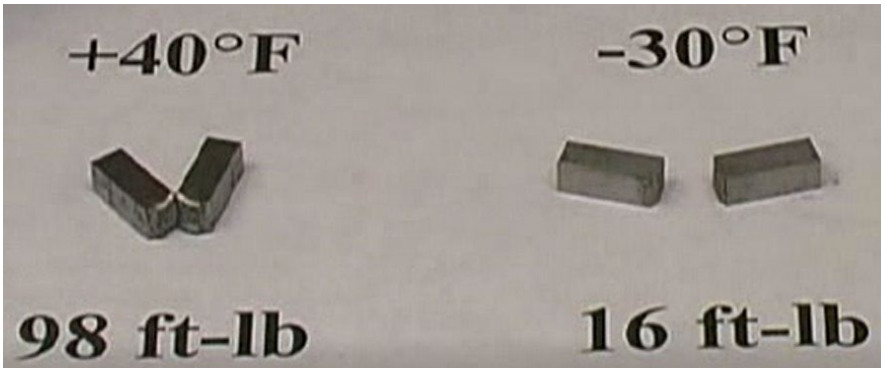


Figure 17.3.16 Fractured Impact Test Specimens for Different Temperatures

Figure 17.3.17 Charpy V-Notch Test

Because the force of the hammer is concentrated at the notch in the coupon, the stress causes fracturing of the specimen. The energy necessary for fracture can be determined based on the mass of the hammer and the difference in the initial height and final height, which is recorded on the dial on the striking hammer. This test can be performed at different coupon temperatures to investigate the susceptibility of steel to brittle failure.

Chemical Analysis

The chemical composition of the steel is an important indication of whether the steel is weldable or there is a demand for special precautions. 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. Coupons can be removed from a bridge and tested in a laboratory using a variety of methods to determine the elements and amount of each element in a steel. Such an equation, based on the relative proportions of various elements in steel, is presented in the ASTM A706 rebar specification:

$$C.E. = C\% + \frac{Mn\%}{6} + \frac{Cu\%}{40} + \frac{Ni\%}{20} + \frac{Cr\%}{10} - \frac{Mo\%}{50} - \frac{V\%}{10}$$

The elements represented in the equation are as follows:

- C – Carbon.
- Mn – Manganese.
- Cu – Copper.
- Ni – Nickel.
- Cr – Chromium.
- Mo – Molybdenum.
- V – Vanadium.

When the C.E. is below 0.55, steel is generally not susceptible to cold cracking, and no special precautions are needed for welding. However, when the C.E. is above 0.55, steel is susceptible to cold cracking, and special precautions are necessary 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 a coupon before it breaks. Once the test is complete, the tensile strength of the steel can be easily determined. Refer to Chapter 6 for details regarding mechanics.

The ends of the test coupon are placed in grips 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 might occur. A brittle fracture will most likely produce a fracture with a smooth surface (see Figure 17.3.18).

Figure 17.3.18 Brittle Failure of a Cast Iron Specimen

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 17.3.19). Plastic deformation typically results in distortion of the member, which should provide a visual warning before the fracture of a member. The reduced cross section is caused by plastic distortion rather than section loss. The fracture produces a rough surface and shear lips that are tilted at approximately 45 degrees.

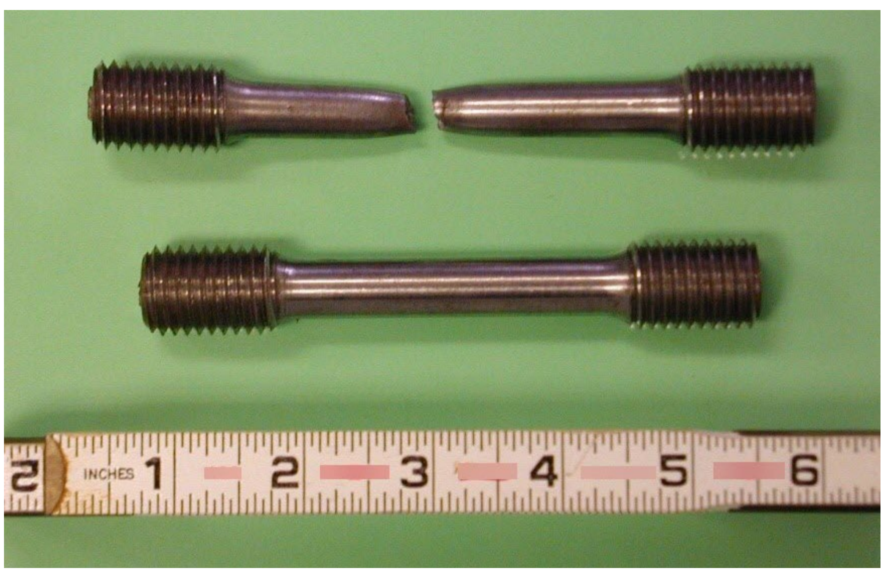


Figure 17.3.19 Ductile Failure of Cold Rolled Steel

Section 17.4 Timber

17.4.1 Nondestructive Evaluation Methods

Nondestructive evaluation or testing (NDE or NDT) refers to advanced inspection methods that do not impair the serviceability or functionality of a structural member.

The FHWA InfoTechnology website provides further, more in-depth, information on several NDE methods for timber members discussed in this section. The website can be found at https://infotechnology.fhwa.dot.gov/.

The following methods are detailed on the InfoTechnology website, but not included in this section. Reference the website for further information on:

- Acoustic tomography.
- Ground penetrating radar.

Stress Wave Velocity

Stress Wave Velocity method, also known as Stress Wave Timing on InfoTechnology, uses sonic waves to produce stress waves in a timber member. The stress waves are then used to find 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 (see Figure 17.4.1). If the member's dimensions are known, the longer travel time of stress waves can indicate the presence of deficiencies.

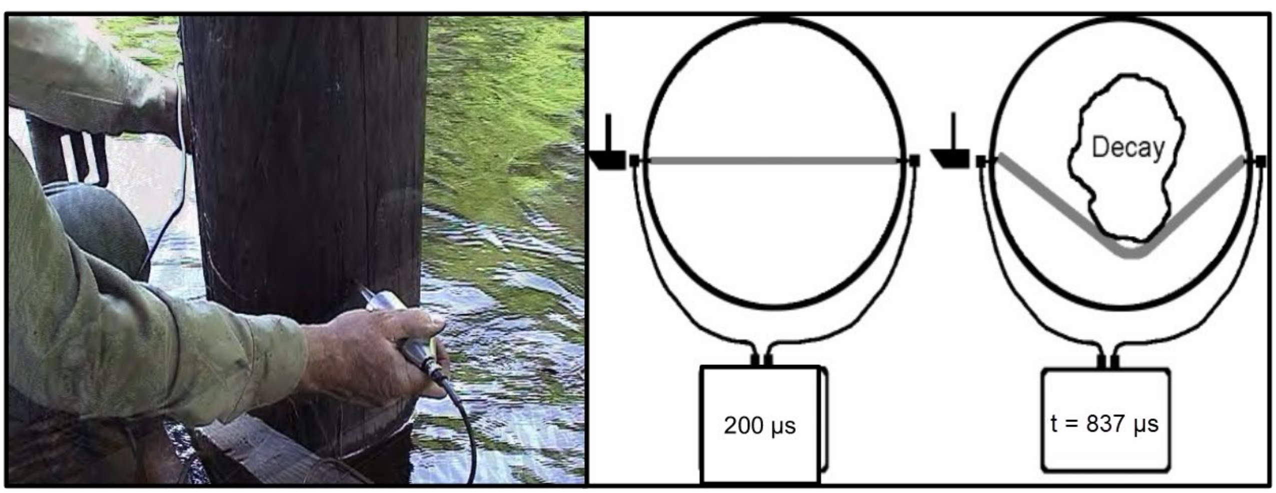


Figure 17.4.1 Stress Wave Procedure and Results for Timber Pier Column

Stress waves are also used to determine the in-place strength of timber members. Sound timber members transmit waves at a higher velocity than decayed wood. The velocity of the stress wave can be calculated by obtaining time of flight readings over a set length. The velocity can be converted into a dynamic modulus of elasticity, which in turn, enables properly trained personnel to estimate the strength 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 typically 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 necessary.

The transmission time is affected by properties such as growth ring orientation, decay, moisture content, and preservative treatment.

Ultrasonic Testing

Ultrasonic testing (UT) consists of high-frequency sound waves introduced by a sending transducer (see Figure 17.4.2). Discontinuities in the specimen interrupt the sound wave and deflect it toward a receiving transducer. The magnitude of the return signal provides 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.

Figure 17.4.2 Ultrasonic Testing Equipment

In timber bridge members, ultrasonic testing can be used to determine the in-place strength of timber bridge members, both above and below water. The load-carrying capacity of the member is correlated to the member's wave velocity normal to the grain and its in-place unit weight.

Vibration

The vibration technique is based on the philosophy that sound timber members vibrate at a certain frequency (see Figure 17.4.3). During the test, if a timber member vibrates at a lower frequency than the established theoretical frequency, the member may be deteriorated. Vibratory testing methods in timber members are generally used to determine the member's modulus of elasticity. From this, other properties of the timber member can be established. This method is referred to as Transverse Vibration of Structural Systems on the InfoTechnology website.

Figure 17.4.3 Vibration Testing on Timber Deck

17.4.2 Destructive Evaluation Methods

Other testing methods, the second main classification, covers advanced inspection procedures that are typically invasive and damage localized areas of bridge members, that are generally repaired after testing.

Boring or Drilling

Drilling and coring are the most common methods for detecting internal deterioration in bridges, but boring is seen as the most dependable and widely used method for detecting internal decay in timber. Drilling and coring are generally used to detect the presence of voids and to determine the thickness of the residual shell when voids are present. Boring enables a direct examination of a sample from a questionable member. A timber boring tool is used to extract wood cores for examination (see Figure 17.4.4).

Coring with timber boring tools produces a solid wood core that can be carefully examined for evidence of decay pockets and other voids. The use of increment cores for assessing the presence and damage due to bacterial and fungal decay necessitates special care. Cleaning of the timber boring tools is necessary after each core extraction to eliminate the transfer of organisms. There are several cleaning agents available to clean the timber boring tools or drill bits.

Core samples are more commonly used to measure the depth of preservative penetration and retention, and to investigate the presence of internal decay pockets. For samples that do not show visible signs of decay, culturing may provide 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 typically practical for assessing the hazard of external decay.

Figure 17.4.4 Timber Boring Tools

Drilling may also be performed using a rechargeable drill or a brace and bit. An abrupt decrease in drilling resistance may indicate decay or 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. Samples are generally not attainable, but 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.

A resistance drill is a drilling and logging tool with a smaller diameter bit. 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 (see Figure 17.4.5). Using a pen, paper, and rotary drum arrangement, it generates a graphic record of the wood resistance during testing. Sound wood typically produces a series of near-vertical markings on the record, however, when decayed wood is encountered, there is less resistance and the markings have 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.

Screw withdrawal testing is similar to drilling, in that the method determines timber's physical and mechanical properties by measuring the withdrawal resistance, or maximum load required to remove the screw. This type of test can determine density, modulus of elasticity, and other timber properties, but only for a localized area of the member.

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 should be treated with a preservative and plugged after testing. Failure to properly treat the wood may result in accelerated decay or deterioration of a timber structure.

Figure 17.4.5 Inspector Using Decay Detection Device

Moisture Meter

Moisture meters can be used to determine the moisture content of a timber member (see Figure 17.4.6). 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 up to a depth of approximately 2 1/2 inches.

The high moisture content of decaying wood typically causes steeper than normal moisture gradients, that enable the meter to determine the extent of the decay. Moisture contents exceeding 20% generally indicate the condition of the wood is conducive to decay.

Figure 17.4.6 Moisture Meter

Section 17.5 Fiber Reinforced Polymer

17.5.1 Nondestructive Evaluation Methods

Acoustic Emission Testing

Acoustic emission testing can be very useful for detecting areas containing deficiencies. This method is based on recording and analyzing stress waves being produced due to deformation, crack initiation, crack growth, breaks in reinforcing fibers, and delaminated areas (see Figure 17.5.1). Given the high level of experience and equipment needed to perform acoustic testing, this type of NDE is typically performed by specialty technicians.

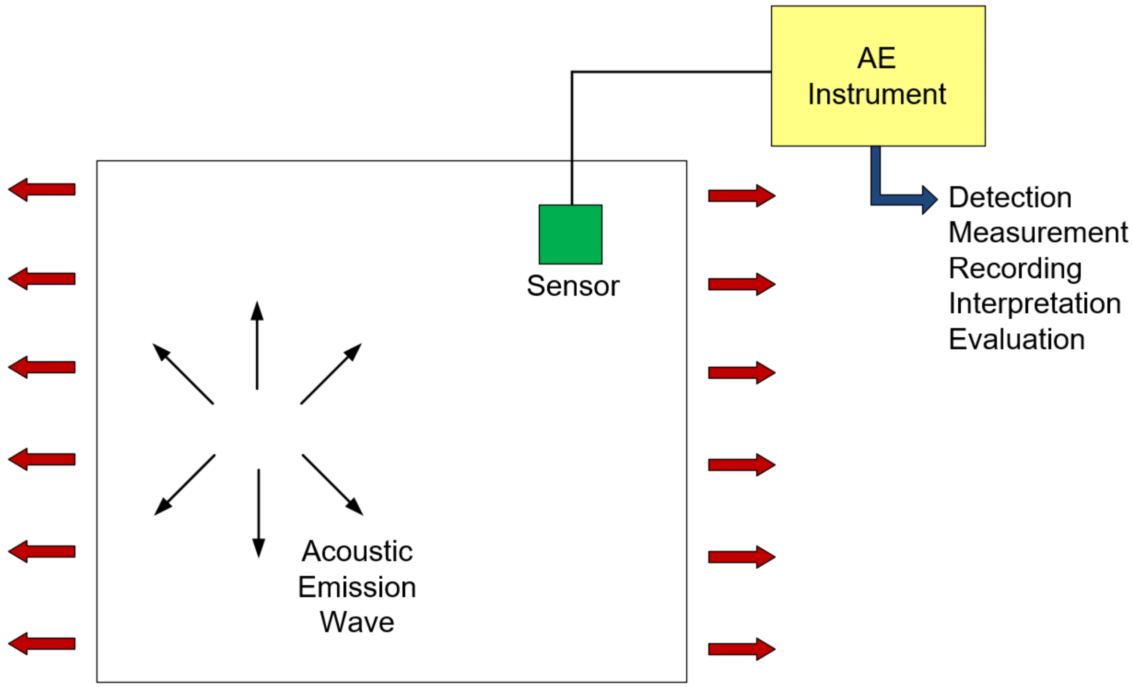


Figure 17.5.1 Acoustic Emission Technique

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 the 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 technician certification from the American Society of Nondestructive Testing (ASNT) is necessary to perform the test. Bridge inspectors already familiar and certified in ultrasonic testing for other materials can easily adapt to the 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. However, this method generally necessitates expensive portable equipment.

Tap Testing

Tap testing can be a relatively quick, inexpensive, and efficient method of detecting areas of debonding or delaminations in FRP. Tap testing can generally be performed by a bridge inspector or engineer with very little training.

This method of physical examination is performed using a small hammer tap (see Figure 17.5.2) or large coin to listen for variations in frequency. Sound areas should produce clear “pings”. Delaminated areas should result in low “thud” sounds. For noisy environments, electronic devices may be used for specific member types.

Figure 17.5.2 Electronic Tap Testing Equipment

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 may identify areas of delamination, impact, moisture, and voids (see Figure 17.5.3). Thermal testing necessitates 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 a practical advanced inspection methods for FRP.

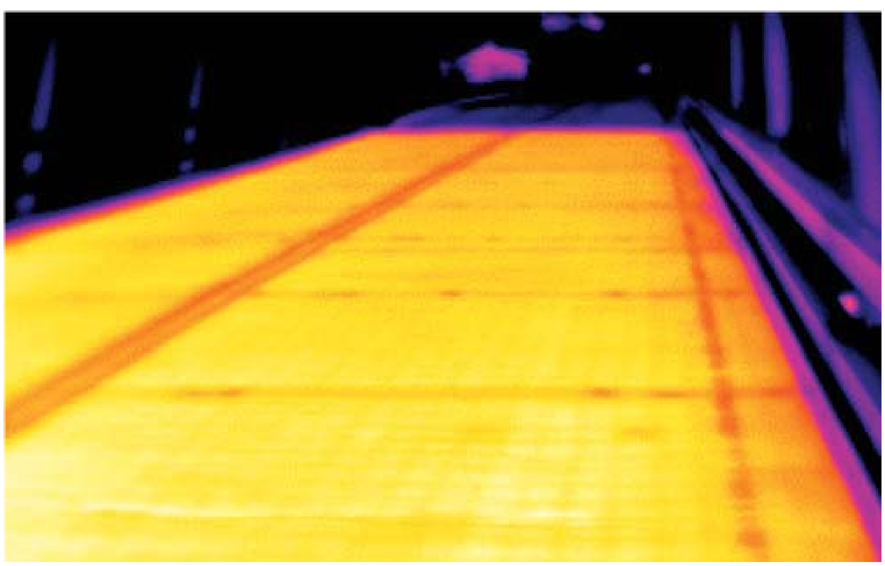


Figure 17.5.3 Thermographic Image of an FRP Bridge Deck

Section 17.6 Advanced Bridge Evaluation

17.6.1 Introduction

In addition to nondestructive and destructive testing methods, there are several types of inspection equipment and bridge evaluation methods that are also considered advanced. The items presented in this section are generally outside the scope of a typical inspection, but can be very helpful in obtaining specific information.

Advanced inspection equipment and tools are likely used in circumstances where access is limited in some way. The equipment generally utilizes innovative technology to gather data that would be difficult to obtain without specialized tools. These tools may also alleviate safety concerns or simply make a task more efficient.

Sensors and other devices capture and report highly accurate and objective data, that can be used to make “data-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 enable 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. Although devices are becoming more readily available and common place, technologies still usually demand some customized hardware and software. For more elaborate systems, an experienced technician is typically needed to install sensing devices or perform the testing, and an engineering professional that has expertise in interpreting the results. Generally, the typical bridge inspector should have a basic understanding of the available advanced bridge evaluation technologies. This basic knowledge enables them to participate in the selection and use of the appropriate technologies to further determine the bridge's condition.

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 17.6.1).

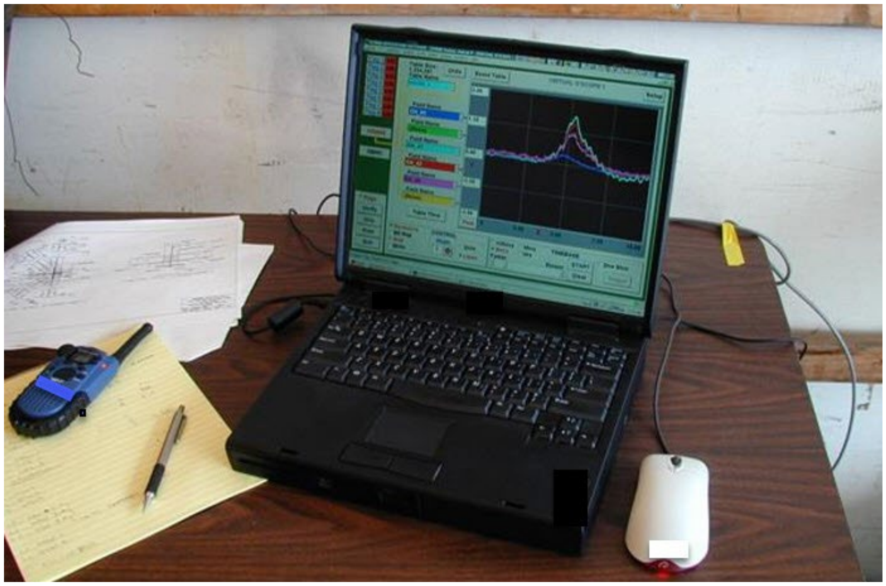


Figure 17.6.1 Viewing Real-Time Data

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's strain, 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.

17.6.2 Advanced Equipment and Inspection Tools

In addition to the standard and special equipment described in Chapter 2, there are options for innovative equipment and advanced technologies available to aid in bridge inspection. The information in this section represents some of the advances in inspection tools and data collection.

RABIT™

The Robotics Assisted Bridge Inspection Tool (RABIT™) was developed to automate nondestructive evaluation data collection (see Figure 17.6.2). The goal is to provide a safer, more efficient, and cost-effective option to evaluate the condition of concrete bridge decks. RABIT™ is remotely controlled and collects an assortment of data in one pass.

Images of the deck surface are recorded by high-resolution industrial cameras mounted on the device. Electrical Resistivity (ER) and Ground Penetrating Radar (GPR) technologies are incorporated to measure and map the corrosion of reinforcement within the concrete. The concrete strength and delamination can be determined with a combination of Impact Echo (IE), and Ultrasonic Surface Waves (USW) testing. RABIT™ is also equipped with GPS to record and mark accurate locations of the testing and deficiencies.

Figure 17.6.2 Robotics Assisted Bridge Inspection Tool (RABIT™)

Scour Monitoring

The following equipment can be used to identify, evaluate, and monitor scour related deficiencies. Refer to Chapter 16 for more detail on the use of sonar for Underwater Inspection.

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 can be utilized for portable scour monitoring at a bridge site. Side scan sonar transmits a specially shaped acoustic beam to both sides of the support craft, which provides for accurate imaging of large areas of the channel bottom. A disadvantage to this method is that most side scan systems do not provide depth information.

Multi-beam Sonar

Multi-beam sonar systems may provide similar fan-shaped coverage to side scan sonar systems, but measures depths instead of images, creating colorful maps based on the varying depths. Multi-beam sonar is typically attached to the surface vessel (see Figure 17.6.3) rather than being towed.

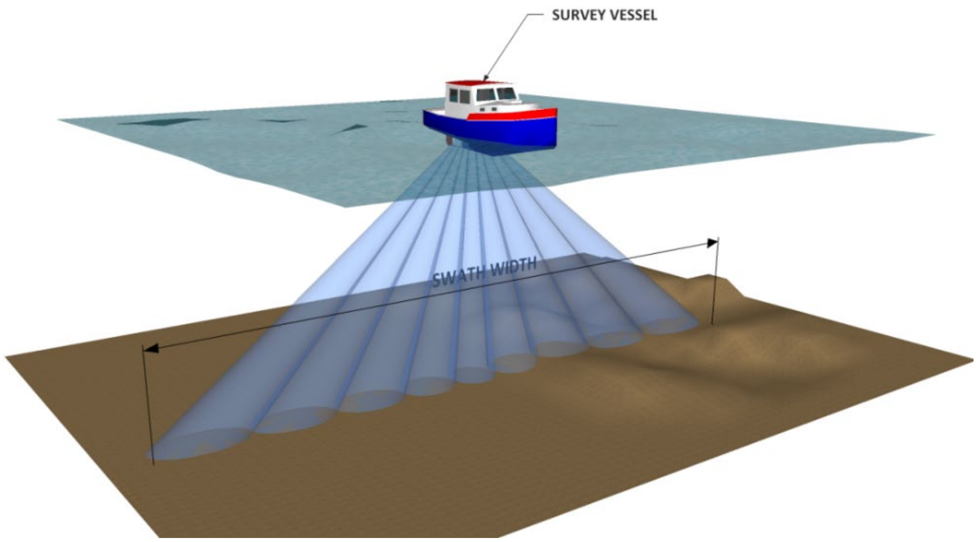


Figure 17.6.3 Multi-beam Sonar

Scanning Sonar

Scanning sonar differs in that it can be completed from a fixed, stationary position. The transducer assembly (or "head") rotates in an arc pattern. During rotation, it emits the beams in every direction.

Portable Depth Finders with Transducers

Portable depth finders with transducers can be used to monitor real-time scour at substructure units during major flood events. Extreme caution should be taken when taking measurements during storm events. Safety is the highest importance. For those bridges that need monitoring, the scour depths and locations of concern are indicated on the Scour Plan of Action.

Web-based Scour Monitoring

Scour monitoring applications enable bridge owners to remotely monitor scour critical bridges and may predict, identify, prepare for, and record potentially destructive flooding events. 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 bridge scour monitoring device data. Transportation officials can efficiently dispatch emergency personnel, bridge safety inspectors, and maintenance workers before, during, and after a flooding event affects a state's bridge inventory.

Unmanned Aerial System

Unmanned Aerial Systems (UAS), commonly referred to as drones, are an advanced technology that can be useful to aid an inspection, but they are not intended to replace the inspectors or access demands (see Figure 17.6.4). Drones used in bridge inspection typically carry payloads such as high-definition cameras or thermal imaging cameras. These images should be monitored during flight and captured as necessary by the bridge inspector and can be saved digitally.

Figure 17.6.4 Drone Used for Bridge Inspection

A certified team leader should be at the bridge site during the entire flight to verify the findings of a UAS. In addition, there are FAA (Federal Aviation Administration) requirements for pilots and flight missions that must be satisfied before performing an inspection with a drone (FAA-S-ACS-10A).

Advantages of utilizing a UAS can include improved safety and fewer traffic impacts. A drone may be effective to identify areas that may or may not need further investigation using regular bridge inspection access equipment by the inspection team. Drones can also provide surveys or collect imagery data that may be useful in emergencies or bridge modeling and provide an aerial view of the site.

High Speed Clearance Measurement System

The high speed underclearance measurement system is designed to mount to a variety of vehicles (see Figure 17.6.5) and serves to measure the underclearance of a bridge at normal highway speeds. The bridge beam clearance can offer precise clearance dimensions and can generally be read to the nearest tenth of a foot. Along with the underclearance data, the GPS data is gathered and recorded to properly identify the overpass or other overhead obstructions. The GPS information can be used in combination with a mapping program to obtain the structure location for future reference.

Figure 17.6.5 High Speed Underclearance Measurement System

Inspection Robots

Although a technology increasing in development, robots similar to drones may prove to be an important addition to the inspector's access equipment. Though 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 an inspector.

Laser Scanning

Laser scanning technology can create accurate and detailed 3D models of a bridge site or bridge member quickly (see Figure 17.6.6).

Figure 17.6.6 Laser Scan of Deteriorated Bridge Member

These digital models after post processing are typically combined and compared with CAD design models to identify deficiencies of structures. This method can replace tedious field measurements for rehabilitation projects. The use of technologies such as laser scanning may necessitate an experienced operator for data collection as well as post processing in addition to the bridge inspector team leader.

17.6.3 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 change. These sensors include foil type, vibrating wire, fiber optic, and a sensor that measures both current and peak strains in one device (see Figure 17.6.7).

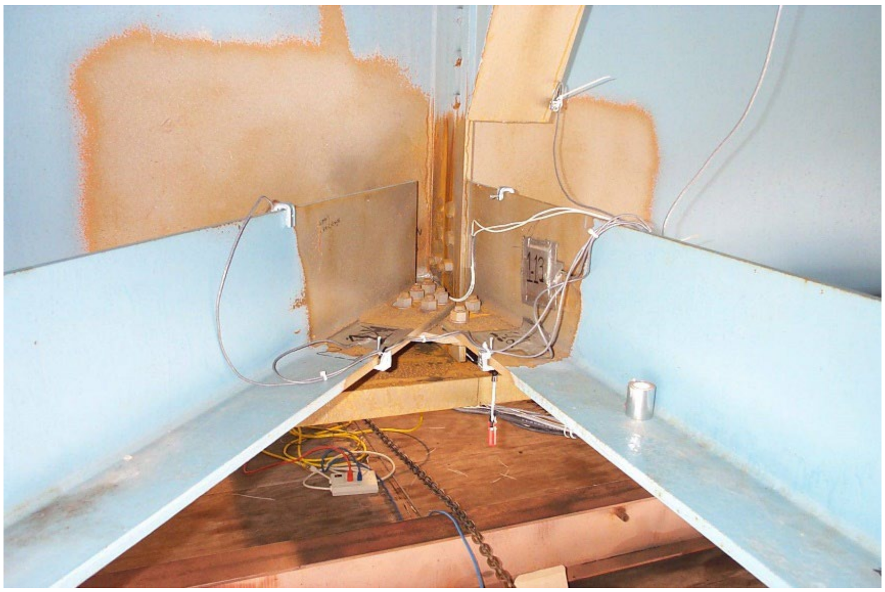


Figure 17.6.7 Strain Gage Used on the Hoan Bridge, Milwaukee, Wisconsin

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 or fully automatic monitoring mode. They enable the acquiring of data at user-defined intervals. The data can be sent wirelessly to a central location for viewing over the internet.

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 magnitude of live load stress within a member.
- The 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 structural 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 is to ascertain the condition of a member or series of members and use that information to infer the safe load-carrying capacity of a structure. Sensors are generally used to aid decision making, by providing additional data to be compared with data from the 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 a 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 provides for secure data capture, data graphing, viewing over the internet, and alerts (by e-mail, cell phone, etc.) 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, laser vibrometers, and even global positioning satellite (or GPS) systems to monitor the 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 the 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 (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, such as in a structural health monitoring system. Specialized personnel are necessary 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 response data from known loads. These bridge evaluations have provided useful information and, in some instances, have revealed bridges that demand closure or restrictions and those that could be safely upgraded (load restrictions removed).

The use of this method involves a combination of strain sensors, on-site data capture, and response modeling. A known load (weighed dump truck) may be driven across a short-span bridge with no other traffic (see Figure 17.6.8). GPS technology is typically used to precisely spot the truck's position, and strain sensors capture member displacements/strains simultaneously. Data capture typically occurs in one day or less. The data can then be used to determine a more accurate distribution factor which in turn can "build" a rudimentary structural model to estimate an actual load-carrying capacity.

This technology gains an advantage over current load capacity protocols as it can consider composite action of the members and contributions to load-carrying capacity from other structural items (sidewalks and parapets) that are typically ignored with traditional analysis methods. Dynamic load testing has been used for many years and has proven its ability to provide an accurate estimation of the load-carrying capacity.

Figure 17.6.8 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 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 both types of response data, the results are generally used to determine structural parameters representing the structural integrity of a bridge.

Initially, system identification may be used to create a structural model, that accurately represents the in-service condition of a structure (see Figure 17.6.9). 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 typically 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 determine the structural integrity of the entire bridge structure.

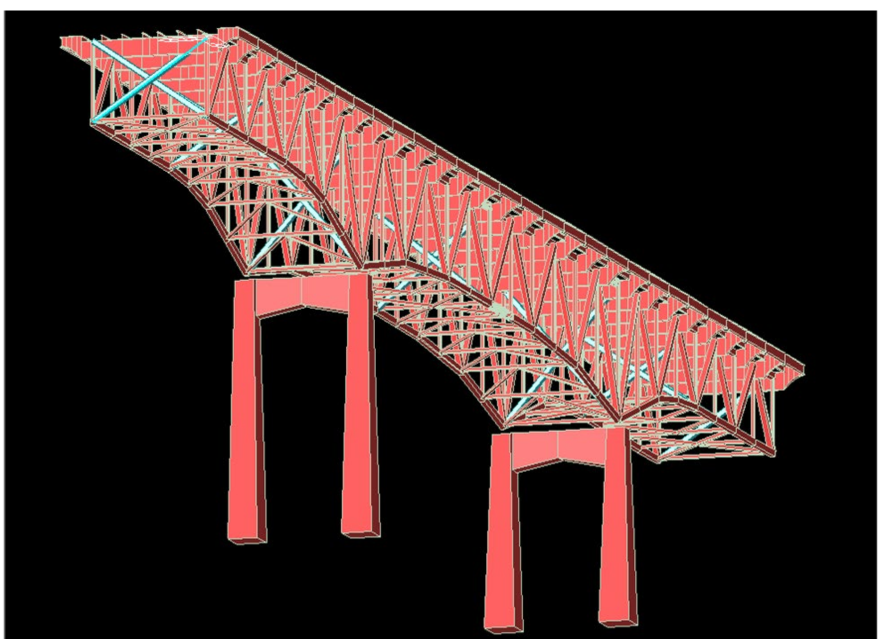


Figure 17.6.9 Structural Model

PART IV - INSPECTION AND DOCUMENTATION

CHAPTER 18 TABLE OF CONTENTS

CHAPTER 18 LIST OF FIGURES

Chapter 18 Preparing the Inspection Report and Other Documentation

Section 18.1 Condition Coding

18.1.1 Introduction

Some information discussed in this chapter is based on generally accepted engineering knowledge and practices, and the application of engineering principles by recognized bridge engineering experts.

As previously discussed in Chapter 3, the overall condition of a bridge component can generally be broken down into one of three categories: Good, Fair, and Poor.

A Good rating would indicate that there are some minor or inherent defects only, if any. These are defects typically occurring during or shortly after construction. Examples for a concrete material include pop-outs, honeycombing, and limited temperature and shrinkage cracking that are not related to ongoing deterioration.

A Fair rating may indicate widespread minor deterioration and/or some moderate defects, but the structural capacity is generally not affected. Examples may include minor section loss, spalling, widespread transverse deck cracking with efflorescence or surface rusting of steel that is typically not measurable.

A rating of Poor suggests that the structural capacity of the member is affected or jeopardized. Examples may include advanced section loss, spalling, cracking, or undermining that may affect the structure's strength or performance.

18.1.2 Condition Descriptions

The general component condition rating systems for the evaluation of bridge components can be found in FHWA's *Specification for the National Bridge Inventory* (*SNBI*). The component condition rating systems presented are general in nature and can be applied to the applicable bridge components and material types.

Component condition ratings in *SNBI* Subsection 7.1 range from 0 to 9, shown below (refer to Chapter 3).

- Ratings 7-9 – Good condition.
- Ratings 5-6 – Fair condition.
- Ratings 0-4 – Poor condition.

Component condition ratings are typically determined by applying condition descriptions, which cover a broad array of bridge components and material types. The inspection team is responsible for being familiar with material type terminology and associated defects to accurately assign component condition ratings consistent with condition descriptions.

Condition ratings indicate the existing field conditions of the bridge components. A condition rating code should consider the type, location, and severity of the defects; the extent to which they exist throughout the item being evaluated; and the degree to which the defects affect strength and/or performance of the bridge or component. The term defect indicates a problem with the component that may be caused by deterioration, damage, or an inherent defect (*SNBI* Subsection 7.1).

The following illustrates several common deterioration and defect terms found in condition descriptions and their associated material types:

- Section loss - usually applies to steel members or reinforcing steel.
- Fatigue cracking - applies to steel members.
- Cracking/spalling - usually applies to concrete.
- Shear cracking usually applies to concrete but may apply to timber as well.
- Checks/splits - applies to timber members.
- Scour - applies to the substructure when support or stability is affected.

Understanding the links between material type and observed defects allows the inspection team to assign component condition ratings 0 through 9 more accurately based on condition descriptions found in the *SNBI* code descriptions. Further information and details on defects and severity guidance can be found in Appendix C of the *SNBI*.

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 effect on structural capacity is generally dependent upon several factors including the type and extent of the deterioration, as well as the location on or along the member. An inspector should discuss the observed condition with an engineer to make this determination.

When these localized conditions are determined to be such that prompt action is necessary 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 (23 CFR 650.313(q)). The NBI component condition rating should be reviewed and appropriately adjusted when the critical issue is identified, and again once the critical finding has been addressed. This adjustment should depend on how the critical finding was addressed and how that action relates to the original rating rationale.

Structural capacity refers to the designed strength of the member, which is different than load-carrying capacity. Load-carrying capacity is 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 adequate structural capacity, yet be load posted because it is unable to carry the current legal loads due to a low design value.

A bridge’s load-carrying capacity should not 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.

Section 18.2 Inspection Reports

18.2.1 Introduction

According to the *NBIS*, the inspection report is a document that summarizes the inspection findings, recommendations, and identifies the team leader responsible for the inspection and report (CFR 650.305). Reference the AASHTO *Manual for Bridge Evaluation (MBE)* for further guidance on the information to be included in the inspection report.

A well-prepared report should 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 should be reported in a factual, objective manner, avoiding speculation. For example, terms such as “hazardous” or “dangerous” are subjective and should not be used in the inspection report or inspection documentation.

In preparing an inspection report, inspectors should 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 serve an important role in future litigation. The language used in reports should be clear and concise and, in the interest of uniformity, should use common terms and descriptions. The information contained in an inspection report is based on field investigations, and may be supplemented by reference material in “as-built” or “field-checked” plans. The source of all references contained in an inspection report should be clearly stated.

The name or identification of the inspection team leader(s) must be identified per CFR 650.305 and the inspection date(s) should be noted in each inspection report. Some state agencies want inspection reports to be signed, dated, and sealed by a professional engineer before accepting them. Other state agencies want inspection reports to be signed and dated by the inspection team leader.

18.2.2 Report Documentation

The inspection report contains documentation of all inspection findings, and includes but is not limited to the NBI data summary sheet including component and element (if applicable) condition data; narrative of all findings including baseline conditions of new elements or components; photos and sketches identifying the bridge, it’s orientation, and any problems found; channel and scour findings; and any recommended maintenance work with a priority assigned for each action. Refer to Chapter 3 and the *MBE Sections 2 and 4.3.4* for more inclusive lists.

Many bridge owners have different inspection report needs which vary significantly in format or necessary inspection forms, but all the information listed above should be in each report for quick reference by the inspectors, program managers, and other personnel as necessary. Some historical bridge information is more suitable for the bridge file, such as scour evaluations; load rating calculations; as-built design plans; construction or inspection history or correspondence; specialized inspection procedures; and recommended equipment. Refer to Chapter 3 and the *MBE Sections 2 and 4.3.4* for more inclusive lists.

Bridge inspection reports should be clear and repeatable. Therefore, the report should be organized and include the following when applicable:

- Member Identification.
- Structure Site Location.
- Bridge Member Orientation.
- Member Dimensions.
- Inspection Notes and Sketches.
- Defect Identification and Documentation.
- Summary of Findings.

Member Identification

Unique identifiers should be used to identify the members by the type of material, construction method, or the function that each member performs, in a logical sequence, to track and locate defects. Inspectors should verify that member descriptions or abbreviations are consistent with bridge owner nomenclature.

Structure Site Location

Structure site location is typically 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:

- I-79, Milepost 155.28 NB.
- SR0019 Segment 0501.
- 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 18.2.1).
- Floorbeam ends are identified by left/right looking in the direction of inventory or south/north or west/east designations.
- Sides of members can be identified by direction (e.g., south side of Floorbeam 2, northeast elevation of Beam 4, or downstream side of Girder G2).
- Span numbers and bay numbers to identify general areas on the bridge (see Figure 18.2.1).
- Individual beams or stringers left to right, looking in the direction of inventory (see Figure 18.2.2).
- Upstream or downstream designations can be assigned to structures over waterways (e.g., upstream truss, downstream girder, or upstream arch) (see Figure 18.2.3).
- For truss elements, identify the member with joint designations and specify if it is an upstream/downstream or north/south truss (e.g., U8M9 U.S.) (see Figure 18.2.4). Number floorbeams in accordance with the panel point numbers (e.g., FB0, FB1).

If the orientation used during the inspection differs in any way with that used in existing documents, then inspectors should clearly state these differences in the inspection notes.

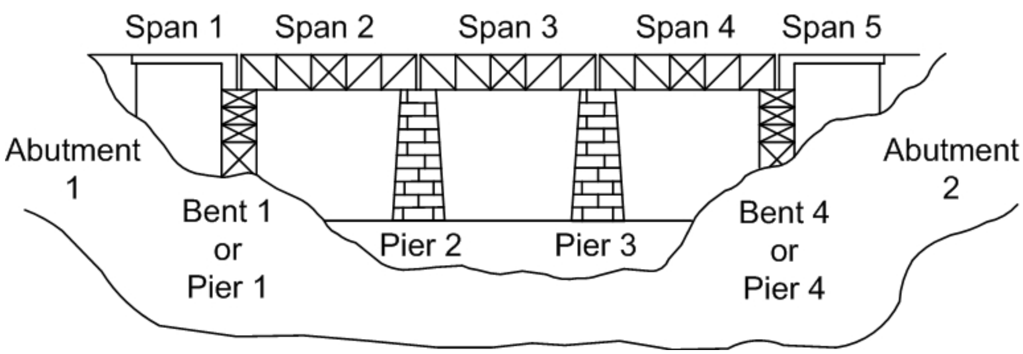


Figure 18.2.1 Sample Span Numbering Scheme

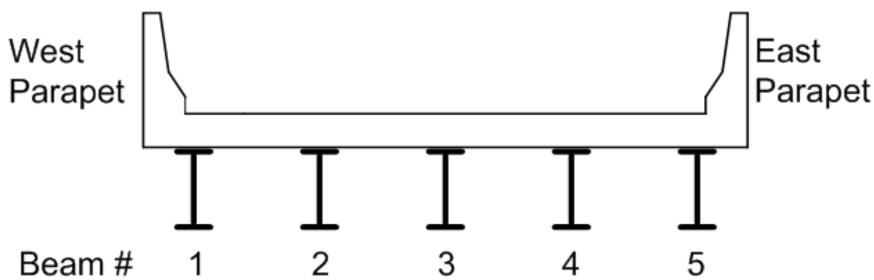


Figure 18.2.2 Sample Typical Section Numbering Scheme

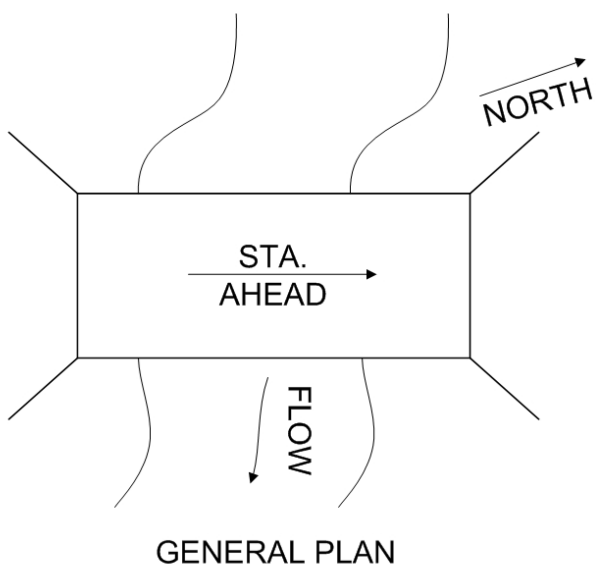


Figure 18.2.3 Sample Structure Orientation Sketch

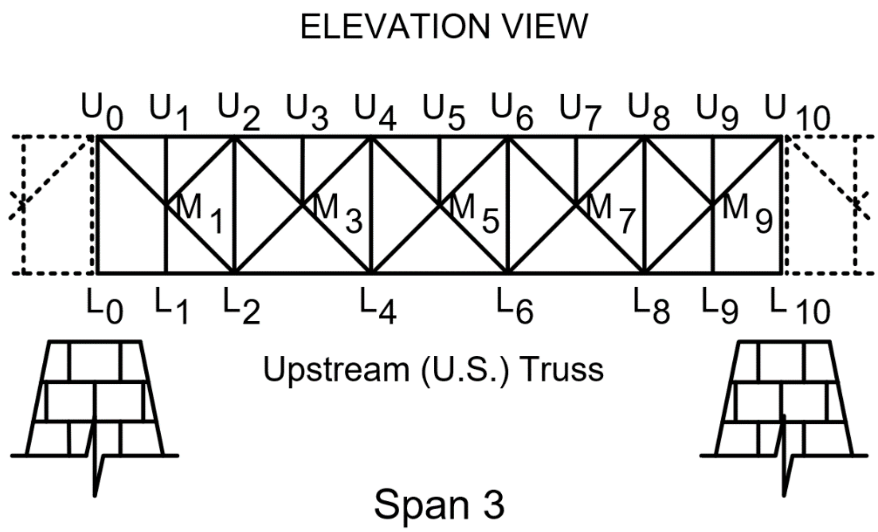


Figure 18.2.4 Sample Truss Numbering Scheme

Member Dimensions

Sufficient dimensions should be documented to establish the size or cross section and other pertinent dimensions of members.

For the deck, these dimensions should include the length, width, and thickness at a minimum.

The length, depth, width, along with flange and web thicknesses, are gathered for superstructure members. Typical members to be measured include beams, girders, floorbeams, stringers, and truss members (see Figure 18.2.5).

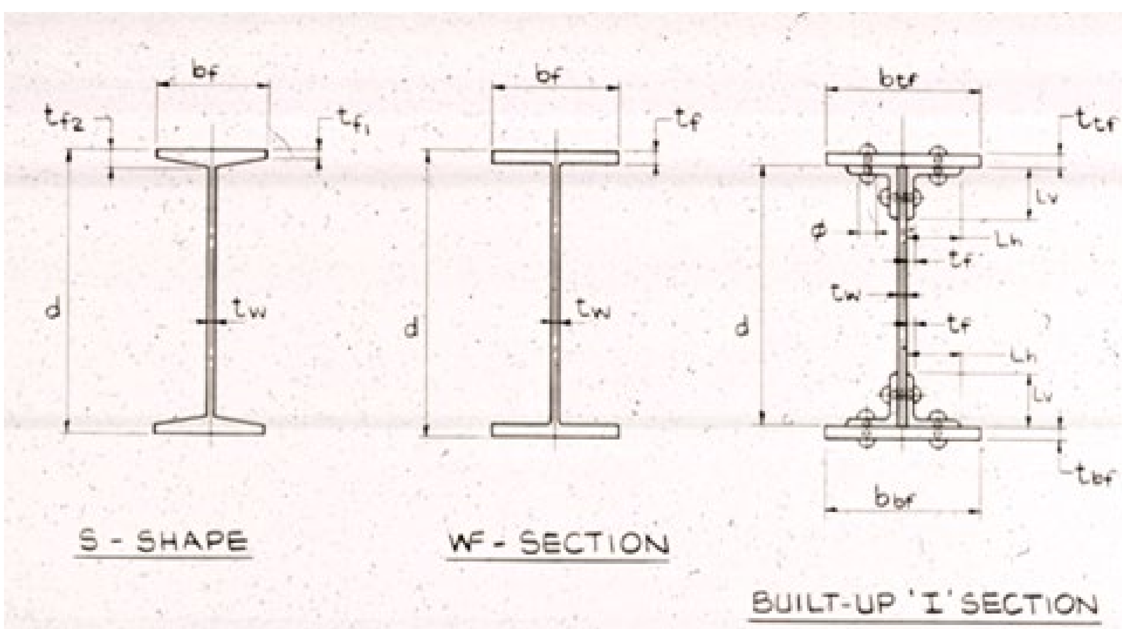


Figure 18.2.5 Steel Superstructure Dimensions

Substructure elements include abutments, columns, piles, and caps. The width and depth (for rectangular shapes), diameter (for round columns), length, spacing, and pile batter and spacing (for pile bents) are relevant dimensions.

Exact member dimensions are necessary to determine section properties used to calculate a load-rating capacity.

Inspection Notes and Sketches

In most cases, it is possible to insert reproductions of portions of the plans in the inspection notes (see Figure 18.2.6). However, in some instances, sketches should be drawn. The inspection team may be able to pre-draw the sketches in the office and fill them out in the field (see Figure 18.2.7). It should be noted that structural reviews and load ratings may utilize inspection report sketches so the inspection team should create notes and sketches that should be clear, accurate, and easy to understand by others.

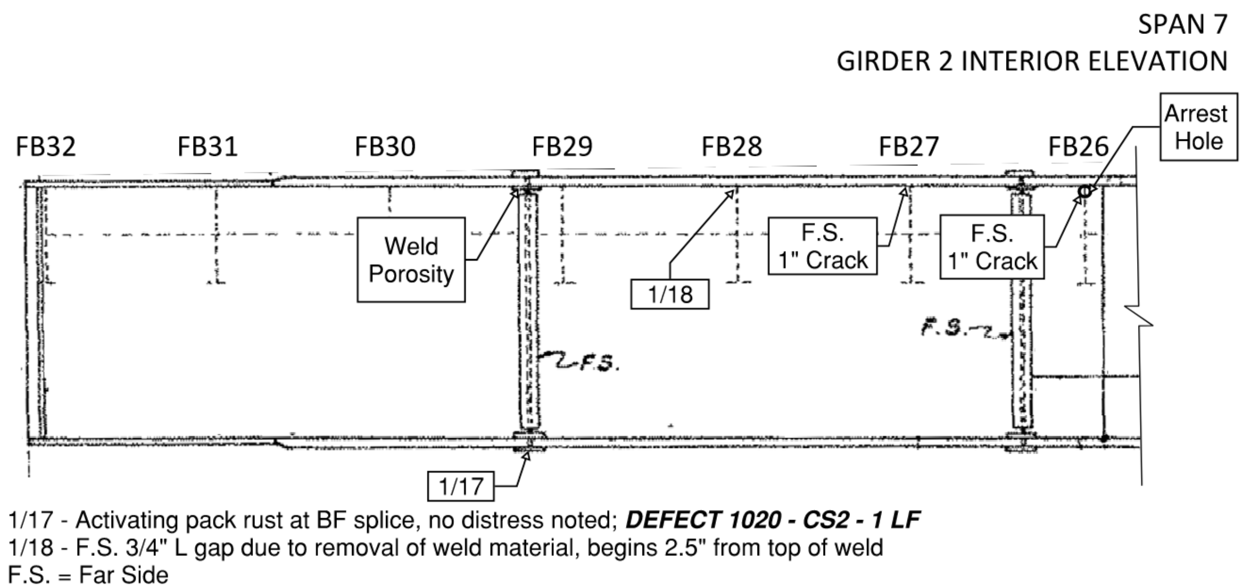


Figure 18.2.6 Girder Elevation

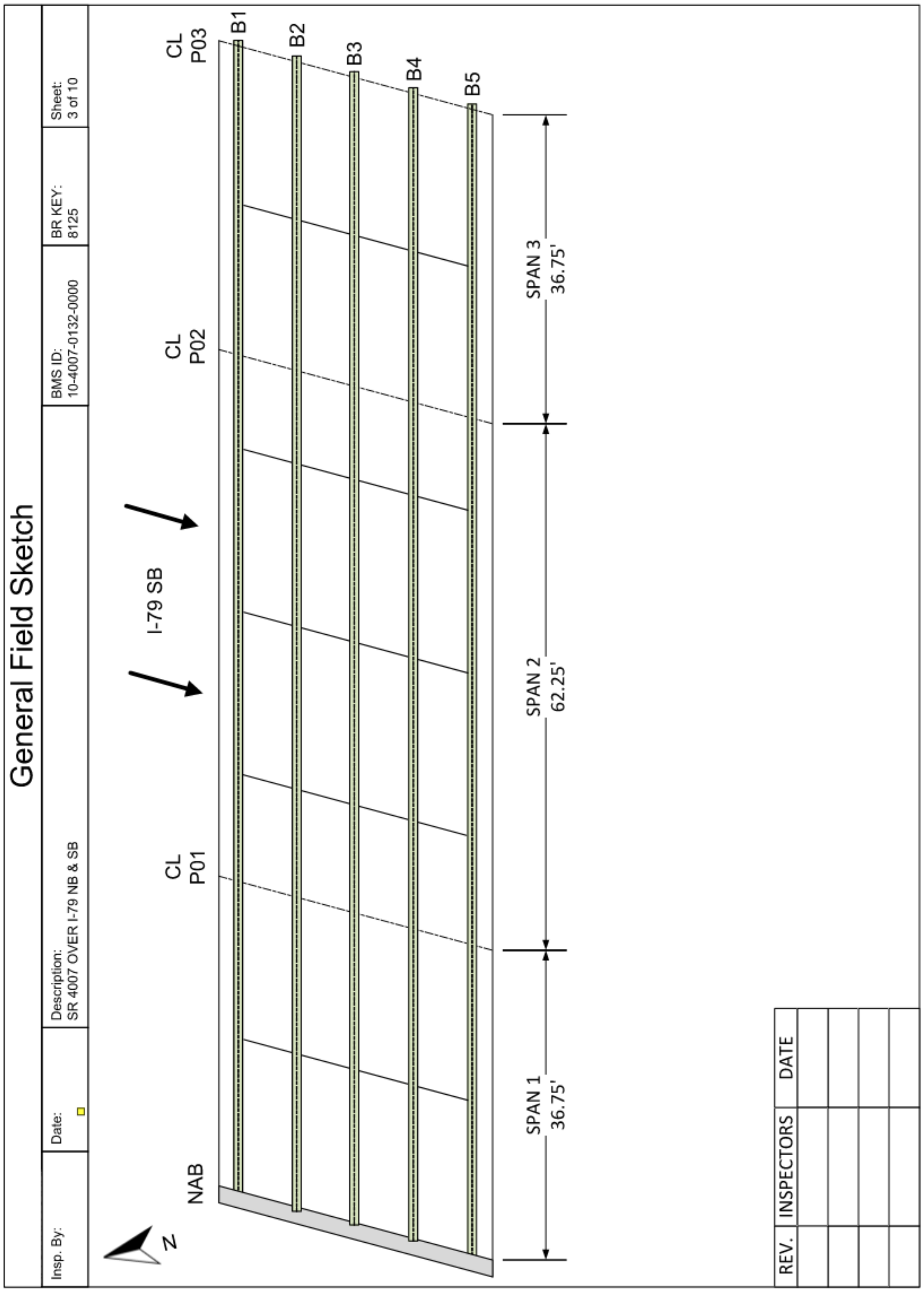
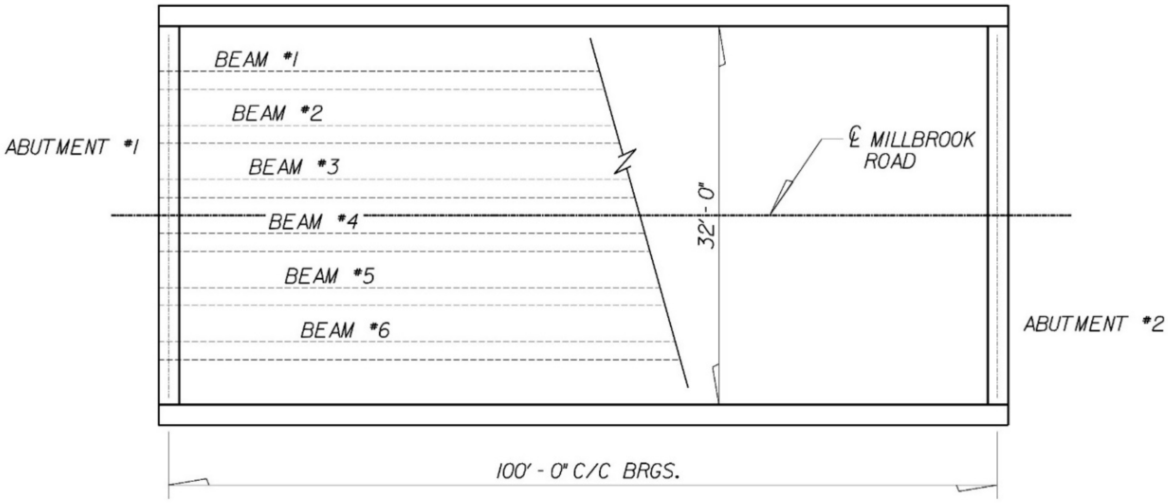


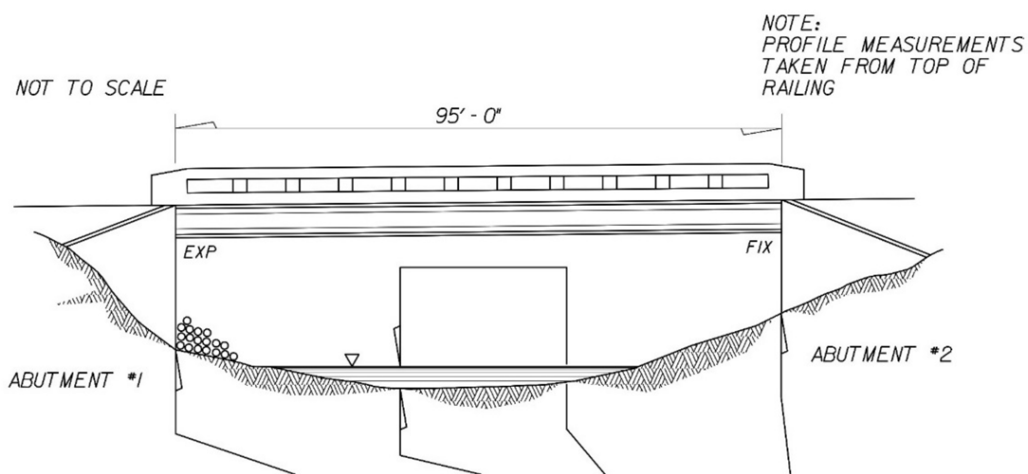
Figure 18.2.7 Steel Girder Framing Plan

The first sketch in the field inspection notes typically portrays the general layout of the bridge and site information, illustrating the structure plan and elevation data (see Figure 18.2.8). Including a north arrow can be helpful. The immediate area, the stream or terrain obstacle layout, major utilities, and any other pertinent details should also be included.

MILLBROOK ROAD OVER FLAT BROOK



PLAN VIEW



DATE	U/S	D/S		T/W	U/S	D/S
10/89	14.7'	———	18.2'	15.6'	11.0'	———
10/95	14.9'	14.6'	18.5'	16.2'	10.8'	11.1'
10/97	14.8'	———	18.2'	16.4'	10.8'	———
9/99	14.8'	14.9'	18.2'	16.0'	10.7'	10.6'
10/03	14.3'	15.1'	17.0'	15.3'	10.9'	10.8'
10/05	14.8'	15.0'	18.1'	16.6'	10.9'	10.4'
10/07	15.0'	15.0'	18.0'	16.1'	10.8'	10.8'

SOUTH (UPSTREAM) ELEVATION

Figure 18.2.8 Sample General Plan and Elevation Sketch

Defect Identification and Documentation

Identification of deficiencies is an important part of an inspection report. Baseline conditions observed in the initial inspection should also be documented. This practice establishes any defects that were present just after construction, so that the tracking of deterioration over time may be accomplished and understood.

Deficiencies of each material should be identified using terminology and descriptions consistent with those presented in the following chapters:

- Chapter 9 - Concrete.
- Chapter 10 - Steel.
- Chapter 11 - Timber, Masonry, and FRP.

The exact location, severity, and extent of deficiencies are typically used to track deterioration over time and are also used to determine the capacity of the bridge in its' current condition. The severity of a deficiency should be described. For example:

- Crack sizes - record lengths, widths, depth, and orientation.
- 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 over a given length.

The dimensions of a defect should be described. Abbreviations of dimensions may be used where appropriate. For example:

- Spalling - 2 ft x 3 ft x 2 inches deep.
- Scaling - 4 ft high by full abutment width.
- Delamination - 12 inches x 6 inches.
- Decay - 2' x 2' x 3" deep.

If there is more than one similar deficiency, their extent should be noted. For example, "Several smaller spalls approximately 1 foot x 1 foot x 1-1/2" deep are present throughout spans 1 & 3." and "Scaling is present throughout nearly the entire length of both side faces of the concrete deck." Notation of all deficiencies is important, but those found in areas of high stress are of particular importance. Refer to Chapter 6 for further guidance on Bridge Mechanics. Particular inspection areas per material type are also outlined in various Chapters for reference.

The exact position of the deficiency on the member should be noted, especially if load capacity may be affected or analysis is to be performed. For example:

- Left side of web, top half, 3 ft from north bearing.
- Top of top flange, from 3 ft to 6 ft west of Pier 2.

The ability to accurately track defects and deterioration over the life of the bridge and perform the load capacity analysis depends on precise location information for deficiencies. Inspectors should identify where deficiencies are situated in respect to permanent reference points.

Deficiencies can affect bending moment capacity at several locations along the length of a member. Maximum positive moment tends to occur at or near midspan. Maximum negative moment tends to occur at the intermediate supports if the structure is continuous.

Maximum shear forces usually occur at or near the supports. Bearing is maximized at the supports.

Axial compression members are generally compromised if there are deficiencies anywhere along the length of the member; therefore, all segments are critical. 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 situated.

Axial tension 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 where the deficiency is found.

Though axially loaded members are critical at all locations, it is not always apparent which members are loaded only in an axial direction. Due to the dead load of the member itself, most are not. Other factors can also contribute to bending forces that might create varying moments, shears, compression, and tension areas within a member that is primarily axial. Because of this, the exact position of the deficiencies should be identified 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 point. Inspectors should avoid using reference points that can change over time.

Some examples of proper referencing include:

- 7 ft-3 inches from fixed bearing on Beam 3 at Abutment 1.
- 3 ft-1 inches from west corner of Abutment 2.
- 2 ft-6 inches below bearing 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, inspectors should include the condition of deck and haunch, expansion joints, construction joints, curbs, sidewalks, parapets, and railings with the deck sketches. An example of various deck inspection notes is shown in Figure 18.2.9.

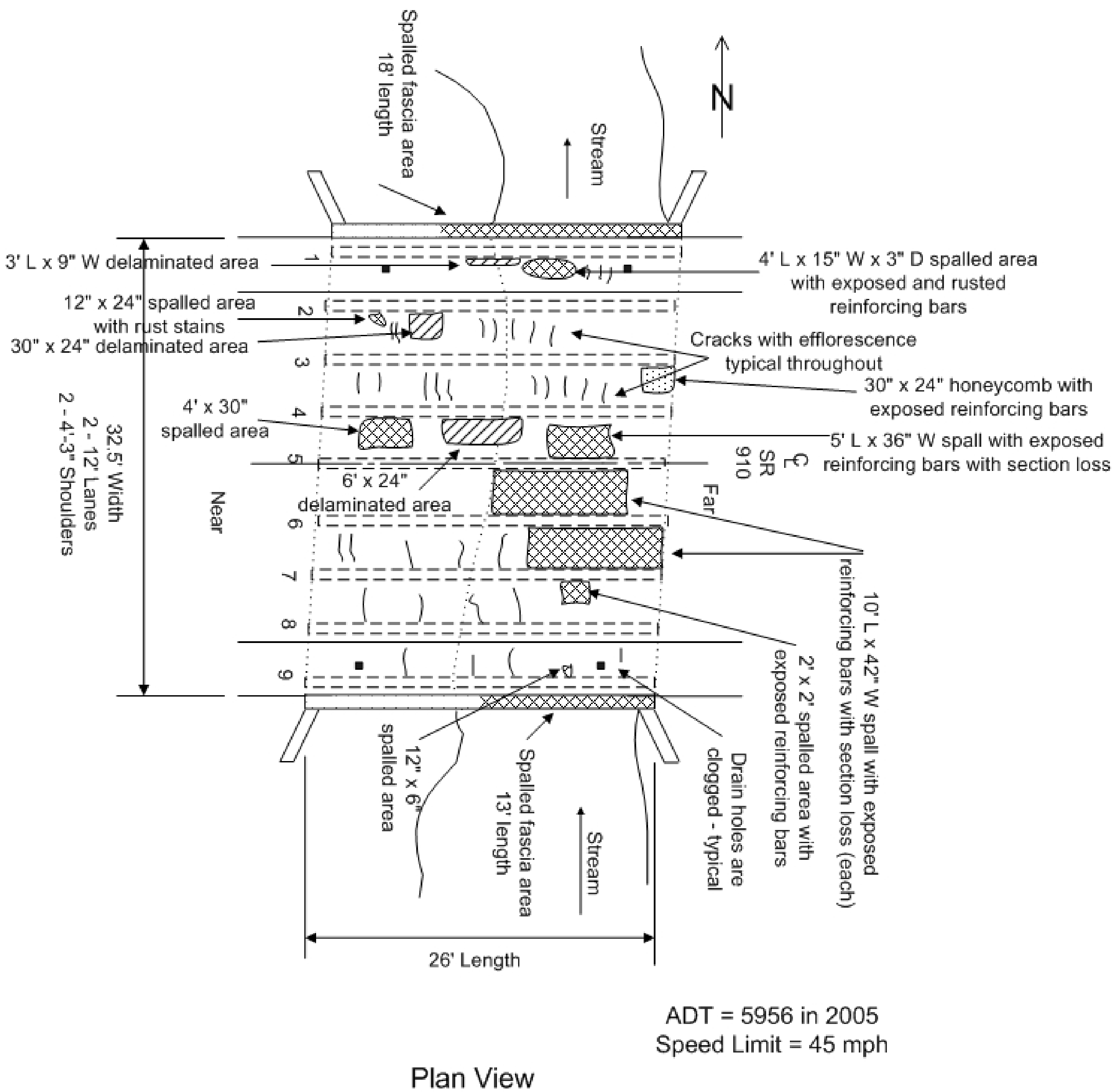


Figure 18.2.9 Sample Deck Inspection Notes

When documenting the deficiency location of the superstructure, the inspector should sketch the superstructure units in the plan view, and the elevation view or cross section if necessary. Items that should be inspected include bearings, main-supporting longitudinal members, floorbeams, stringers, bracing, and diaphragms (see Figure 18.2.10).

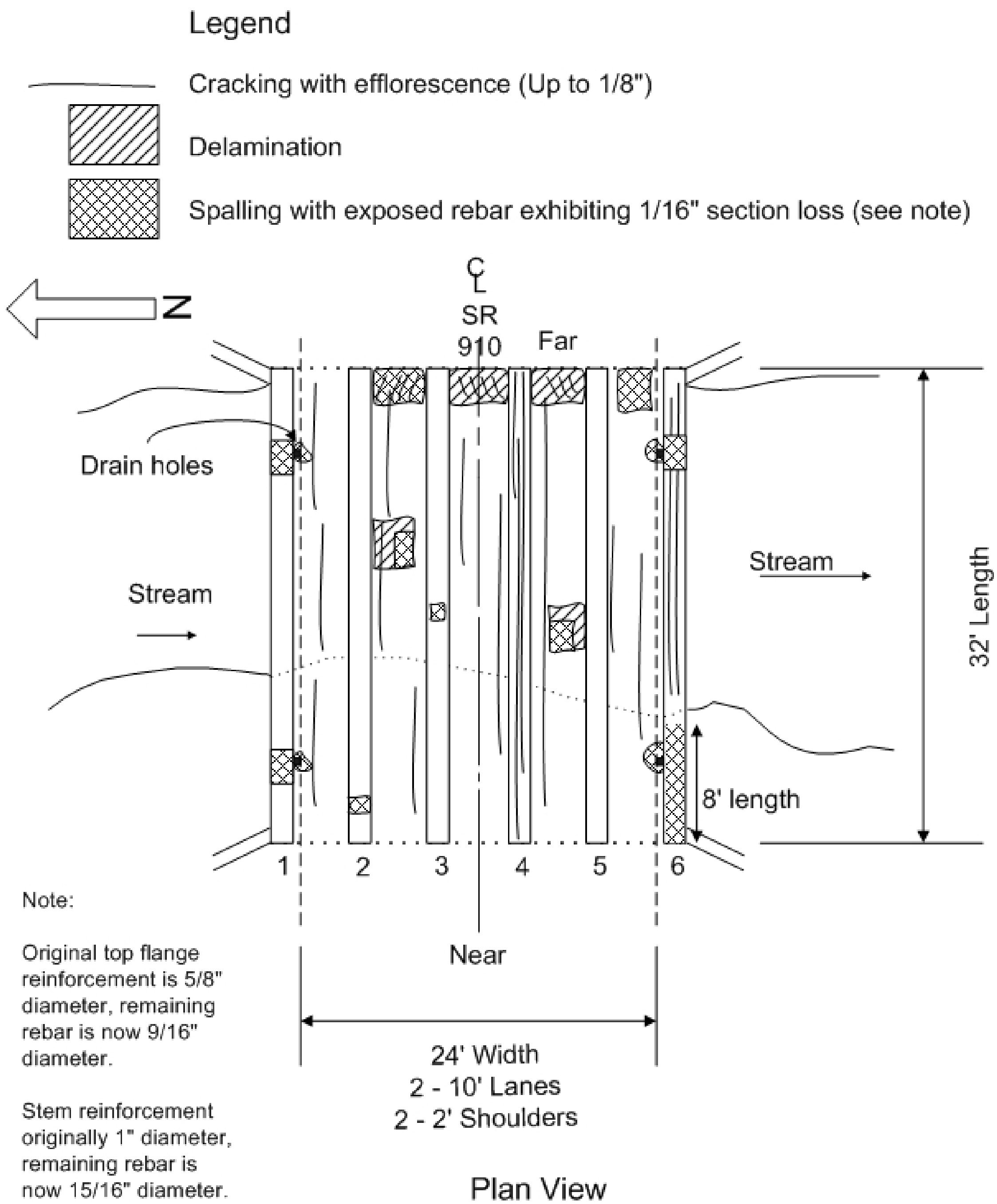


Figure 18.2.10 Sample Superstructure Inspection Sketch with Notes

Inspectors should include sketches or drawings to describe the condition of each substructure unit (see Figure 18.2.11). In many cases, it may be sufficient to draw typical units that identify the principal elements and deficiencies of the substructure. Each element of the substructure unit should be identified so that they can be cross referenced to the notes or sketches. Items that should be identified include piling, footings, vertical supports, lateral bracing of members, and caps.

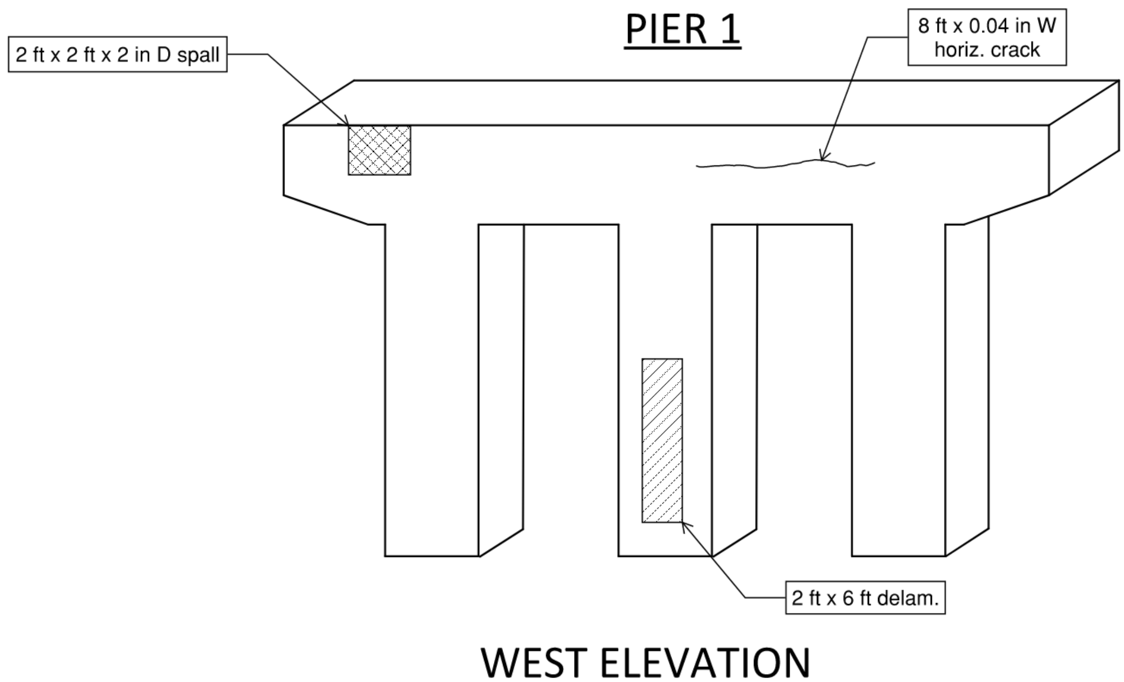


Figure 18.2.11 Sample Substructure Inspection Notes

Sketches or drawings should be included to describe the condition of the channel (see Figure 18.2.12). Streambed materials, alignment, condition of the banks, and the condition of the bottom of the waterway (including scour holes) should be included in the sketch.

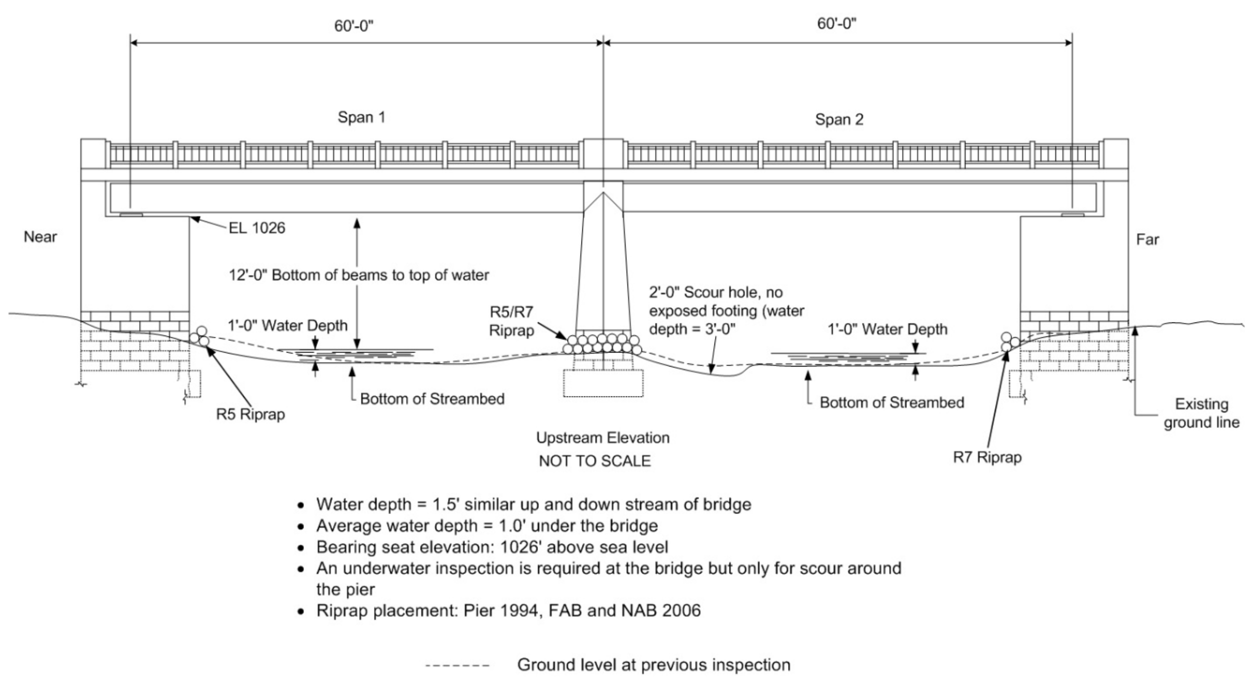


Figure 18.2.12 Sample Channel Inspection Notes

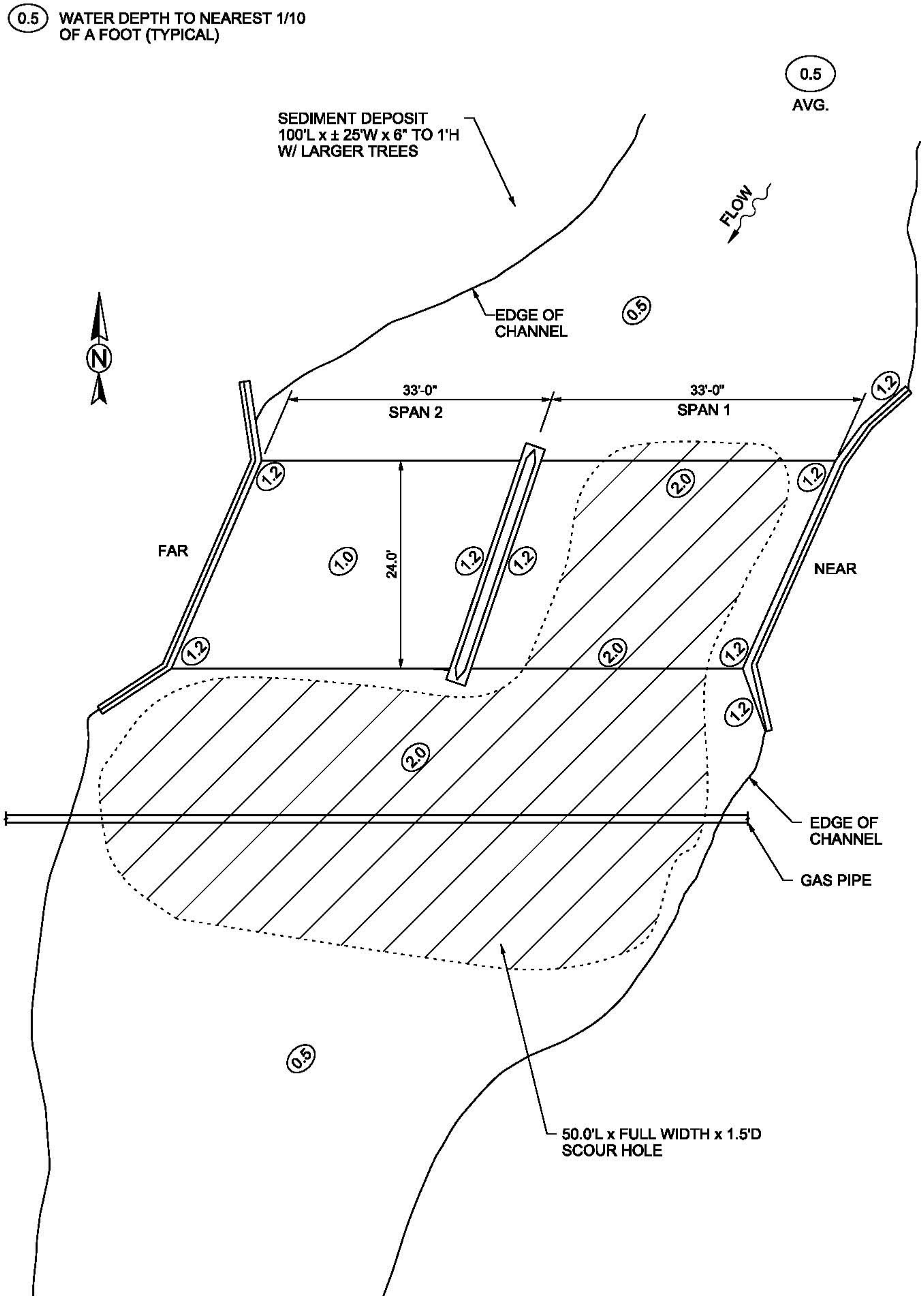


Figure 18.2.13 Sample Stream Sketch

Summary of Findings

All deficiencies should be reported, although minor widespread or numerous deficiencies can often be described by severity and extent instead of individually. Also, inspectors should note their type and location as well.

If there are no deficiencies present, the inspection team should indicate within the findings that no deficiencies were found. This note serves as confirmation that the area was fully inspected.

18.2.3 Routine Bridge Inspection Report

Minimum Contents

Inspection Results

Narrative descriptions of the conditions both quantitative and qualitative should be provided, indicating the extent of the affected areas and where they are situated. Inspectors should use agency-approved forms consistent with similar inspections. All signs of distress, failure, or deficiencies should be noted with sufficient precision so that a condition rating can be accurately assigned, and a deterioration rate can be determined. This is also important for determining estimated remaining life and an optimal preservation strategy. Photographs should be captured in the field to show deficiencies and cross reference in the report or on forms where deficiencies are noted. Photographs are most effective when more than one is used to show a different perspective of issues, such as different angles or a general view followed by a close-up view. Written notes should be supplemented with sketches and photos to show position and physical characteristics of deficiencies, including a known object in the photograph for scale reference.

Any load, speed, or traffic restrictions on the bridge should be noted. Inspectors should indicate if the signs are missing or damaged. Approach roadway photographs should be taken to confirm placement of load posting signs that includes the approach roadway, bridge, and sign. Inspectors should check for and note any advanced warning signs. Information about high water marks and unusual loadings should be included. The weather conditions such as temperature, rain, or snow should be noted. Inspectors should note work or repairs to the bridge completed since last inspection. Inspectors should verify or obtain new dimensions when improvement work has altered the structure. Updated streambed profiles and cross sections to detect scour, channel migration, or channel aggradation and degradation should be obtained. Any channel restrictions (e.g., debris) that could impact stream flow and increase scour potential should be noted.

If an Underwater, Nonredundant Steel Tension Member, or Damage inspection is necessary for a bridge, the results of that inspection, including condition rating recommendations, should be incorporated into the Routine inspection report. Details from the non-routine reports should be specifically cross-referenced.

Field Inspection Notes

Inspectors should 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, which emphasizes the importance that the notes be legible.

Drawings and Sketches

Sketches and drawings necessary 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). Cross section sketches and channel profiles provide valuable information to determine scour potential.

Original drawings are typically very helpful during future investigations with determining the progression of deficiencies 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 to accurately describe and document a particular deficiency.

Inspection Forms

The inspection forms typically contain the actual field notes, as well as the numerical condition and appraisal ratings by the inspection team. The inspection forms are typically signed by the inspection team leader. A complete NBI data summary is included in the appendix. Inspectors should compare previous inspection forms to current conditions for inventory data accuracy. Inspection forms are useful since many of them are checklists and these lists help remind inspectors where to look.

Photographs

Photographs are an important asset to anyone reviewing reports on bridge structures. It is recommended that pictures be taken of any typical and significant defects. Pictures should be taken even if you think you can explain it completely in writing. It is preferred to take several photographs that may be considered unessential than to omit a photograph that could cause misinterpretation or misunderstanding of the report.

At a minimum, two general photographs of every structure should be provided. One of these depicts the structure from the roadway. The other photo is a view of the side elevation (see Figure 18.2.14 and Figure 18.2.15). Additional recommended standard photographs include a typical section facing both directions, photos looking upstream and downstream from the bridge, and an elevation view of the structure from both upstream and downstream. A photograph of the typical underside view of the bridge can be very helpful in showing the superstructure type and layout of each span. Captions should be provided for each photograph. Clarifying graphics, such as arrows or short narratives, may be added to photos to enhance the photo’s purpose. Photographs should be 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 18.2.14 West Approach

Figure 18.2.15 Downstream Elevation

Preferred Additional Information

Preferred additional information is typically provided in consultant-developed reports for completeness. Owner content needs govern in the report preparation.

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. The level of detail is typically dependent upon the level of complexity of the bridge inspection report.

Location Map

A map is typically included with a scale large enough to positively locate the structure. The bridge should be clearly marked and labeled, and the map should have a north arrow 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 should contain all pertinent data concerning the design, construction, and service of the bridge. The type of superstructure should generally be given first, followed by the type of abutments and piers or bents, along with their foundations. If data is available, the type of foundation soil, maximum bearing pressures, and deep foundation capacities should be indicated. The type of deck should also be 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 a 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 that 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.

The design information in the report typically 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.

A construction history of the bridge typically 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.

Material testing may be performed on a structure in order to determine the strength and properties of an unknown or suspect material. The testing lab’s report can be included as an attachment or important excerpts may be added to the report in the bridge history.

The annual average daily traffic (AADT) count and the annual average daily truck traffic (AADTT) count should be included, along with the date of record. The AADT and AADTT should be updated at intervals in accordance with the standards for the HPMS and standards/policies within the state. Other service data to consider includes the year of AADT and AADTT, 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, should be noted in the report.

Inspection Procedures

The procedures used to inspect the bridge should be documented in the inspection report. In most instances, it may be 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 can be useful in planning future inspections and should also serve as a checklist to make sure that all elements and components were inspected.

The following information is typically included:

- Necessary equipment (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, flaggers, and hours of operation).
- Necessary permits (e.g., railroad or Coast Guard).
- Inspection methods (e.g., visual, physical, or advanced).
- Personnel (e.g., by name and classification).
- Special equipment (e.g., UAS, material testing, and underwater inspection).
- Necessary time for inspection.

When structure plans are not available in the bridge records and a load rating is necessary, field measurements obtained during the inspection may be necessary to assist in the calculation of the load capacity of the structure.

Executive Summary

The executive summary should be a concise narrative presentation summarizing the inspection procedures and findings in regard to the qualitative condition and the load capacity of the bridge, along with an overview of recommendations including any high priority repair items.

Conclusions and Recommendations

A good inspection report explains in detail the type, severity, and extent of any deficiency found on the bridge. It also points out any deviations or modifications that are contrary to the "as-built" construction plans or significant changes as compared to the previous inspection. The depth of the report should be consistent with the importance of the deficiencies. Not all deficiencies are of equal importance. For example, a transverse horizontal crack in a prestressed concrete box beam which enables 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 wingwall.

The inspector's experience and judgment are called upon when interpreting inspection results and arriving at reasonable and practical conclusions. Improper and misinformed conclusions may lead to improper recommendations. The inspection team should have an understanding of the loading conditions and the bridge site to conclude why, how, or when certain deficiencies occurred. It is important to seek advice from more experienced personnel when necessary to interpret the inspection findings.

The recommendations made by the inspection team constitute the "focal point" of the operation of inspecting, recording, and reporting. The inspection team 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, that range from the most severe or significant (critical) to a maintenance item that is considered routine or may only necessitate monitoring (non-critical). The specific prioritization levels are set forth by each bridge-owning agency. Examples of agency priority maintenance procedures are listed in Chapter 3.

The inspection team makes the initial determination whether a deficiency is a critical finding and necessitates immediate action using agency procedures. Usually this is easily determined, but occasionally the experience and judgment of a professional engineer may be necessary to reach a proper decision. A large hole through the deck of a bridge should obviously have close observation, 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 recommendations submitted by the bridge inspection team should be in the category of non-critical work. The recommended work is carefully described in the report, usually with a cost estimate. The conclusions and recommendations section of the report summarizes the following:

- Overall condition.
- Major deficiencies.
- Load-carrying capacity.
- Recommendations for:
    - Further inspection.
    - Maintenance.
    - Repairs.
    - Posting.
    - Rehabilitation.
    - Replacement.

Some state and local agencies designate separate personnel, not the inspection team in the field, to prepare recommendations and cost estimates.

Load Rating Summary

A summary of any load capacity rating analysis that has been performed should be included in the report. The summary may be presented in a table or chart. Governing load ratings should be shown for both inventory and operating levels for all types of loadings used in the analysis. Inspectors should identify the governing member for each rating. The governing member is typically 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 SU7 trucks, compared to all other stringers, floorbeams, or girders. This would be identified as the governing member in the load rating summary. The HL-93 inventory and operating ratings should also be included in the load rating summary and are reported to FHWA.

Not all inspections necessitate a new load rating analysis. A new load rating analysis is typically performed if the condition of the primary members has changed considerably since the last inspection. The report should also include 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.

18.2.4 Initial Bridge Inspection Report

In addition to the items outlined for a Routine Inspection Report section above, an Initial inspection report could also include some further information helpful for this or future inspections.

Minimum Contents

An initial inspection report should include documentation of the full inspection of primary members of the bridge. This inspection is essentially the first Routine inspection and sets the baseline for future Routine inspections. Elements are identified and quantified during the Initial Inspection. Initial inspections should be completed within 3 months of a new or rehabilitated structure being open to traffic.

Also, the need for an underwater and/or NSTM inspection should be determined or verified during or before the Initial inspection. A scour appraisal should also be performed for bridges over water. If necessary, these NSTM or Underwater inspections should be completed within 12 months of a new or rehabilitated structure being open to traffic.

Preferred Additional Information

An initial inspection should confirm the dimensions and information portrayed on the design and as-built plans.

18.2.5 Damage Inspection Report

A Damage inspection report would likely include more localized information than a Routine Inspection Report, as the Damage inspections do not include a review of the entire bridge, but should detail all portions of the bridge that were affected.

The scope of the Damage inspection should be sufficient to determine any necessary repairs, traffic restrictions, or bridge closures in extreme situations.

Minimum Contents

It is imperative to thoroughly document the location and dimensions of all damaged members of the structure. Photos and sketches should be included, as well as detailed notes explaining the extent of damage. Examples of damage may include vehicle or vessel collisions, seismic events, or weather-related damage. These inspection types are generally scheduled by the bridge owners.

If any advanced testing is completed, the results of these tests should be documented for future reference.

Preferred Additional Information

Accident reports for the incident may be included with the inspection report. Repair details or plans for future action may also be incorporated.

18.2.6 In-Depth Inspection Report

An In-Depth inspection serves to provide a detailed, hands-on inspection of one or more bridge members using visual or nondestructive evaluation (NDE) techniques as necessary to identify any deficiencies not readily detectable using routine inspection procedures. In-depth inspections may occur at intervals more or less frequent than routine inspections, as necessary for a particular bridge.

A common example of an in-depth inspection is a hands-on inspection of a pin and hanger system, utilizing NDE testing procedures. The report would in turn document the procedures, scope, and results of that evaluation.

Minimum Contents

An In-Depth inspection report typically includes supplementary data, as compared to a Routine Inspection Report. The report should identify the members and elements inspected, procedures used, observations and findings, condition rating determinations, and recommendations. Procedures should be in place to ensure that condition ratings and load ratings controlled by the findings of the inspection are reflected in the state or agency database and the subsequent Routine inspection report.

Preferred Additional Information

Executive Summary

The executive summary should be a concise narrative presentation summarizing the inspection procedures and findings in regard to the qualitative condition and the load capacity of the bridge, along with an overview of recommendations including any high priority repair items.

18.2.7 Nonredundant Steel Tension Member Inspection Report

A Nonredundant Steel Tension Member inspection report typically includes more specific information on nonredundant steel tension members (NSTMs) and fatigue-prone details than a Routine inspection report.

Minimum Contents

The NSTM inspection report should include the NSTM inspection procedures, in addition to the findings and the identification of all the NSTM members.

It is imperative to thoroughly document both the location and AASHTO Fatigue Category of fatigue details in steel members in tension or experiencing stress reversal. These members should be identified before the inspection so the inspectors can perform detailed inspection procedures including hands-on inspection of these details and may be part of the deck, superstructure, or substructure. Fatigue details typically include areas where two or more pieces of steel are connected by welding, bolting, or riveting. Photos and sketches should be included, as well as detailed notes explaining the AASHTO Fatigue Category or extent of damage.

If any advanced testing is completed, the results of these tests should be documented for future reference.

The report should also contain the recommended NSTM Inspection Condition rating based on the condition of the NSTMs only. For a bridge with NSTMs in both the superstructure and substructure, report only the lower of the two condition values (*SNBI* Subsection 7.1, Item B.C.14 Commentary).

Preferred Additional Information

Repair or retrofit details or plans for future action may also be incorporated. Often, finite element modeling is used to estimate local stresses before and after retrofits have been designed. Reduced inspection intervals may also be conducted until the bridge owner has confidence a retrofit detail has adequately lowered the stress in the fatigue-prone areas. Typically, documentation of the completed repair is valuable to include in the inspection report.

Any in-depth structural analysis calculations, or reference to analysis, providing documentation of internal or system redundancy for the bridge or specific NSTMs may be included in the report.

18.2.8 Underwater Inspection Report

An Underwater inspection report is typically prepared by the inspectors or dive team and includes inspection of the bridge components and channel under the water elevation. If an Underwater inspection of the substructure has been performed, the summary of findings of the Underwater inspection report is usually included in the Routine inspection report or cross-referenced to another location of the report.

Minimum Contents

Since most Underwater inspections only focus on the substructure, the reported conditions and element data included in the report mainly are geared towards only a few of the bridge members. It is typically important to identify which bridge members were included. The report should contain a list and description of the substructure units that require an Underwater inspection, verification that each was inspected, and the applicable inspection intervals. Underwater inspection procedures should be documents either directly in the inspection report, or otherwise provided in the bridge file.

The report should also contain a recommended Underwater Inspection Condition rating, based on the condition of the underwater members of the substructure only. The condition rating for the underwater members will then be incorporated into the Substructure Condition Rating.

Preferred Additional Information

Advanced Underwater inspection methods may be utilized to gather scour or undermining data at the bridge. Any relevant inspection data should be included in the report. If there are testing results, they may also be included.

Good photographic documentation can provide vital support and validation to the conclusions reached during an inspection. Although more difficult under water, attempts should be made to take photos below water level whenever feasible. This is particularly important if there are deficiencies to record. If photos are not possible, thorough inspection sketches are key to the understanding of the underwater substructure condition. A sketch that clearly identifies all underwater elements inspected can be helpful.

Because taking quality photographs is difficult underwater in limited visibility conditions, special techniques for photography may be necessary such as use of a clear water box, or an Underwater inspection could include audio or video recordings taken during the inspection. These techniques should be described and referenced in the report. If any acoustic imaging methods are utilized during the inspection, the methodologies and findings should be reported.

An Underwater inspection report should also include a water depth soundings log. Inspection notes and sketches should focus on the bridge substructure and waterway details.

The dive plan and job hazard analysis are also generally a part of an underwater report. It is good practice to include information regarding the dive team, any specialized equipment used, and any bridge specific underwater inspection procedures if they apply.

18.2.9 Special Inspection Report

A Special inspection report documents the inspection or monitoring of a particular known deficiency(ies) at a bridge site. This documentation of monitoring could be included as a supplemental to the Routine inspection report.

Although Special inspections are not sufficiently comprehensive to meet NBIS requirements as routine inspections, these activities may be performed at the same time as the routine inspection. The Special inspection report is generally a brief report, focused on the known deficiency.

Minimum Contents

The information within a Special inspection report should primarily focus on the member that is identified for inspection or periodic monitoring by the bridge owner. At a minimum, the report should outline the scope of the inspection and document all findings. It should also detail the reason for the Special inspection and make recommendations for condition rating changes, repairs necessary, or future monitoring activities.

For example, based on a recent gusset plate failure, states completed special inspections on all truss bridges with gusset plates. The gusset plate inspection reports commonly included an inventory of all gussets, along with sketches, photos, and records of section loss on each gusset plate. Some reports included load capacity analysis of particular gusset plates.

If specialty inspection personnel are required to perform Special inspections, document the qualifications of these personnel within the report. Examples include nondestructive testing experts or weld inspectors.

Special inspections may also be used to provide a reduced interval for a particular detail or member and would therefore be scheduled on a regular basis.

Preferred Additional Information

If a particular member has had multiple Special inspections, the historical data and monitoring information should be included in the report.

18.2.10 Scour Monitoring Inspection Report

The Scour Monitoring inspection type is utilized when a scour Plan of Action (POA) necessitates scour monitoring. The inspection is performed during or after a triggering storm event, by personnel with appropriate qualifications (*SNBI* Definitions). This may include periodic remote electronic readings of streambed changes, when necessary, in the POA.

At a minimum, the inspection report should contain the date, scope, and results of any inspection or monitoring activity. Any updates to the Scour Condition Rating as a result of this inspection should be reported.

18.2.11 Service Inspection Report

A Service inspection is an interim inspection performed between routine inspections for a bridge with a risk-based routine inspection interval that exceeds 48 months. The Service Inspection detects any major visible safety deficiencies that have developed since the previous Routine inspection.

The field inspection should be performed by personnel with general knowledge of bridges. The results should be documented in the bridge file and include the date, scope, and any significant findings that differ from the previous findings from the full Routine inspection. If significant changes in conditions have occurred, the Routine inspection interval should be reassessed accordingly.

Section 18.3 NBI Data Preparation and Reporting

18.3.1 Introduction

The FHWA collects and inventories specific National Bridge Inventory (NBI) data as outlined in the *SNBI*, as a measure of ensuring that all bridges on public roads are inspected and maintained properly. The bridge owner is responsible for all preparation and proper reporting of this inventory data, in accordance with 23 CFR 650.315.

18.3.2 NBI Reporting Data

A complete list of data items and detailed information on each item can be found in the FHWA *SNBI*. The data is categorized as follows:

- Bridge identification.
- Bridge material and type.
- Bridge geometry.
- Features.
- Loads, load rating, and posting.
- Inspections.
- Bridge condition.

Bridge Identification

Bridge identification information includes data with regards to the identification, location, and classification of the structure. This includes the identifying bridge number and name for the bridge. The location information collected contains the geographical data for the bridge, such as state and county, latitude and longitude, and highway agency district. Special considerations are included for bridges which span borders. The classification of a bridge is made up of the ownership and maintenance responsibility, as well as other pertinent data varying by bridge. These items generally remain consistent over the life of a bridge.

Bridge Material and Type

Bridge material and type data items are grouped into three subsections including span material and type, substructure material and type, and roadside hardware. In these subsections, materials, structure types, and configurations that make up the bridge are identified. The number of spans, deck and wearing surface materials (if present) are also inventoried. Substructure and foundation data are also recorded. Roadside hardware identifies crash-tested hardware such as bridge railing and guardrail transitions. A majority of the items in this section will not change once the bridge has been inventoried after an initial inspection.

Bridge Geometry

Bridge geometry data is verified and reported for each bridge. These items can include dimensions such as NBIS bridge length, total bridge length, maximum and minimum span length, and bridge width. These items also document the presence and geometry of sidewalks and median barriers.

The skew, as well an any curvature, among other items, are also reported. Any geometric data changes that may have occurred based on rehabilitation or retrofit should be noted.

Features

Bridge feature data is grouped into five subsections including feature identification, routes, highways, railroads, and navigable waterways. Feature data items identify what is above, below, and carried by the NBIS inventoried bridge. Once the bridge is inventoried after an initial inspection, these data items will typically remain unchanged.

Loads, Load Rating, and Posting

Data in this section is grouped into three subsections including load and load rating, load posting status, and load evaluation and posting. Loads and load rating data describe the load carrying capacity of a bridge and the methods used to determine the load rating and posting. The design loads and methods should be recorded for future reference. The results of the load rating should be reported based on the load rating method. If a posting limit is assigned due to a reduced load capacity, the weight limits, vehicle types, and posting dates should be reported posting data items. Some of the data items remain static once a bridge has been inventoried, but others may change after reevaluation of the load rating.

Inspections

Inspection items summarize the type and interval of inspections to be performed on the bridge. These *SNBI* data items are grouped into two subsections including inspection requirements and inspection events. Inspection requirements data specify whether necessary non-routine inspection types are required, and any special inspection features present.

Inspection events data include information on each inspection performed for the bridge, including team leader identification, and inspection equipment used during the inspection. Data for these items can change from inspection to inspection. It is imperative to code the proper type of inspection and the inspection interval.

For all field inspections, the type and date should be reported. The Inspection Type is captured with Item B.IE.01. The corresponding dates are reported with Item B.IE.02 Inspection Begin Date and Item B.IE.03 Inspection Completion Date. For single day inspections, report the same date that the field inspection begins.

Inspection Interval, Item B.IE.05, should reflect the planned interval at which the bridge is to be inspected per the NBIS. This applies for most inspection types, but Damage, Scour Monitoring, and Special inspections likely do not have a defined interval and therefore a 0 is reported for this item.

Item B.IE.11, Inspection Note, should be used to describe the members inspected, when only portions of the bridge are inspected. For example, during a Damage inspection for collision damage, the Inspection Note item should detail which members were inspected for the resulting damage. This item may not be necessary for Routine inspections if all portions of the bridge are inspected.

Inspection Equipment is reported using Item B.IE.12. This item includes information about standard and special access equipment used to perform the inspection, as well as underwater and nondestructive evaluation or testing equipment. This should be verified and updated if needed during each inspection.

Bridge Condition

Bridge condition data is reported with both Component Condition Rating data items and Element Identification and Condition data items.

Component Condition Ratings

Bridge condition data items provide information about the state of the bridge or culvert and any adjacent waterways. These items include component condition ratings and element conditions, which are detailed in Chapter 3 and Section 18.1. Additionally, data for element identification, appraisal, and work events are also included in the Bridge Condition Section. Work events data records the year that the bridge was built, along with subsequent work or maintenance performed on the bridge.

Component condition ratings record the condition of all aspects of a bridge including bridge railing and transition, bearings, joints, channel, channel protection, and scour in addition to the traditional deck, superstructure, substructure, and culvert.

Element Identification and Condition

According to the *SNBI*, element data are only required to be reported to FHWA for bridges that carry NHS routes. Element data reporting for bridges carrying non-NHS routes is optional. Element identification data tends to remain static through the life of the bridge while other data types in this section may change periodically.

As discussed in Chapter 3, element level evaluation may include National Bridge Elements (NBEs), Bridge Management Elements (BMEs), and possibly agency developed elements (ADEs) as well. The AASHTO *Manual for Bridge Element Inspection* provides the basis for NBEs and BMEs. ADEs are not submitted to the National Bridge Inventory.

Depending on an agency's needs, ADEs generally fall into three main categories: subsets of NBEs, BMEs, or elements that are independent of defined elements.

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

18.3.3 Uses of Reported NBI Data

Data reported by bridge owner agencies is maintained in the NBI database and provides necessary information for state-level and national-level analyses and reporting. Information within an owner agency's inventory may be requested by the FHWA at any time.

The data informs Federal funding programs and streamlines the identification of freight and defense-critical corridors and connectors. It may be used by the media, governmental institutions, academic and research facilities, and members of industry. Having a complete and thorough inventory facilitates accurate reports based on the number, condition, and performance of the Nation's bridges.

Section 18.4 Quality Control and Quality Assurance

18.4.1 Introduction

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 should not 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 (23 CFR 650.313 (p)). QC and QA activities include:

1. Documentation of the extent, interval, and responsible party for the review of inspection teams in the field, inspection reports, NBI data, and computations, including scour appraisal and load ratings. QC and QA reviews are to be performed by personnel other than the individual who completed the original report or calculations.

2. Performing QC and QA reviews and document the results of the QC and QA process, including the tracking and completion of actions identified in the procedures.

3. Addressing the findings of the QC and QA reviews.

The AASHTO *MBE* provides guidance for the implementation of appropriate quality control and quality assurance procedures.

The FHWA has developed a recommended framework for the documentation of QC and QA programs for bridge inspection. It can be found on the FHWA website at https://www.fhwa.dot.gov/bridge/nbis/nbisframework.cfm and is outlined below:

1. Develop, document, and maintain Quality Control and Quality Assurance (QC and QA) procedures in accordance with this recommended framework. This may take the form of its own document or can be detailed within a bridge inspection manual.
2. Elaborate on the purpose and benefits of the QC and QA program.
3. Provide appropriate definitions.

18.4.2 Quality Control

Quality control procedures include the procedures that are intended to maintain the quality of a bridge inspection and load rating at or above a specified level (23 CFR 650.305). A QC review should verify that the report is complete and accurate. This included the checking of inspection report information before finalization. Also included is checking of calculations such as load ratings.

The following is a recommended framework for bridge inspection QC procedures:

1. Define and document QC roles and responsibilities.
2. Document and maintain a registry of qualifications required for Program Manager, Nationally Certified Bridge Inspector (Team Leader), 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 recertification training, including:
   a. NHI refresher and other training courses, other specialized training courses, and/or periodic meetings.
   b. Define recertification 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 and calculations such as load ratings performed under the supervision of a registered professional engineer or scour evaluations. Verify that load ratings were performed under the supervision of a registered professional engineer (23 CFR 650.309 (d)).
7. Document procedures for identification and resolution of data errors, omissions and/or changes.

The FHWA *SNBI* also includes Inspection Quality Control Date as an item within Subsection 6.2 Inspection Events. This item identifies inspections that have had independent QC reviews to maintain inspection quality at or above a specified level.

18.4.3 Quality Assurance

Quality assurance procedures include the use of sampling and other measures to assure the adequacy of QC procedures in order to verify or measure the quality level of the entire bridge inspection and load rating program (23 CFR 650.313). This is generally an independent review, completed by a party not directly involved in the inspection program.

The following is a recommended framework for bridge inspection 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 QA review results; including an annual report.
    b. Establish review interval parameters. Parameters should include:
        1). Recommended review interval 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.
        2). Recommended number of bridges to review.
    c. Procedures and sampling parameters for selecting bridges to review. Procedure should be a random sampling but could also consider some focus areas:
        1) Whether the bridge is or is not posted.
        2) Whether the bridge is programmed for rehabilitation or replacement.
        3) Whether the bridge has had critical findings and the status of any follow-up action.
        4) Bridges with unusual changes in condition ratings (e.g., more than 1 appraisal rating change from previous inspection).
        5) Bridges that require special inspections (underwater, NSTM, other special).
        6) Location of bridge.
        7) Structure type and material.
    d. Procedures for reviewing current inspection report, bridge file, and load rating.
    e. Procedures to validate qualifications of Team Leader.
    f. Procedures for conducting the QA field review including defining "out-of-tolerance" for condition rating and load rating. (e.g., component condition rating of +/- 1, Element Identification and Condition States or load ratings that differ by more than 15%).
    g. Checklists covering typical items to review as part of QA procedures.
        1) Bridge file contents.
        2) Field inspection reports.
        3) Load rating analysis.
        4) Others.

The FHWA *SNBI* also includes Inspection Quality Assurance Date as an item within Subsection 6.2 Inspection Events. This item identifies inspections that have had independent QA reviews to measure or verify the overall quality of the inspection program.

18.4.4 Roles and Responsibilities

Proper quality control and quality assurance is the responsibility of all parties involved in the bridge inspection process. Each individual generally has a particular role to ensure the quality of data gathered and reported during a bridge inspection and the subsequent management of that data.

Inspector (Non-Team Leader)

The inspector assists the team leader and is responsible for identifying and documenting the condition of the bridge at the individual sites. The inspector cross checks information with the team leader, in order for them to work as a team. They ensure quality by clearly describing any deficiencies and providing reference documentation for condition ratings assigned to each component. It is important to note the justification for assigned condition ratings, regardless of the degree of deterioration. This role is not defined by the NBIS.

Team Leader

The inspection team leader, being an active member of the inspection team, confirms the data with the other bridge inspector(s) on the team. The team leader is on site at all times and is ultimately responsible for the accuracy of the component condition ratings and element condition state evaluations for each bridge. Another role of the team leader is to certify completeness of the inspection report and quickly report any potential critical findings. The team leader is a nationally certified bridge inspector and must be identified within the report (23 CFR 650.305).

Consultant Project Manager

If the inspection work is performed by a consulting engineering firm, the consultant's project manager is the primary contact with their firm and the bridge owner. They are ultimately responsible for the quality of the inspection work that is submitted to the bridge owner.

Bridge Inspection Program Manager

The Bridge Inspection program manager is the one in charge of the program for the entire state or agency regardless of owner, and should put in place the QC and QA procedures for a particular owning agency or the entire state. The program manager is responsible for carrying out the steps within the framework of the procedures.

Quality Manager

Many consultant companies and owning agencies have a quality manager, which provides an additional layer of observation to quality. This person may also run the QC and/or QA programs and provide an independent review of inspection reports and data in respect to state or consultant written QC and QA plans before submitting the report to the owner.

FHWA

The FHWA oversees the adequacy of inspection data and reporting, and the inspection program as a whole. FHWA performs a compliance review of the state(s) or Federal agency(ies) to verify quality work is being performed in accordance to accepted QC and QA procedures. Inspection data is reported annually for this review per 23 CFR 650.313(r).

Section 18.5 Bridge Files

18.5.1 Introduction

The contents of any particular bridge file may vary depending upon the size, complexity, 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 AASHTO *MBE* provides requirements on the contents of bridge files and supplemental documentation. It is recommended that the following types of information be assembled when possible.

18.5.2 General File Information

According to the *MBE*, the bridge record includes the following general information. Typically, much of this information is contained in the bridge inspection reports that is also contained in the bridge file.

- General plan and elevation.
- NBI data summary sheets.
- Photographs.
- History of structural damage.
- Inventories and inspections.
- Inspection history.
- Critical findings and actions taken.
- Channel cross-sections and other waterway information.
- Inspection requirements.
- Load rating documentation.
- Posting documentation.
- Scour Assessment.
- Scour plan of action.
- Significant correspondence.

General Plan and Elevation

A general plan and elevation of the bridge is typically available via the structure plans, but some bridges may not have design or construction plans. In this case a sketch should be included, depicting the layout of the bridge. This information should be consistent with a plan and elevation drawing of a bridge and include clear labels for members and elements, which facilitates complete assessment of the bridge.

Record of NBI Data

A chronological record of bridge inventory information reported by the bridge owner should be included in the bridge record. Refer to Chapter 3 for a description of NBI data summary sheets.

Photographs

Photographs are typically used to supplement the inspection notes and sketches. Various photographs are generally included in the bridge record: a topside view of the bridge roadway and at least one elevation view of the bridge. Refer to Section 18.2.3 for more detailed recommendations. Photographs showing major deficiencies or other features, such as utility attachments or channel alignment, should also be included. Photographs that show load posting signs are also provided, if applicable.

History of Structural Damage

The inspection team should include details of accidents or damage to the bridge in the bridge record (see Figure 18.5.1). This information should include the date of the occurrence, description of the accident, member damage and repairs, and any investigative reports.

Figure 18.5.1 Bridge Damage from Construction Equipment

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.

Many bridge owners have standard inspection forms. These forms are generally used for each bridge in their system and give the inspection team a checklist of items that are to be reviewed. Another benefit of standardized forms is that it organizes bridge reports into a consistent format.

Inspection History

Reports from previous inspections can be particularly useful in identifying specific locations that necessitate special observation 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, deck evaluations, and corrosion studies are also typically included when available.

Critical Findings and Actions Taken

Inspectors should provide detailed descriptions, including photographs, of any critical findings. The information in the bridge file is typically considered sufficient to document safety or structural concerns. Any actions taken should be identified, immediately or as a follow-up. Any monitoring activities that may be considered necessary during future inspections should be noted.

Channel Cross-sections and Other Waterway Information

Inspectors should document and include channel or stream profile sketches for bridges over water. A history of the waterway with regards to channel shifting, extent of scour, and general stability of the channel should be provided.

A chronological history of major flooding events should be included (see Figure 18.5.2). This history typically includes the high-water marks at the bridge site and scour history.

Figure 18.5.2 Flood Event

Hydraulic data is also assembled where available, including structure profile grade line, 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. It is very important to provide this information, as many bridges have failed due to flood events.

Inspection Procedures

Bridge owners may have general overall inspection procedures outlined in their bridge inspection manual or from the AASHTO *MBE* that address common aspects of inspections, however, bridges requiring additional observation should have written inspection procedures specific to each bridge which address any unique items. Procedures aside, all members should be assessed during any routine, visual inspection. NSTM, underwater, in-depth, and complex feature inspection procedures should be documented per the NBIS (23 CFR 650.313). The following information should be included in the bridge file and be updated as appropriate if inspection procedures evolve:

- Each of the members to be inspected (NSTM elements, past repairs, underwater elements, complex features, fatigue-prone details, scour countermeasures, etc.) should be identified on plan sheets, drawings, or sketches.
- Special access needs or equipment necessary to gain access to inspect the features (under bridge inspection trucks, lifts, traveler system, climbing, etc.) should be identified.
- The inspection team should describe the inspection method(s) and interval to be used for the elements. For example, “Visually inspect all identified NSTMs at arm’s length for cracks, deterioration, missing bolts, loose connections, broken welds… using penetrant testing to verify the existence of suspected cracks.”
- Necessary proximity to details, such as “arm’s length”, should be addressed.
- The inspection team should identify special qualifications made necessary for inspection personnel by the program manager, if any (i.e., successfully passed a specialized training course, certified electrician for movable bridge electrical components, qualified bridge inspection diver, etc.).

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

- Special coordination procedures before 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 need to ensure inspection team and public safety, including a traffic management plan, are also included.

Refer to Chapter 2 for details on inspection fundamentals, specifically planning and performing the inspection and methods of access.

Load Rating Documentation

A complete record of the determination of the bridge's load-carrying capacity should be included in the bridge record. This information should include the design load to indicate the live load the bridge was designed for, the analysis methods used to determine the ratings, and the legal load ratings for the bridge.

The capacity calculations should be signed and dated by the individual who determined them, along with any assumptions used. Such analysis is typically performed by engineers in the office, not by the inspection team but must be performed under the supervision of a registered professional engineer.

Post or restrict the bridge in accordance with the NBIS, AASHTO MBE or in accordance with state law, when the maximum unrestricted legal loads or state permit loads exceed that allowed under the calculated rating factors. NBIS requires that posting shall be made as soon as possible, but not later than 30 days after a load rating determines a need for such posting. Posting date is recorded using *SNBI* Item B.PS.02, Posting Status Change Date.

Posting Documentation

Each bridge record should include load capacity calculations and any necessary 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 18.5.3). Proper approval forms should also be included.

If a posting situation is resolved, information regarding actions taken should be included as well.

Figure 18.5.3 Posted Bridge

Scour Assessment

The scour vulnerability of a bridge is appraised according to the NBIS and guidelines in the FHWA *SNBI*. Reference documents related to the development of the coding for this item is included in the bridge file.

Scour Plan of Action

If a structure is determined to be scour critical, further documentation should be provided. The scour plan of action (POA) typically provides procedures for bridge inspectors and engineers in managing bridges in their inventory determined to be scour critical, have unknown foundations, or identified as candidates for scour countermeasures. The document addresses a schedule for repair or installation of scour countermeasures, and/or the monitoring, inspection, closing, and opening a bridge to traffic during and after flood events to protect the traveling public. The POA is typically reviewed during each inspection to assure it is complete and current. Use *SNBI* Item B.AP.04, Scour Plan of Action to report whether a Scour POA exists.

Significant Correspondence

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

18.5.3 Supplemental Bridge Record Documentation

According to the AASHTO *MBE*, the bridge record may include the following supplemental documentation:

- As-built and construction shop drawings.
- Construction documentation.
- Original design documents (specifications).
- Unique considerations.
- Utilities or other attachments.
- Maintenance and repair history.
- Traffic data.
- Testing records.

As-built and Construction Shop Drawings

Construction, "as-built," or shop and working plans should be included in a bridge record. If plans are not available, the inspection team should determine the following types of construction information when possible: date built; type of structure, including size, shape, and material; design capacity; and design service life.

Construction Documentation

The bridge record can include 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 typically included in the file. The edition and date of the general specifications are noted in the bridge record.

Certificates for the type, grade, and quality of materials used in construction of the bridge are generally included in the bridge record. Examples include steel mill certificates, concrete delivery slips, and any other manufacturers' certificates. The certificates should be retained in accordance with bridge owner policy and statute of limitations. Coating history documents typically contain information about 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.

Original Design Documents

Design calculations and analyses should be included in the bridge file as well. Assumptions made during the design process and references to the design code used can be extremely helpful for development of repairs and in general during the life of the bridge.

Unique Considerations

The inspection team should keep a record of any unique coordination or access necessary for the inspection of the bridge. The actual use of the previously recorded access equipment should be verified during each inspection. Any safety or environmental concerns should also be noted.

Utilities or Other Attachments

Information on utility attachments or any other attachment to the bridge, should be identified and documented. Utilities in the general vicinity of the bridge are also useful, particularly as it pertains to access or safety.

Maintenance and Repair History

Information about repairs and rehabilitation activities are typically 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 quantities 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.

Traffic Data

The AADT should be updated at intervals in accordance with the standards for the Highway Performance Monitoring System (HPMS) and standards/policies within the state. When available, the bridge record contains a history of the variations in AADT and AADTT including the frequency and types of vehicles using the bridge. AADT and AADTT 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 should also be included in the bridge record.

Testing Records

Reports for any nondestructive or laboratory testing during or after construction should be included in the file. If any field load testing is performed, the inspection team should provide the reports in the bridge record. In addition to the reports being a part of the bridge record, any NDE methods used during the inspection should also be captured in *SNBI* Item B.IE.11, Inspection Note. Within the *SNBI* item, a reference to lab reports in the bridge record may be appropriate.

18.5.4 Bridge Management

The data in the inspection report can also be used for analysis by the bridge owners and the FHWA for bridge management. The intent of the analysis is to aid in the decisions 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.

18.5.5 Long Term Bridge Performance (LTBP) InfoBridge

The Long-Term Bridge Performance (LTBP) Program is a Federal Highway Administration (FHWA) long-term research effort to help the bridge community increasingly understand bridge performance. The overall objectives of the LTBP Program are to monitor representative samples of bridges nationwide to collect, document, maintain, manage, and disseminate high-quality quantitative performance data over an extended time horizon. This should be accomplished by taking advantage of advanced nondestructive evaluation (NDE) and structural health monitoring (SHM) technologies in addition to traditional visual bridge inspection approaches. The LTBP Program is designed to collect critical performance data and merge them with data gathered from available sources.

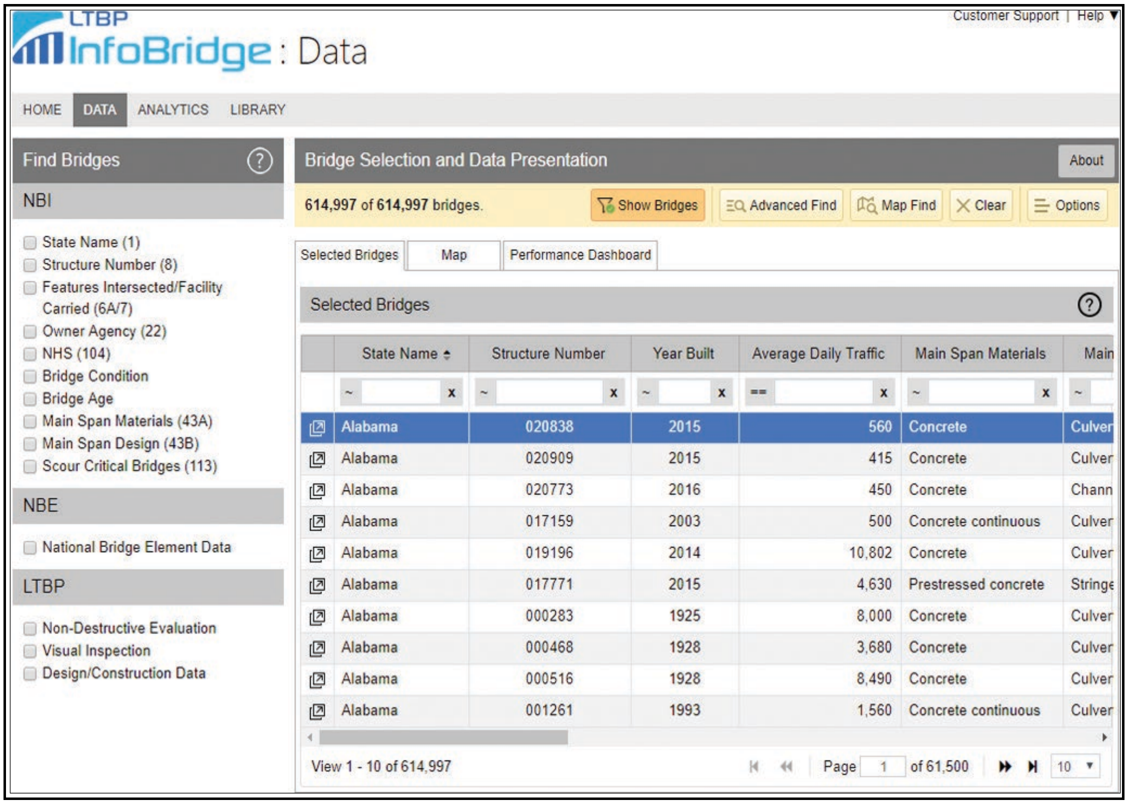


Figure 18.5.4 InfoBridge Find Bridge

The LTBP InfoBridge web portal is a centralized gateway providing efficient and quick access to bridge performance-related data and information. The portal includes multiple tools that facilitate bridge data analytics. It provides for storage, retrieval, dissemination, analysis, and visualization of data collected through state, national, and LTBP Program efforts to provide users with the ability to holistically assess bridge performance on a network or individual bridge basis.

LTBP InfoBridge provides a user-friendly web front-end that includes intuitive tools for finding, viewing, and analyzing bridge performance information to efficiently share data selections and summary reports.

LTBP InfoBridge can be accessed at the following website: https://infobridge.fhwa.dot.gov/.

This page intentionally left blank.

APPENDIX – National Bridge Inspection Standards

23 CFR Part 650 - BRIDGES, STRUCTURES, AND HYDRAULICS

Subpart C - National Bridge Inspection Standards (NBIS)

Authority: 23 U.S.C. 119, 144, and 315.

Effective Date: 06/06/2022

§ 650.301 Purpose.

This subpart sets the national minimum standards for the proper safety inspection and evaluation of all highway bridges in accordance with 23 144(h) and the requirements for preparing and maintaining an inventory in accordance with 23 U.S.C. 144(b).

§ 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, on and off Federal-aid highways, including tribally-owned and federally-owned bridges, private bridges that are connected to a public road on both ends of the bridge, temporary bridges, and bridges under construction with portions open to traffic.

§ 650.305 Definitions.

The following terms used in this subpart are defined as follows:

AASHTO Manual. The term ''AASHTO Manual'' means the American Association of State Highway and Transportation Officials (AASHTO) ''Manual for Bridge Evaluation'', including Interim Revisions, excluding the 3rd paragraph in Article 6B.7.1, incorporated by reference in § 650.317.

Attribute. Characteristic of the design, loading, conditions, and environment that affect the reliability of a bridge or bridge member.

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 under copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it includes 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 this subpart, in either a field inspection, supervisory, or management role. Some of the experience may come from relevant bridge design, bridge load rating, bridge construction, and bridge maintenance experience provided it develops the skills necessary to properly perform a NBIS bridge inspection.

Bridge inspection refresher training. The National Highway Institute 1 (NHI) ‘‘Bridge Inspection Refresher Training Course’’ or other State, federally, or tribally developed instruction aimed to improve quality of inspections, introduce new techniques, and maintain consistency in the inspection program.

Bridge Inspector's Reference Manual or the BIRM. A comprehensive FHWA manual on procedures and techniques for inspecting and evaluating a variety of in-service highway bridges. This manual is available at the following URL: *www.fhwa.dot.gov/bridge/ nbis.cfm.* This manual may be purchased from the Government Publishing Office, Washington, DC 20402 and from National Technical Information Service, Springfield, VA 22161.

Complex feature. Bridge component(s) or member(s) with advanced or unique structural members or operational characteristics, construction methods, and/or requiring specific inspection procedures. This includes mechanical and electrical elements of moveable spans and cable- related members of suspension and cable-stayed superstructures.

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 BIRM for the recommended material to be covered in a comprehensive training course).

Consequence. A measure of impacts to structural safety and serviceability in a hypothetical scenario where a deterioration mode progresses to the point of requiring immediate action. This may include costs to restore the bridge to safe operating condition or other costs.

Critical finding. A structural or safety related deficiency that requires immediate action to ensure public safety.

Damage inspection. An unscheduled inspection to assess structural damage resulting from environmental factors or human actions.

Deterioration mode. Typical deterioration or damage affecting the condition of a bridge member that may affect the structural safety or serviceability of the bridge.

Element level bridge inspection data. Quantitative condition assessment data, collected during bridge inspections, that indicates the severity and extent of defects in bridge elements.

End-of-course assessment. A comprehensive examination given to students after the completion of the delivery of a training course.

Hands-on inspection. Inspection within arm's length of the member. Inspection uses visual techniques that may be supplemented by nondestructive evaluation techniques.

Highway. The term ‘‘highway’’ is defined in 23 U.S.C. 101.

In-depth inspection. A close-up, detailed inspection of one or more bridge members located above or below water, using visual or nondestructive evaluation techniques as required to identify any deficiencies not readily detectable using routine inspection procedures. Hands-on inspection may be necessary at some locations. In-depth inspections may occur more or less frequently than routine inspections, as outlined in bridge specific inspection procedures.

Initial inspection. The first inspection of a new, replaced, or rehabilitated bridge. This inspection serves to record required bridge inventory data, establish baseline conditions, and establish the intervals for other inspection types.

Inspection date. The date on which the field portion of the bridge inspection is completed.

Inspection due date. The last inspection date plus the current inspection interval.

Inspection report. The document which summarizes the bridge inspection findings, recommendations, and identifies the team leader responsible for the inspection and report.

Internal redundancy. A redundancy that exists within a primary member cross-section without load path redundancy, such that fracture of one component will not propagate through the entire member, is discoverable by the applicable inspection procedures, and will not cause a portion of or the entire bridge to collapse.

Inventory data. All data reported to the National Bridge Inventory (NBI) in accordance with the § 650.315.

Legal load. The maximum load for each vehicle configuration, including the weight of the vehicle and its payload, permitted by law for the State in which the bridge is located.

Legal load rating. The maximum permissible legal load to which the structure may be subjected with the unlimited numbers of passages over the duration of a specified bridge evaluation period. Legal load rating is a term used in Load and Resistance Factor Rating method.

Load path redundancy. A redundancy that exists based on the number of primary load-carrying members between points of support, such that fracture of the cross section at one location of a member will not cause a portion of or the entire bridge to collapse.

Load posting. Regulatory signs installed in accordance with 23 CFR 655.601 and State or local law which represent the maximum vehicular live load which the bridge may safely carry.

Load rating. The analysis to determine the safe vehicular live load carrying capacity of a bridge using bridge plans and supplemented by measurements and other information gathered from an inspection.

Nationally certified bridge inspector. An individual meeting the team leader requirements of § 650.309(b).

Nonredundant Steel Tension Member (NSTM). A primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse.

NSTM inspection. A hands-on inspection of a nonredundant steel tension member.

NSTM inspection training. Training that covers all aspects of NSTM inspections to relate conditions observed on a bridge to established criteria.

Operating rating. The maximum permissible live load to which the structure may be subjected for the load configuration used in the load rating. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge. Operating rating is a term used in either the Allowable Stress or Load Factor Rating method.

Private bridge. A bridge open to public travel and not owned by a public authority as defined in 23 U.S.C. 101.

Procedures. Written documentation of policies, methods, considerations, criteria, and other conditions that direct the actions of personnel so that a desired end result is achieved consistently.

Probability. Extent to which an event is likely to occur during a given interval. This may be based on the frequency of events, such as in the quantitative probability of failure, or on degree of belief or expectation. Degrees of belief about probability can be chosen using qualitative scales, ranks, or categories such as, remote, low, moderate, or high.

Professional engineer (PE). An individual, who has fulfilled education and experience requirements and passed examinations for professional engineering and/or structural engineering license that, under State licensure laws, permits the individual to offer engineering services within areas of expertise directly to the public.

Program manager. The individual in charge of the program, that has been assigned the duties and responsibilities for bridge inspection, reporting, and inventory, and has the overall responsibility to ensure the program conforms with the requirements of this subpart. 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.

Quality assurance (QA). The use of sampling and other measures to assure the adequacy of QC 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.

Rehabilitation. The major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects.

Risk. The exposure to the possibility of structural safety or serviceability loss during the interval between inspections. It is the combination of the probability of an event and its consequence.

Risk assessment panel (RAP). A group of well experienced panel members that performs a rigorous assessment of risk to establish policy for bridge inspection intervals.

Routine inspection. Regularly scheduled comprehensive inspection consisting of observations and measurements needed to determine the physical and functional condition of the bridge and identify changes from previously recorded conditions.

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.

Safe load capacity. A live load that can safely utilize a bridge repeatedly over the duration of a specified inspection interval.

Scour. Erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges.

Scour appraisal. A risk-based and data-driven determination of a bridge's vulnerability to scour, resulting from the least stable result of scour that is either observed, or estimated through a scour evaluation or a scour assessment.

Scour assessment. The determination of an existing bridge's vulnerability to scour which considers stream stability and scour potential.

Scour critical bridge. A bridge with a foundation member that is unstable, or may become unstable, as determined by the scour appraisal.

Scour evaluation. The application of hydraulic analysis to estimate scour depths and determine bridge and substructure stability considering potential scour.

Scour plan of action (POA). Procedures for bridge inspectors and engineers in managing each bridge determined to be scour critical or that has unknown foundations.

Service inspection. An inspection to identify major deficiencies and safety issues, performed by personnel with general knowledge of bridge maintenance or bridge inspection.

Special inspection. An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency, or to monitor special details or unusual characteristics of a bridge that does not necessarily have defects.

Special 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 and routine permit loads, typically authorized for single or limited trips.

State transportation department. The term ''State transportation department'' is defined in 23 U.S.C. 101.

System redundancy. A redundancy that exists in a bridge system without load path redundancy, such that fracture of the cross section at one location of a primary member will not cause a portion of or the entire bridge to collapse.

Team leader. The on-site, nationally certified bridge inspector in charge of an inspection team and responsible for planning, preparing, performing, and reporting on bridge field inspections.

Temporary bridge. A bridge which is constructed to carry highway traffic until the permanent facility is built, repaired, rehabilitated, or replaced.

Underwater bridge inspection diver. The individual performing the inspection of the underwater portion of the bridge.

Underwater Bridge Inspection Manual. A comprehensive FHWA manual on the procedures and techniques for underwater bridge inspection. This manual is available at the following URL: *www.fhwa.dot.gov/ bridge/nbis.cfm.* This manual may be purchased from the Government Publishing Office, Washington, DC 20402 and from National Technical Information Service, Springfield, VA 22161.

Underwater bridge inspection training. Training that covers all aspects of underwater bridge inspection to relate the conditions of underwater bridge members to established criteria (see Underwater Bridge Inspection Manual and the BIRM section on underwater inspection for the recommended material to be covered in an underwater 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 or by wading or probing, and generally requiring diving or other appropriate techniques.

Unknown Foundations. Foundations of bridges over waterways where complete details are unknown because either the foundation type and depth are unknown, or the foundation type is known, but its depth is unknown, and therefore cannot be appraised for scour vulnerability.

§ 650.307 Bridge inspection organization responsibilities.

(a) Each State transportation department must perform, or cause to be performed, the proper inspection and evaluation of all highway bridges that are fully or partially located within the State's boundaries, except for bridges that are owned by Federal agencies or Tribal governments.

(b) Each Federal agency must perform, or cause to be performed, the proper inspection and evaluation of all highway bridges that are fully or partially located within the respective Federal agency's responsibility or jurisdiction.

(c) Each Tribal government, in consultation with the Bureau of Indian Affairs (BIA) or FHWA, must perform, or cause to be performed, the proper inspection and evaluation of all highway bridges that are fully or partially located within the respective Tribal government's responsibility or jurisdiction.

(d) Where a bridge crosses a border between a State transportation department, Federal agency, or Tribal government jurisdiction, all entities must determine through a joint written agreement the responsibilities of each entity for that bridge under this subpart, including the designated lead State for reporting NBI data.

(e) Each State transportation department, Federal agency, and Tribal government must include a bridge inspection organization that is responsible for the following:

(1) Developing and implementing written Statewide, Federal agencywide, or Tribal governmentwide bridge inspection policies and procedures;

(2) Maintaining a registry of nationally certified bridge inspectors that are performing the duties of a team leader in their State or Federal agency or Tribal government that includes, at a minimum, a method to positively identify each inspector, inspector's qualification records, inspector's current contact information, and detailed information about any adverse action that may affect the good standing of the inspector;

(3) Documenting the criteria for inspection intervals for the inspection types identified in these standards;

(4) Documenting the roles and responsibilities of personnel involved in the bridge inspection program;

(5) Managing bridge inspection reports and files;

(6) Performing quality control and quality assurance activities;

(7) Preparing, maintaining, and reporting bridge inventory data;

(8) Producing valid load ratings and when required, implementing load posting or other restrictions;

(9) Managing the activities and corrective actions taken in response to a critical finding;

(10) Managing scour appraisals and scour plans of action; and

(11) Managing other requirements of these standards.

(f) Functions identified in paragraphs (e)(3) through (11) of this section may be delegated to other individuals, agencies, or entities. The delegated roles and functions of all individuals, agencies, and entities involved must be documented by the responsible State transportation department, Federal agency, or Tribal government. Except as provided below, such delegation does not relieve the State transportation department, Federal agency, or Tribal government of any of its responsibilities under this subpart. A Tribal government may, with BIA's or FHWA's concurrence via a formal written agreement, delegate its functions and responsibilities under this subpart to the BIA or FHWA.

(g) Each State transportation department, Federal agency, or Tribal government bridge inspection organization must have a program manager with the qualifications defined in § 650.309(a). An employee of the BIA or FHWA having the qualification of a program manager as defined in § 650.309(a) may serve as the program manager for a Tribal government if the Tribal government delegates this responsibility to the BIA or FHWA in accordance with paragraph (f) of this section.

§ 650.309 Qualifications of personnel.

(a) A program manager must, at a minimum:

(1) Be a registered Professional Engineer, or have 10 years of bridge inspection experience;

(2) Complete an FHWA-approved comprehensive bridge inspection training course as described in paragraph (h) of this section and score 70 percent or greater on an end-of- course assessment (completion of FHWA-approved comprehensive bridge inspection training under FHWA regulations in this subpart in effect before June 6, 2022, satisfies the intent of the requirement in this paragraph (a));

(3) Complete a cumulative total of 18 hours of FHWA-approved bridge inspection refresher training over each 60 month period;

(4) Maintain documentation supporting the satisfaction of paragraphs (a)(1) through (3) of this section; and

(5) Satisfy the requirements of this paragraph (a) within 24 months from June 6, 2022, if serving as a program manager who was qualified under prior FHWA regulations in this subpart.

(b) A team leader must, at a minimum:

(1) Meet one of the four qualifications listed in paragraphs (b)(1)(i) through (iv) of this section:

(i) Be a registered Professional Engineer and have 6 months of bridge inspection experience;

(ii) Have 5 years of bridge inspection experience;

(iii) Have all of the following:

(A) A bachelor'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; and

(B) Successfully passed the National Council of Examiners for Engineering and Surveying Fundamentals of Engineering examination; and

(C) Two (2) years of bridge inspection experience; or

(iv) Have all of the following:

(A) 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; and

(B) Four (4) years of bridge inspection experience;

(2) Complete an FHWA-approved comprehensive bridge inspection training course as described in paragraph (h) of this section and score 70 percent or greater on an end-of- course assessment (completion of FHWA-approved comprehensive bridge inspection training under FHWA regulations in this subpart in effect before June 6, 2022, satisfies the intent of the requirement in this paragraph (b));

(3) Complete a cumulative total of 18 hours of FHWA-approved bridge inspection refresher training over each 60 month period;

(4) Provide documentation supporting the satisfaction of paragraphs (b)(1) through (3) of this section to the program manager of each State transportation department, Federal agency, or Tribal government for which they are performing bridge inspections; and

(5) Satisfy the requirements of this paragraph (b) within 24 months from June 6, 2022, if serving as a team leader who was qualified under prior FHWA regulations in this subpart.

(c) Team leaders on NSTM inspections must, at a minimum:

(1) Meet the requirements in paragraph (b) of this section;

(2) Complete an FHWA-approved training course on the inspection of NSTMs as defined in paragraph (h) of this section and score 70 percent or greater on an end-of-course assessment (completion of FHWA-approved NSTM inspection training prior to June 6, 2022, satisfies the intent of the requirement in this paragraph (c)); and

(3) Satisfy the requirements of this paragraph (c) within 24 months from June 6, 2022.

(d) Load ratings must be performed by, or under the direct supervision of, a registered professional engineer.

(e) An Underwater Bridge Inspection Diver must complete FHWA-approved underwater bridge inspection training as described in paragraph (h) of this section and score 70 percent or greater on an end-of-course assessment (completion of FHWA-approved comprehensive bridge inspection training or FHWA-approved underwater bridge inspection training under FHWA regulations in this subpart in effect before June 6, 2022, satisfies the intent of the requirement in this paragraph (e)).

(f) State transportation departments, Federal agencies, and Tribal governments must establish documented personnel qualifications for Damage and Special Inspection types.

(g) State transportation departments, Federal agencies, and Tribal governments that establish risk-based routine inspection intervals that exceed 48 months under § 650.311(a)(2) must establish documented personnel qualifications for the Service Inspection type.

(h) The following are considered acceptable bridge inspection training:

(1) *National Highway Institute training*. Acceptable NHI courses include:

(i) Comprehensive bridge inspection training, which must include topics of importance to bridge inspection; bridge mechanics and terminology; personal and public safety issues associated with bridge inspections; properties and deficiencies of concrete, steel, timber, and masonry; inspection equipment needs for various types of bridges and site conditions; inspection procedures, evaluations, documentation, data collection, and critical findings for bridge decks, superstructures, substructures, culverts, waterways (including underwater members), joints, bearings, drainage systems, lighting, signs, and traffic safety features; nondestructive evaluation techniques; load path redundancy and fatigue concepts; and practical applications of the concepts listed in this paragraph (h)(1)(i);

(ii) Bridge inspection refresher training, which must include topics on documentation of inspections, commonly miscoded items, recognition of critical inspection findings, recent events impacting bridge inspections, and quality assurance activities;

(iii) Underwater bridge inspection training, which must include topics on the need for and benefits of underwater bridge inspections; typical defects and deterioration in underwater members; inspection equipment needs for various types of bridges and site conditions; inspection planning and hazard analysis; and underwater inspection procedures, evaluations, documentation, data collection, and critical findings; and

(iv) NSTM inspection training, which must include topics on the identification of NSTMs and related problematic structural details; the recognition of areas most susceptible to fatigue and fracture; the evaluation and recording of defects on NSTMs; and the application of nondestructive evaluation techniques.

(2) *FHWA approval of alternate training.* A State transportation department, Federal agency, or Tribal government may submit to FHWA a training course as an alternate to any of the NHI courses listed in paragraph (h)(1) of this section. An alternate must include all the topics described in paragraph (h)(1) and be consistent with the related content. FHWA must approve alternate course materials and end-of-course assessments for national consistency and certification purposes. Alternate training courses must be reviewed by the program manager every 5 years to ensure the material is current. Updates to approved course materials and end-of-course assessments must be resubmitted to FHWA for approval.

(3) *FHWA-approved alternate training under prior regulations.* Agencies that have alternate training courses approved by FHWA prior to June 6, 2022, have 24 months to review and update training materials to satisfy requirements as defined in § 650.305 and paragraph (h)(1) of this section and resubmit to FHWA for approval.

§ 650.311 Inspection interval.

(a) Routine inspections. Each bridge must be inspected at regular intervals not to exceed the interval established using one of the risk-based methods outlined in paragraph (a)(1) or (2) of this section.

(1) *Method 1.* Inspection intervals are determined by a simplified assessment of risk to classify each bridge into one of three categories with an inspection interval as described below.

(i) Regular intervals. Each bridge must be inspected at regular intervals not to exceed 24 months, except as required in paragraph (a)(1)(ii) of this section and allowed in paragraphs (a)(1)(iii) of this section.

(ii) Reduced intervals.

(A) State transportation departments, Federal agencies, or Tribal governments must develop and document criteria used to determine when intervals must be reduced below 24 months. Factors to consider include structure type, design, materials, age, condition ratings, scour, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle impact damage, loads and safe load capacity, and other known deficiencies.

(B) Certain bridges meeting any of the following criteria as recorded in the National Bridge Inventory (NBI) (see § 650.315) must be inspected at intervals not to exceed 12 months:

(1) One or more of the deck, superstructure, or substructure, or culvert components is rated in serious or worse condition, as recorded by the Deck, Superstructure, or Substructure Condition Rating items, or the Culvert Condition Rating item, coded three (3) or less; or

(2) The observed scour condition is rated serious or worse, as recorded by the Scour Condition Rating item coded three (3) or less.

(C) Where condition ratings are coded three (3) or less due to localized deficiencies, a special inspection limited to those deficiencies, as described in § 650.313(h), can be used to meet this requirement in lieu of a routine inspection. In such cases, a complete routine inspection must be conducted in accordance with paragraph (a)(1)(i) of this section.

(iii) Extended intervals.

(A) Certain bridges meeting all of the following criteria as recorded in the NBI (see § 650.315) may be inspected at intervals not to exceed 48 months:

(1) The deck, superstructure, and substructure, or culvert, components are all rated in satisfactory or better condition, as recorded by the Deck, Superstructure, and

Substructure Condition Rating items, or the Culvert Condition Rating item coded six (6) or greater;

(2) The channel and channel protection are rated in satisfactory or better condition, as recorded by the Channel Condition and Channel Protection Condition items coded six (6) or greater;

(3) The inventory rating is greater than or equal to the standard AASHTO HS-20 or HL-93 loading and routine permit loads are not restricted or not carried/issued, as recorded by the Inventory Load Rating Factor item coded greater than or equal to 1.0 and the Routine Permit Loads item coded A or N;

(4) A steel bridge does not have Category E or E' fatigue details, as recorded by the Fatigue Details item coded N;

(5) All roadway vertical clearances are greater than or equal to 14'-0", as recorded in the Highway Minimum Vertical Clearance item;

(6) All superstructure materials limited to concrete and steel and all superstructure types limited to certain arches, box girders/beams, frames, girders/beams, slabs, and culverts, as recorded by the Span Material items coded C01–C05 or S01–S05, and the Span Type items coded A01, B02-B03, F01-F02, G01-G08, S01-S02, or P01-P02; and

(7) Stable for potential scour and observed scour condition is rated satisfactory or better, as recorded by the Scour Vulnerability item coded A or B and the Scour Condition Rating item coded six (6) or greater.

(B) State transportation departments, Federal agencies, or Tribal governments that implement paragraph (a)(1)(iii)(A) of this section must develop and document an extended interval policy and must notify FHWA in writing prior to implementation. Factors to consider include structure type, design, materials, age, condition ratings, scour, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle impact damage, loads and safe load capacity, and other known deficiencies.

(2) *Method 2.* Inspection intervals are determined by a more rigorous assessment of risk to classify each bridge, or a group of bridges, into one of four categories, with inspection intervals not to exceed 12, 24, 48, or 72 months. The risk assessment process must be developed by a Risk Assessment Panel (RAP) and documented as a formal policy. The RAP must be comprised of not less than four people, at least two of which are professional engineers, with collective knowledge in bridge design, evaluation, inspection, maintenance, materials, and construction, and include the NBIS program manager. The policy and criteria which establishes intervals, including subsequent changes, must be submitted by the State transportation department, Federal agency, or Tribal government for FHWA approval. The request must include the items in paragraphs (a)(2)(i) through (vi) of this section:

(i) Endorsement from a RAP, which must be used to develop a formal policy.

(ii) Definitions for risk factors, categories, and the probability and consequence levels that are used to define the risk for each bridge to be assessed.

(iii) Deterioration modes and attributes that are used in classifying probability and consequence levels, depending on their relevance to the bridge being considered. A system of screening, scoring, and thresholds are defined by the RAP to assess the risks. Scoring is based on prioritizing attributes and their relative influence on deterioration modes.

(A) A set of screening criteria must be used to determine how a bridge should be considered in the assessment and to establish maximum inspection intervals. The screening criteria must include:

(1) Requirements for flexure and shear cracking in concrete primary load members;

(2) Requirements for fatigue cracking and corrosion in steel primary load members;

(3) Requirements for other details, loadings, conditions, and inspection findings that are likely to affect the safety or serviceability of the bridge or its members;

(4) Bridges classified as in poor condition cannot have an inspection interval greater than 24 months; and

(5) Bridges classified as in fair condition cannot have an inspection interval greater than 48 months.

(B) The attributes in each assessment must include material properties, loads and safe load capacity, and condition.

(C) The deterioration modes in each assessment must include:

(1) For steel members: Section loss, fatigue, and fracture;

(2) For concrete members: Flexural cracking, shear cracking, and reinforcing and prestressing steel corrosion;

(3) For superstructure members: Settlement, rotation, overload, and vehicle/vessel impact; and

(4) For substructure members: Settlement, rotation, and scour.

(D) A set of criteria to assess risk for each bridge member in terms of probability and consequence of structural safety or serviceability loss in the time between inspections.

(iv) A set of risk assessment criteria, written in standard logical format amenable for computer programming.

(v) Supplemental inspection procedures and data collection that are aligned with the level of inspection required to obtain the data to apply the criteria.

(vi) A list classifying each bridge into one of four risk categories with a routine inspection interval not to exceed 12, 24, 48, or 72 months.

(3) Service inspection. A service inspection must be performed during the month midway between routine inspections when a risk-based, routine inspection interval exceeds 48 months.

(4) Additional routine inspection interval eligibility. Any new, rehabilitated, or structurally modified bridge must receive an initial inspection, be in service for 24 months, and receive its next routine inspection before being eligible for inspection intervals greater than 24 months.

(b) *Underwater inspections.* Each bridge must be inspected at regular intervals not to exceed the interval established using one of the risk-based methods outlined in paragraph (b)(1) or (2) of this section.

(1) *Method 1.* Inspection intervals are determined by a simplified assessment of risk to classify each bridge into one of three categories for an underwater inspection interval as described in this section.

(i) Regular intervals. Each bridge must be inspected at regular intervals not to exceed 60 months, except as required in paragraph (b)(1)(ii) of this section and allowed in paragraph (b)(1)(iii) of this section.

(ii) Reduced intervals.

(A) State transportation departments, Federal agencies, or Tribal governments must develop and document criteria used to determine when intervals must be reduced below 60 months. Factors to consider include structure type, design, materials, age, condition ratings, scour, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle/vessel impact damage, loads and safe load capacity, and other known deficiencies.

(B) Certain bridges meeting at least any of the following criteria as recorded in the NBI (see § 650.315) must be inspected at intervals not to exceed 24 months:

(1) The underwater portions of the bridge are in serious or worse condition, as recorded by the Underwater Inspection Condition item coded three (3) or less;

(2) The channel or channel protection is in serious or worse condition, as recorded by the Channel Condition and Channel Protection Condition items coded three (3) or less; or

(3) The observed scour condition is three (3) or less, as recorded by the Scour Condition Rating item.

(C) Where condition ratings are coded three (3) or less due to localized deficiencies, a special inspection of the underwater portions of the bridge limited to those deficiencies, as described in § 650.313(h), can be used to meet this requirement in lieu of a complete underwater inspection. In such cases, a complete underwater inspection must be conducted in accordance with paragraph (b)(1)(i) of this section.

(iii) Extended intervals.

(A) Certain bridges meeting all of the following criteria as recorded in the NBI (see § 650.315) may be inspected at intervals not to exceed 72 months:

(1) The underwater portions of the bridge are in satisfactory or better condition, as recorded by the Underwater Inspection Condition item coded six (6) or greater;

(2) The channel and channel protection are in satisfactory or better condition, as indicated by the Channel Condition and Channel Protection Condition items coded six (6) or greater;

(3) Stable for potential scour, Scour Vulnerability item coded A or B, and Scour Condition Rating item is satisfactory or better, coded six (6) or greater.

(B) State transportation departments, Federal agencies, or Tribal governments that implement paragraph (b)(1)(iii)(A) of this section must develop and document an underwater extended interval policy and must notify FHWA in writing prior to implementation. Factors to consider include structure type, design, materials, age, condition ratings, scour, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle/ vessel impact damage, loads and safe load capacity, and other known deficiencies.

(2) *Method 2.* Inspection intervals are determined by a more rigorous assessment of risk. The policy and criteria which establishes intervals, including subsequent changes, must be submitted by the State transportation department, Federal agency, or Tribal government for FHWA approval. The process and criteria must be similar to that outlined in paragraph (a)(2) of this section except that each bridge must be classified into one of three risk categories with an underwater inspection interval not to exceed 24, 60, and 72 months.

(c) *NSTM inspections.* NSTMs must be inspected at regular intervals not to exceed the interval established using one of the risk-based methods outlined in paragraph (c)(1) or (2) of this section.

(1) *Method 1.* Inspection intervals are determined by a simplified assessment of risk to classify each bridge into one of three risk categories with an interval not to exceed 12, 24, or 48 months.

(i) Regular intervals. Each NSTM must be inspected at intervals not to exceed 24 months except as required in paragraph (c)(1)(ii) of this section and allowed in paragraph (c)(1)(iii) of this section.

(ii) Reduced intervals.

(A) State transportation departments, Federal agencies, or Tribal governments must develop and document criteria to determine when intervals must be reduced below 24 months. Factors to consider include structure type, design, materials, age, condition, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle impact damage, loads and safe load capacity, and other known deficiencies.

(B) Certain NSTMs meeting the following criteria as recorded in the NBI (see § 650.315) must be inspected at intervals not to exceed 12 months:

(1) The NSTMs are rated in poor or worse condition, as recorded by the NSTM Inspection Condition item, coded 4 or less; or

(2) [Reserved].

(iii) Extended intervals.

(A) Certain NSTMs meeting all of the following criteria may be inspected at intervals not to exceed 48 months:

(1) Bridge was constructed after 1978 as recorded in the NBI (see § 650.315) Year Built item and fabricated in accordance with a fracture control plan;

(2) All NSTMs have no fatigue details with finite life;

(3) All NSTMs have no history of fatigue cracks;

(4) All NSTMs are rated in satisfactory or better condition, as recorded in the NBI (see § 650.315) by the NSTM Inspection Condition item, coded 6 or greater; and

(5) The bridge's inventory rating is greater than or equal to the standard AASHTO HS-20 or HL-93 loading and routine permit loads are not restricted or not carried/issued, as recorded in the NBI (see § 650.315) by the Inventory Load Rating Factor item coded greater than or equal to 1.0 and the Routine Permit Loads item coded A or N;

(6) All NSTMs do not include pin and hanger assemblies.

(B) State transportation departments, Federal agencies, or Tribal governments that implement paragraph (c)(1)(iii)(A) of this section must develop and document an extended interval policy, and notify FHWA in writing prior to implementation. Factors to consider include structure type, design, materials, age, condition, environment, annual average daily traffic and annual average daily truck traffic, history of vehicle impact damage, loads and safe load capacity, and other known deficiencies.

(2) *Method 2.* Inspection intervals are determined by a more rigorous assessment of risk. The policy and criteria which establishes intervals, including subsequent changes must be submitted by the State transportation department, Federal agency, or Tribal government for FHWA approval. The process and criteria must be similar to that outlined in paragraph (a)(2) of this section except that each bridge must be classified into one of three risk categories with a NSTM inspection interval not to exceed 12, 24, or 48 months.

(d) *Damage, in-depth, and special inspections.* A State transportation department, Federal agency, or Tribal government must document the criteria to determine the level and interval for these inspections in its bridge inspection policies and procedures.

(e) *Bridge inspection interval tolerance.*

(1) The acceptable tolerance for intervals of less than 24 months for the next inspection is up to two (2) months after the month in which the inspection was due.

(2) The acceptable tolerance for intervals of 24 months or greater for the next inspection is up to three (3) months after the month in which the inspection was due.

(3) Exceptions to the inspection interval tolerance due to rare and unusual circumstances must be approved by FHWA in advance of the inspection due date plus the tolerance in paragraphs (e)(1) and (2) of this section.

(f) *Next inspection.* Establish the next inspection interval for each inspection type based on results of the inspection and requirements of this section.

(g) *Implementation.*

(1) The requirements of paragraphs (a)(1)(ii), (b)(1)(ii), and (c)(1)(ii) of this section must be satisfied within 24 months from June 6, 2022.

(2) Prior FHWA approved extended inspection interval policies will be rescinded 24 months after June 6, 2022.

§ 650.313 Inspection procedures.

(a) *General.* Inspect each bridge to determine condition, identify deficiencies, and document results in an inspection report in accordance with the inspection procedures in Section 4.2, AASHTO Manual (incorporated by reference, see § 650.317). Special equipment or techniques, and/or traffic control are necessary for inspections in circumstances where their use provide the only practical means of accessing and/or determining the condition of the bridge. The equipment may include advanced technologies listed in the BIRM.

(b) *Initial inspection.* Perform an initial inspection in accordance with Section 4.2, AASHTO Manual (incorporated by reference, see § 650.317) for each new, replaced, rehabilitated, and temporary bridge as soon as practical, but within 3 months of the bridge opening to traffic.

(c) *Routine inspection.* Perform a routine inspection in accordance with Section 4.2, AASHTO Manual (incorporated by reference, see § 650.317).

(d) *In-depth inspection.* Identify the location of bridge members that need an in-depth inspection and document in the bridge files. Perform in-depth inspections in accordance with the procedures developed in paragraph (g) of this section.

(e) *Underwater inspection.* Identify the locations of underwater portions of the bridge in the bridge files that cannot be inspected using wading and probing during a routine inspection. Perform underwater inspections in accordance with the procedures developed in paragraph (g) of this section. Perform the first underwater inspection for each bridge and for each bridge with portions underwater that have been rehabilitated as soon as practical, but within 12 months of the bridge opening to traffic.

(f) *NSTM inspection.*

(1) Identify the locations of NSTMs in the bridge files.

(i) A State transportation department, Federal agency, or Tribal government may choose to demonstrate a member has system or internal redundancy such that it is not considered an NSTM. The entity may develop and submit a formal request for FHWA approval of procedures using a nationally recognized method to determine that a member has system or internal redundancy. FHWA will review the procedures for approval based upon conformance with the nationally recognized method. The request must include:

(A) Written policy and procedures for determining system or internal redundancy.

(B) Identification of the nationally recognized method used to determine system or internal redundancy. Nationally recognized means developed, endorsed and disseminated by a national organization with affiliates based in two or more States; or currently adopted for use by one or more State governments or by the Federal Government; and is the most current version.

(C) Baseline condition of the bridge(s) to which the policy is being applied.

(D) Description of design and construction details on the member(s) that may affect the system or internal redundancy.

(E) Routine inspection requirements for bridges with system or internally redundant members.

(F) Special inspection requirements for the members with system or internal redundancy.

(G) Evaluation criteria for when members should be reviewed to ensure they still have system and internal redundancy.

(ii) Inspect the bridge using the approved methods outlined in paragraphs (f)(1)(i)(E) and (F) of this section.

(2) Perform hands-on inspections of NSTMs in accordance with the procedures developed in paragraph (g) of this section.

(3) Perform the first NSTM inspection for each bridge and for each bridge with rehabilitated NSTMs as soon as practical, but within 12 months of the bridge opening to traffic.

(g) *NSTM, underwater, in-depth, and complex feature inspection procedures.* Develop and document inspection procedures for bridges which require NSTM, underwater, in-depth, and complex feature inspections in accordance with Section 4.2, AASHTO Manual (incorporated by reference, see § 650.317). State transportation departments, Federal agencies, and Tribal governments can include general procedures applicable to many bridges in their procedures manual. Specific procedures for unique and complex structural features must be developed for each bridge and contained in the bridge file.

(h) *Special inspection.* For special inspections used to monitor conditions as described in paragraphs (a)(1)(ii) and (b)(1)(ii) of this section, develop and document procedures in accordance with Section 4.2, AASHTO Manual (incorporated by reference, see § 650.317).

(i) *Service inspection.* Perform a service inspection when the routine inspection interval is greater than 48 months. Document the inspection date and any required follow up actions in the bridge file.

(j) *Team leader.* Provide at least one team leader at the bridge who meets the minimum qualifications stated in § 650.309 and actively participates in the inspection at all times during each initial, routine, in-depth, NSTM, underwater inspection, and special inspection described in paragraph (h) of this section.

(k) *Load rating.*

(1) Rate each bridge as to its safe load capacity in accordance with the incorporated articles in Sections 6 and 8, AASHTO Manual (incorporated by reference, see § 650.317).

(2) Develop and document procedures for completion of new and updated bridge load ratings. Load ratings must be completed as soon as practical, but no later than 3 months after the initial inspection and when a change is identified that warrants a re-rating such as, but not limited to, changes in condition, reconstruction, new construction, or changes in dead or live loads.

(3) Analyze routine and special permit loads for each bridge that these loads cross to verify the bridge can safely carry the load.

(l) *Load posting.*

(1) Implement load posting or restriction for a bridge in accordance with the incorporated articles in Section 6, AASHTO Manual (incorporated by reference, see § 650.317), when the

maximum unrestricted legal loads or State routine permit loads exceed that allowed under the operating rating, legal load rating, or permit load analysis.

(2) Develop and document procedures for timely load posting based upon the load capacity and characteristics such as annual average daily traffic, annual average daily truck traffic, and loading conditions. Posting shall be made as soon as possible but not later than 30 days after a load rating determines a need for such posting. Implement load posting in accordance with these procedures.

(3) Missing or illegible posting signs shall be corrected as soon as possible but not later than 30 days after inspection or other notification determines a need.

(m) *Closed bridges.* Develop and document criteria for closing a bridge which considers condition and load carrying capacity for each legal vehicle. Bridges that meet the criteria must be closed immediately. Bridges must be closed when the gross live load capacity is less than 3 tons.

(n) *Bridge files.* Prepare and maintain bridge files in accordance with Section 2.2, AASHTO Manual (incorporated by reference, see § 650.317).

(o) *Scour.*

(1) Perform a scour appraisal for all bridges over water, and document the process and results in the bridge file. Re-appraise when necessary to reflect changing scour conditions. Scour appraisal procedures should be consistent with Hydraulic Engineering Circulars (HEC) 18 and 20. Guidance for scour evaluations is located in HEC 18 and 20, and guidance for scour assessment is located in HEC 20.

(2) For bridges which are determined to be scour critical or have unknown foundations, prepare and document a scour POA for deployment of scour countermeasures for known and potential deficiencies, and to address safety concerns. The plan must address a schedule for repairing or installing physical and/or hydraulic scour countermeasures, and/or the use of monitoring as a scour countermeasure. Scour plans of actions should be consistent with HEC 18 and 23.

(3) Execute action in accordance with the plan.

(p) *Quality control and quality assurance.*

(1) Assure systematic QC and QA procedures identified in Section 1.4, AASHTO Manual (incorporated by reference, see § 650.317) are used to maintain a high degree of accuracy and consistency in the inspection program.

(2) Document the extent, interval, and responsible party for the review of inspection teams in the field, inspection reports, NBI data, and computations, including scour appraisal and load ratings. QC and QA reviews are to be performed by personnel other than the individual who completed the original report or calculations.

(3) Perform QC and QA reviews and document the results of the QC and QA process,

including the tracking and completion of actions identified in the procedures.

(4) Address the findings of the QC and QA reviews.

(q) *Critical findings.*

(1) Document procedures to address critical findings in a timely manner. Procedures must:

(i) Define critical findings considering the location and the redundancy of the member affected and the extent and consequence of a deficiency. Deficiencies include, but are not limited to scour, damage, corrosion, section loss, settlement, cracking, deflection, distortion, delamination, loss of bearing, and any condition posing an imminent threat to public safety. At a minimum, include findings which warrant the following:

(A) Full or partial closure of any bridge;

(B) An NSTM to be rated in serious or worse condition, as defined in the NBI (see § 650.315) by the NSTM Inspection item, coded three (3) or less;

(C) A deck, superstructure, substructure, or culvert component to be rated in critical or worse condition, as defined in the NBI (see § 650.315) by the Deck, Superstructure, or Substructure Condition Rating items, or the Culvert Condition Rating item, coded two (2) or less;

(D) The channel condition or scour condition to be rated in critical or worse condition as defined in the NBI (see § 650.315) by the Channel Condition Rating or Scour Condition Rating items, coded critical (2) or less; or

(E) Immediate load restriction or posting, or immediate repair work to a bridge, including shoring, in order to remain open.

(ii) Develop and document timeframes to address critical findings identified in paragraph (q)(1)(i) of this section.

(2) State transportation departments, Federal agencies, and Tribal governments must inform FHWA of all critical findings and actions taken, underway, or planned to resolve critical findings as follows:

(i) Notify FHWA within 24 hours of discovery of each critical finding on the National Highway System (NHS) as identified in paragraphs (q)(1)(i)(A) and (B) of this section;

(ii) Provide monthly, or as requested, a written status report for each critical finding as identified in paragraph (q)(1)(i) of this section until resolved. The report must contain:

(A) Owner;

(B) NBI Structure Number;

(C) Date of finding;

(D) Description and photos (if available) of critical finding;

(E) Description of completed, temporary and/or planned corrective actions to address critical finding;

(F) Status of corrective actions: Active/Completed;

(G) Estimated date of completion if corrective actions are active; and

(H) Date of completion if corrective actions are completed.

(r) *Review of compliance.* Provide information annually or as required in cooperation with any FHWA review of compliance with this subpart.

§ 650.315 Inventory.

(a) Each State transportation department, Federal agency, or Tribal government must prepare and maintain an inventory of all bridges subject to this subpart. Inventory data, as defined in § 650.305, must be collected, updated, and retained by the responsible State transportation department, Federal agency, or Tribal government and submitted to FHWA on an annual basis or whenever requested. For temporary bridges open to traffic greater than 24 months, inventory data must be collected and submitted per this section. Inventory data must include element level bridge inspection data for bridges on the NHS collected in accordance with the ''Manual for Bridge Element Inspection'' (incorporated by reference, see § 650.317). Specifications for collecting and reporting this data are contained in the ''Specifications for the National Bridge Inventory'' (incorporated by reference, see § 650.317).

(b) For all inspection types, enter changes to the inventory data into the State transportation department, Federal agency, or Tribal government inventory within 3 months after the month when the field portion of the inspection is completed.

(c) For modifications to existing bridges that alter previously recorded inventory data and for newly constructed bridges, enter the inventory data into the State transportation department, Federal agency, or Tribal government inventory within 3 months after the month of opening to traffic.

(d) For changes in load restriction or closure status, enter the revised inventory data into the State transportation department, Federal agency, or Tribal government inventory within 3 months after the month the change in load restriction or closure status of the bridge is implemented.

(e) Each State transportation department, Federal agency, or Tribal government must establish and document a process that ensures the time constraint requirements of paragraphs (b) through (d) of this section are fulfilled.

§ 650.317 Incorporation by reference.

Certain material is incorporated by reference (IBR) into this subpart with the approval of the Director of the Federal Register under 5 U.S.C. 552(a) and 1 CFR part 51. All approved material is available for inspection at the U.S. Department of Transportation (DOT) and the National Archives and Records Administration (NARA). Contact DOT at: U.S. Department of Transportation Library, 1200 New Jersey Avenue SE, Washington, DC 20590 in Room W12–300, (800) 853–1351, www.ntl.bts.gov/ntl. For information on the availability of this material at NARA email: fr.inspection@nara.gov or go to: *www.archives.gov/federal-register/cfr/ ibr-locations.html.* The material may be obtained from the following sources:

(a) AASHTO. American Association of State Highway and Transportation Officials, 555 12th Street NW, Suite 1000, Washington, DC 20004; 1–800– 231–3475; https://store.transportation.org.

(1) MBE–3. ''The Manual for Bridge Evaluation,'' Third Edition, 2018; IBR approved for § 650.305 and 650.313.:

(2) MBE–3–I1–OL. The Manual for Bridge Evaluation, 2019 Interim Revisions [to 2018 Third Edition], copyright 2018; IBR approved for § 650.305 and 650.313.

(3) MBE–3–I2. The Manual for Bridge Evaluation, 2020 Interim Revisions [to 2018 Third Edition], copyright 2020; IBR approved for § 650.305 and 650.313.

(4) MBEI–2: Manual for Bridge Element Inspection, Second Edition, 2019, IBR approved for § 650.315.

(b) FHWA. Federal Highway Administration, 1200 New Jersey Avenue SE, Washington, DC 20590: 1– 202–366–4000; *www.fhwa.dot.gov/ bridge/nbi.cfm.*

(1) FHWA–HIF–22–017: Specifications for the National Bridge Inventory, March, 2022, IBR approved for § 650.315.

(2) [Reserved].

This page intentionally left blank.

GLOSSARY

A

AASHTO *MBE* - The term ''AASHTO *MBE*'' means the American Association of State Highway and Transportation Officials (AASHTO) ''Manual for Bridge Evaluation'', including Interim Revisions, excluding the 3rd paragraph in Article 6B.7.1, incorporated by reference in § 650.317.

AASHTO *MBEI* - *AASHTO Manual for Bridge Element Inspection, 2nd Edition* is a reference for standardized element definitions, element quantity calculations, condition state definitions, element feasible actions, and inspection conventions. This manual is used for element descriptions, quantity calculations, and condition state definitions.

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.

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.

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.

Annual Average Daily Traffic (AADT) - the total annual volume of traffic passing a point or segment of a highway in both directions divided by the number of days in a year.

Annual Average Daily Truck Traffic (AADTT) - the total annual volume of truck traffic passing a point or segment of a highway in both directions divided by the number of days in a year.

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.

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.

attribute - a characteristic of the design, loading, conditions, and environment that affect the reliability of a bridge or bridge member.

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

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.

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.

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.

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.**

bay - the area of a bridge floor system between adjacent beams or between adjacent floorbeams.

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 resist.

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.

bedding - the soil or backfill material used to support pipe culverts.

bedrock - the undisturbed rock layer below the surface soil.

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.

bitumen - a black sticky mixture of hydrocarbons obtained from natural deposits or from distilling petroleum; tar.

bolt -a mechanical fastener with machine threads at one end to receive a nut, and an integral head at the other end.

bond - in reinforced concrete, the grip of the concrete on the reinforcing bars, which prevents slippage of the bars relative to the concrete mass.

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 (also known as four-sided frame).

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 under copings of abutments or spring lines of arches, or extreme ends of openings for multiple boxes; it includes 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 - individual parts of a bridge that are subsets of bridge components, inventoried separately as functional groups. Elements inventoried on the bridge include: the total quantity for each element, and the element quantity that exists in each of four condition states reported to the NBI in accordance with 23 U.S.C. 144(d)(2).

bridge inspection experience - active participation in bridge inspections in accordance with the NBIS, in either a field inspection, supervisory, or management role. Some of the experience may come from relevant bridge design, bridge load rating, bridge construction, and bridge maintenance experience provided it develops the skills necessary to properly perform a NBIS bridge inspection.

bridge inspection refresher training - the National Highway Institute (NHI) "Bridge Inspection Refresher Training Course" [1] or other acceptable training as per 23 CFR 650.309(h) including State, federally, or tribally 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: (https://www.nhi.fhwa.dot.gov/home.aspx).

Bridge Inspector's Reference Manual (BIRM) - A comprehensive FHWA manual on procedures and techniques for inspecting and evaluating a variety of in-service highway bridges.

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.

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.

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 tee 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.

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 **cable-stayed bridge and 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 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.

carbon steel - steel (iron with dissolved carbon) owing its properties principally to its carbon content; ordinary, unalloyed steel.

cast-in-place (CIP) - 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.

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 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.

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 recrystallized soluble salts, which causes accelerated corrosion of the steel reinforcement.

chord - a generally horizontal member of a truss.

clearance - the unobstructed vertical or horizontal space provided between two objects.

clear span - the unobstructed space or distance between support elements of a bridge or bridge member.

clip angle - see **connection angle.**

closed median - a median in which the area between the two roadways on the structure is bridged over and is capable of supporting traffic.

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**.

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.

compact section - a structural member in which the entire section reaches the yield point in compression prior to any local buckling failures.

compaction - the process by which a sufficient amount of energy (compressive pressure) is applied to soil or other material to increase its density.

complex feature - bridge component(s) or member(s) with advanced or unique structural elements or operational characteristics, construction methods, and/or requiring specific inspection procedures. This includes mechanical and electrical elements of movable spans and cable-related elements of suspension and cable-stayed superstructures.

component - a general term reserved to define parts of a bridge, such as the 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 BIRM 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 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.

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.

consequence - a measure of impacts to structural safety and serviceability in a hypothetical scenario where a deterioration mode progresses to the point of requiring immediate action. This may include costs to restore the bridge to safe operating condition or other costs.

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 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".

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.

counterfort 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.

counterfort wall - a retaining 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.

critical finding - a structural or safety related deficiency that requires immediate action to ensure public safety.

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.

crushing - occurs perpendicular to the grain, usually at support points.

culvert - a structure comprised of one or more barrels, beneath an embankment and designed structurally to account for soil-structure interaction. These structures are hydraulically and structurally designed to convey water, sediment, debris, and, in many cases, aquatic and terrestrial organisms through roadway embankments. Culvert barrels have many sizes and shapes and have inverts that are either integral or open, i.e. supported by spread or pile-supported footings. Many culverts take advantage of headwater submergence of the inlet to increase hydraulic efficiency and economy.

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.

cyclic stress - stress that varies with the passage of live loads; see **stress range.**

D

damage inspection - 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.**

defect - flaw within a bridge material; with regards to element level evaluation it is the method by which distress is categorized and assigned a condition state.

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 glue laminated 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.

deterioration mode - Typical deterioration or damage affecting the condition of a bridge member that may affect the structural safety or serviceability of the bridge.

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.**

differential settlement - uneven settlement of individual or independent elements of a substructure; tilting in the longitudinal or transverse direction due to deformation or loss of foundation material.

dike - an earthen embankment constructed to retain or redirect water; when used in conjunction with a bridge, it prevents stream erosion and localized scour and/or so directs the stream current such that debris does not accumulate; see **spur.**

discharge - the volume of fluid per unit of time flowing along a pipe or channel.

distributed load - a load uniformly applied along the length of an element or component of a bridge.

diver - see “underwater bridge inspection diver”.

divided highway - A highway with separated roadways for traffic traveling in opposite directions.

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 deck bridge - a bridge consisting of two decks, tiers, or levels. These bridges may incorporate highway lanes on both levels or highway lanes on one level and other transportation modes on the other level.

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.**

drawbridge - a general term applied to a bridge over a navigable body of water having a movable superstructure span of any type.

driver expectation - relates to the likelihood that a driver will respond to common situations in predictable ways that the driver has found successful in the past. A driver's readiness to respond to situations, events, and information in predictable and successful ways.

drop inlet - a type of inlet structure that conveys the water from a higher elevation to a lower outlet elevation smoothly without a free fall at the discharge.

duct - the hollow space where a prestressing tendon is placed in a post-tensioned prestressed concrete girder.

ductile - capable of being molded or shaped without breaking; plastic.

ductile fracture - a fracture characterized by plastic deformation.

ductility - the ability to withstand non-elastic deformation without rupture.

E

efflorescence - a deposit on concrete, brick, stone, or mortar caused by crystallization of carbonates brought to the surface by moisture in the masonry or concrete. Efflorescence is a combination of calcium carbonate leached out of the cement paste and other recrystallized carbonate and chloride compounds.

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.

element level bridge inspection data - Quantitative condition assessment data, collected during bridge inspections, that indicates the severity and extent of defects in bridge elements.

elevation view - a drawing of the side view of a structure.

elliptical 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 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.

engineered wood - products that utilize veneers, plywood, reconstituted wood panel products, or engineered wood assemblies. Some engineered wood products include glued laminated timber, I-joists, and laminated veneer lumber.

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.

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.

exterior girder - an outermost girder supporting the bridge floor.

extrados - the curve defining the exterior (upper) surface of an arch.

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.

feature - highway, railroad, waterway, or other travelway that are above, below, or carried on the bridge.

Federal Information Processing Series (FIPS) - a system of numeric and/or alphabetic coding issued by the National Institute of Standards and Technology (NIST), an agency of the US

Department of Commerce. FIPS codes are assigned for a variety of geographic entities including American Indian and Alaska Native Areas, Hawaiian homelands, congressional districts, counties, county subdivisions, metropolitan areas, places, and states. FIPS codes were discontinued by NIST in 2005, but the Census Bureau continues to maintain and issue codes for the geographic entities covered. (Reference: https://www.fhwa.dot.gov/bridge/nbi.cfm)

Federal lands - lands under the jurisdiction of federal agencies. FHWA's Federal Land Management Agency partners currently include: National Park Service (NPS); USDA Forest Service (Forest Service); U.S. Fish and Wildlife Service (USFWS); Bureau of Indian Affairs (BIA) and Tribal Governments; Bureau of Land Management (BLM); Department of Defense (DOD); U.S. Army Corps Of Engineers (USACE); and Bureau of Reclamation (BOR). (Reference: https://highways.dot.gov/federal-lands/about)

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.

ferry transfer bridge - a bridging structure that enables vehicular movement from a dock or approach roadway to a ferry.

FHWA *SNBI* – *FHWA Specifications for the National Bridge Inventory* is a reference that provides the specifications for reporting data for highway bridges, open to the public, to the FHWA for inclusion in the National Bridge Inventory.

fiber reinforced polymer composite - fiber reinforced polymer composite (FRP) is also known as fiberglass reinforced plastic and is a composite made from glass fiber or carbon fiber reinforcement in a plastic (polymer) matrix. With reinforcement of the plastic matrix, a wide variety of physical strengths and properties can be designed into the material. Additionally, the type and configuration of the reinforcement can be selected, along with the type of polymer and additives within the matrix.

fill - material, usually earth, used to change the surface contour of an area, or to construct an embankment.

filler - a piece used primarily to fill a space beneath a batten, splice plate, gusset, connection angle, stiffener, or other element; also known as filler plate.

filler metal - metal prepared in wire, rod, electrode, or other form to be fused with the structure metal in the formation of a weld.

filler plate - see **filler**.

fillet - a curved portion forming a junction of two surfaces that would otherwise intersect at an angle.

fillet weld - a weld of triangular or fillet shaped cross-section between two pieces at right angles.

filling - see **fill**.

fine aggregate - sand or grit for concrete or mortar that passes a No. 4 sieve (4.75 mm).

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 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.

floating bridge - a bridge supported by floating on pontoons moored to the lakebed or riverbed; a portion may be removable to facilitate navigation.

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.

flux - a material that protects the weld from oxidation during the fusion process.

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**.

formwork - see **forms.**

foundation - the supporting material upon which the substructure portion of a bridge is placed.

foundation failure - failure of a foundation by differential settlement or by shear failure of the soil.

foundation pile - see **pile.**

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.**

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.

frost heave - the upward movement of, or force exerted by, soil due to freezing of retained moisture.

frost line - the depth to which soil may be frozen.

G

gabion - rock filled wire baskets used to retain earth and provide erosion control.

galvanic action - electrical current between two unlike metals.

galvanize - to coat with zinc.

gauge - the distance between parallel lines of rails, rivet holes, etc.; a measure of thickness of sheet metal or wire; also known as gage.

geometry - shape or form; relationship between lines or points.

girder - a horizontal flexural member that is the main or primary support for a structure; any large beam, especially if built up.

girder bridge - a bridge whose superstructure consists of two or more girders supporting a separate floor system as differentiated from a multi-beam bridge or a slab bridge.

girder span - a span in which the major longitudinal supporting members are girders.

glue laminated - a member created by gluing together two or more pieces of lumber.

grade - the fall or rise per unit horizontal length; see **gradient**.

grade crossing - a term applicable to an intersection of two highways, two railroads or a railroad and a highway at a common grade or elevation; now commonly accepted as meaning the last of these combinations.

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.

grid flooring - a steel floor system comprising a lattice pattern that may or may not be filled with concrete.

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.

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.**

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.

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.

hands-on inspection - inspection within arms-length of the member. Inspection uses visual techniques that may be supplemented by nondestructive evaluation techniques.

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.

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 number of various operations involving controlled heating and cooling 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 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. (23 U.S.C. 101(a))

Highway Performance Monitoring System - a national level highway information system that includes data on the extent, condition, performance, use, and operating characteristics of the nation’s highways.
(HPMS Field Manual: https://www.fhwa.dot.gov/policyinformation/hpms/fieldmanual/)

hinge - a point in a structure at which a member is free to rotate.

homogeneous - consisting of parts of the same kind.

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.

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 - human-made or human-placed devices designed to direct streamflow and to protect against lateral migration and scour.

hydraulic review - A review by a person qualified to evaluate the field-observed hydraulic conditions and make a determination of the impacts of the conditions on the performance of the channel, channel protection, or when working with structural staff, determine the scour vulnerability of a bridge member or entire bridge. Hydraulic reviews may include a review of the field inspection notes and photographs, review of as-built plans, scour appraisals, and scour POAs, or performance of a hydraulic analysis as deemed appropriate.

hydraulics - the mechanics of fluids.

hydrology - study of the accumulation and flow of water from watershed areas.

hydroplaning - loss of contact between a tire and the roadway surface when the tire planes or glides on a film of water.

I

I-beam - a structural member with a cross-sectional shape similar to the capital letter "I".

ice guard - see **fender**.

impact - A factor that describes the effect on live load due to dynamic and vibratory effects of a moving load; in bridge design, a load based on a percentage of live load to include dynamic and vibratory effects; in fracture mechanics, a rapidly applied load, such as a collision or explosion.

in-depth inspection - a close-up, detailed inspection of one or more bridge members located above or below water, using visual or nondestructive evaluation techniques as required to identify any deficiencies not readily detectable using routine inspection procedures. Hands-on inspection may be necessary at some locations. In-depth inspections may occur more or less frequently than routine inspections, as outlined in bridge specific inspection procedures.

initial inspection - the first inspection of a new, replaced, or rehabilitated bridge. This inspection serves to record required bridge inventory data, establish baseline conditions, and establish the intervals for other inspection types.

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 date - the date on which the field portion of the bridge inspection is completed.

inspection due date - The last inspection date plus the current inspection interval.

inspection interval - the interval with which the bridge is inspected.

inspection report - the document which summarizes the bridge inspection findings, recommendations, and identifies the team leader responsible for the inspection and report.

Inspector - a member of the bridge safety inspection team; where silent, this typically refers to the team leader.

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.

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 redundancy that exists within a primary member cross-section without load path redundancy, such that fracture of one component will not propagate through the entire member, is discoverable by the applicable inspection procedures, and will not cause a portion of or the entire bridge to collapse.

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 data - all data reported to the National Bridge Inventory (NBI) in accordance with the § 650.315.

inventory item - data contained in the structure file pertaining to bridge identification, bridge material and type, bridge geometry, features, loads, load rating, and posting, inspections, and bridge condition.

inventory rating - the capacity of a bridge to withstand loads under normal service conditions based on 55 percent of yield strength. For LRFR, this is the rating factor for the design load rating at the inventory level of reliability using the HL-93 loading considering all applicable strength and serviceability limit states.

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

jacking - the lifting of elements using a type of jack (e.g., hydraulic), sometimes acts as a temporary support system.

jacket - a protective shell surrounding a pile made of fabric, concrete, or other material.

jersey barrier - a concrete barrier with sloping front face that was developed by the New Jersey Department of Transportation.

joint - in masonry, the space between individual stones or bricks; in concrete, a division in continuity of the concrete; in a truss, point at which members of a truss are joined.

K

keeper plate - a plate, which is connected to a sole plate, designed to prohibit a beam from becoming dislodged from the bearing.

key - a raised portion of concrete on one face of a joint that fits into a depression on the adjacent face.

keystone - the symmetrically shaped, wedge-like stone located in a head ring course at the crown of an arch; the final stone placed, thereby closing the arch.

king post - the vertical member in a "king-post" type truss; also known as king rod.

king post truss - two triangular panels with a common center vertical; the simplest of triangular system trusses.

kip - a kilo pound (1000 lb.); convenient unit for structural calculations.

knee brace - a short member engaging at its ends two other members that are joined to form a right angle or a near-right angle to strengthen and stiffen the connecting joint.

knots - separations of the wood fibers due to the trunk growing around an embedded limb.

L

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.

lane loading - a design loading which represents a line of trucks crossing over a bridge.

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.

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 load for each vehicle configuration, including the weight of the vehicle and its payload, permitted by law for the State in which the bridge is located.

legal load rating - the maximum permissible legal load to which the structure may be subjected with the unlimited numbers of passages over the duration of a specified bridge evaluation period. Legal load rating is a term used in Load and Resistance Factor Rating method.

legally enforceable load posting - Posting of a load restriction sign (or signs) at a bridge in accordance with State law that is legally enforceable by law enforcement personnel.

light-weight concrete - concrete of less than standard unit weight; may be no-fines concrete, aerated concrete, or concrete made with lightweight aggregate.

Linear Referencing System - Provides a geospatial representation of a road network through a set of procedures for determining and retaining a record of specific points along a highway. Typical methods used are mile point, milepost, reference point, or link node. LRS data are required for the annual Highway Performance Monitoring System (HPMS) data submittal from the States to FHWA.

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.

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 path redundancy - a redundancy that exists based on the number of primary load-carrying members between points of support, such that fracture of the cross section at one location of a member will not cause a portion of or the entire bridge to collapse.

load posting - Regulatory signs installed in accordance with the 23 CFR 655.601 and State or local law which represent the maximum vehicular live load which the bridge may safely carry.

load rating - The analysis to determine the safe vehicular live load carrying capacity of a bridge using bridge plans and supplemented by measurements and other information gathered from an 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.

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.

lower chord - the bottom horizontal member of a truss.

luminaire - a lighting fixture.

M

maintenance - basic repairs performed on a facility to keep it at an adequate level of service.

maintenance and protection of traffic (MPT) - the management of vehicular and pedestrian traffic through a construction zone to ensure the safety of the public and the construction workforce; see **traffic protection.**

major rehabilitation - the major work required to restore the structural integrity or serviceability of a bridge as well as work necessary to correct major safety defects.

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 plate - a steel plate placed on the substructure to support a superstructure bearing and to distribute the load to the masonry beneath.

meander - a twisting, winding action from side to side; characterizes the serpentine curvature of a narrow, slow flowing stream in a wide flood plain.

mechanically stabilized earth (MSE) - 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 **reinforced earth.**

median - the portion of a highway separating opposing directions of the traveled way.

member - an individual angle, beam, plate, or built component piece intended ultimately to become an integral part of an assembled frame or structure.

midspan - a reference point halfway between the supports of a beam or span.

mild steel - steel containing from 0.04 to 0.25 percent 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.

minor rehabilitation - the minor work required to preserve or restore the structural integrity of a bridge or serviceability as well as the work necessary to correct minor safety defects.

modular joint - a bridge joint designed to handle large movements consisting of an assembly of several strip or compression seals.

modulus of elasticity (E) – the ratio between the stress applied and the resulting elastic strain; Young's modulus; the stiffness of a material.

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, 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.

multi-level interchange - a multilevel highway intersection or junction of intersecting roads and bridges arranged so that vehicles may move from one road to another without crossing the streams of traffic.

N

nail laminated - a laminated member produced by nailing two or more pieces of timber together face to face.

National Bridge Inspection Standards (NBIS) - Federal regulations establishing national policy regarding bridge inspection organization, bridge inspection interval, inspector qualifications, inventory requirements, report formats, and inspection and rating procedures, as described in 23 CFR 650 Subpart C.

National Bridge Inventory (NBI) - an aggregation of State transportation department, Federal agency and Tribal government bridge and associated highway data maintained by the Federal Highway Administration (FHWA). The NBIS requires each State transportation department, Federal agency, and Tribal government to prepare and maintain a bridge inventory, which must be submitted to the FHWA in accordance with these specifications on an annual basis or whenever requested.

National Highway Freight Network - a national highway freight network established by FHWA to assist States in strategically directing resources toward improved movement of freight on highways. The National Highway Freight Network consists of a Primary Highway Freight System, the portions of the Interstate System not designated as part of the Primary Highway Freight System, and Critical Rural Freight Corridors and Critical Urban Freight Corridors designated by states.
(Reference: https://ops.fhwa.dot.gov/freight/infrastructure/nfn/index.htm).

Nationally Certified Bridge Inspector - an individual meeting the team leader requirements of 23 CFR 650.309(b).

Navigable Waterway - navigable waterways are determined by the Commandant of the United States Coast Guard. Title 33 of the Code of Federal Regulations, Section 2.36, defines navigable waterways as consisting of:

1. Territorial seas of the United States;
2. Internal waters of the United States that are subject to tidal influence; and
3. Internal waters of the United States not subject to tidal influence that:
    a. Are or have been used, or are or have been susceptible for use, by themselves or in connection with other waters, as highways for substantial interstate or foreign commerce, notwithstanding natural or human-made obstructions that require portage, or
    b. A governmental or non-governmental body, having expertise in waterway improvement, determines to be capable of improvement at a reasonable cost (a favorable balance between cost and need) to provide, by themselves or in connection with other waters, highways for substantial interstate or foreign commerce.

necking - the elongation and contraction in area that occurs when a ductile material is stressed.

Neighboring State - the State responsible for reporting an abbreviated bridge record for a border bridge. The Designated Lead State and the Neighboring State are determined through agreement between the two border States.

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.

nondestructive evaluation (NDE) - also referred to as nondestructive testing (NDT); any testing method of checking structural quality of materials that does not damage them.

nonredundant steel tension member (NSTM) - a primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse.

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.

NSTM inspection - a hands-on inspection of a nonredundant steel tension member.

NSTM inspection training - training that covers all aspects of NSTM inspections to relate conditions observed on a bridge to established criteria.

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 maximum permissible live load to which the structure may be subjected for the load configuration used in the load rating. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge. Operating rating is a term generally used in either the Allowable Stress or Load Factor Rating method. For LRFR, this is the rating factor for the design load rating at the operating level of reliability using the HL-93 loading considering all applicable strength and serviceability limit states.

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.**

orthotropic deck - an orthotropic deck consists of a flat, thin steel plate stiffened by a series of closely spaced longitudinal ribs at right angles to the floor beams. The deck acts integrally with the steel superstructure.

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.

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.

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.

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.

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.

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 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 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 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.

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.

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 (PCC) - 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-tensioning - 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.

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 **post-tensioning**, **pre-tensioning.**

pre-tensioning - 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 loads - vertical loading from vehicular traffic and permanent vertical loads on the structure.

primary member - a member in the direct load path of primary loads; considered synonymous with the term "main member."

private bridge - a bridge open to public travel and not owned by a public authority as defined in 23 U.S.C. 101.

probability - extent to which an event is likely to occur during a given interval. This may be based on the frequency of events, such as in the quantitative probability of failure, or on degree of belief or expectation. Degrees of belief about probability can be chosen using qualitative scales, ranks, or categories such as, remote, low, moderate, or high.

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.

procedures - written documentation of policies, methods, considerations, criteria, and other conditions that direct the actions of personnel so that a desired end result is achieved consistently.

Professional Engineer (PE) - an individual, who has fulfilled education and experience requirements and passed examinations for professional engineering and/or structural engineering license that, under State licensure laws, permits the individual to offer engineering services within areas of expertise directly to the public.

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 the duties and responsibilities for bridge inspection, reporting, and inventory, and has the overall responsibility to ensure the program conforms with the requirements of this subpart. The program manager provides overall leadership and is available to inspection team leaders to provide guidance.

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.

Q

quality assurance (QA) - the use of sampling and other measures to assure the adequacy of quality control (QC) 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.

railroad flat car - a salvaged flatbed railroad car used as a bridge superstructure, typically on low-volume roads. This type of bridge often has NSTMs.

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.

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 - the major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects.

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.

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.

replacement - total replacement of a bridge with a new facility constructed in the same general traffic corridor.

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 **mechanically stabilized earth.**

retaining wall - a structure designed to restrain and hold back a mass of earth.

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.

risk - the exposure to the possibility of structural safety or serviceability loss during the interval between inspections. It is the combination of the probability of an event and its consequence.

risk assessment panel (RAP) - a group of well experienced panel members that performs a rigorous assessment of risk to establish policy for bridge inspection intervals.

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 a highway, including shoulders, for vehicular use. A divided highway has two or more roadways.

rocker bearing - a bridge support that accommodates expansion and contraction of the superstructure through a tilting action.

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.

route - A specific road, highway, or travel way open to public travel.

routine inspection - regularly scheduled comprehensive inspection consisting of observations and measurements needed to determine the physical and functional condition of the bridge and identify changes from previously recorded conditions.

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 protection - see **cathodic protection.**

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.

safe load capacity - a live load that can safely utilize a bridge repeatedly over the duration of a specified inspection interval.

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 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.

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 - erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges.

scour appraisal - a risk-based and data-driven determination of a bridge's vulnerability to scour, resulting from the least stable result of scour that is either observed, or estimated through a scour evaluation or a scour assessment.

scour assessment - the determination of an existing bridge's vulnerability to scour which considers stream stability and scour potential.

scour critical bridge - a bridge with a foundation member that is unstable, or may become unstable, as determined by the scour appraisal.

scour evaluation - the application of hydraulic analysis to estimate scour depths and determine bridge and substructure stability considering potential scour.

scour monitoring inspection - an inspection performed during or after a triggering storm event as required by a Scour Plan of Action (POA), by personnel with qualifications required by the agency.

scour plan of action (POA) - procedures for bridge inspectors and engineers in managing each bridge determined to be scour critical or that has unknown foundations.

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 diver.

scupper - an opening in the deck of a bridge to provide means for water accumulated upon the roadway surface to drain.

seat - a base on which an object or member is placed.

secondary loads - loads other than primary loads, such as loads from vehicular braking forces, wind, stream and ice flow, temperature changes, etc.

secondary member - a member to brace or stabilize primary members or resist secondary loads, and is not in the direct load path of primary loads.

section loss - loss of a member's cross-sectional area usually by corrosion or decay.

section view - an internal representation of a structure or a member as if a slice was made through the member.

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 inspection - an inspection to identify major deficiencies and safety issues, performed by personnel with general knowledge of bridge maintenance or bridge inspection.

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 stress - the shear force per unit of cross-sectional area; also referred to as diagonal tensile stress.

shear stud - a type of shear connector in the form of a road with a head that is attached to a beam with an automatic stud-welding gun.

sheet pile cofferdam - a wall-like barrier composed of driven piling constructed to surround the area to be occupied by a structure and permit dewatering of the enclosure so that the excavation may be performed in the open air.

sheet piles - flattened Z-shaped interlocking piles driven into the ground to keep earth or water out of an excavation or to protect an embankment.

sheet piling - a general or collective term used to describe a number of sheet piles installed to form a crib, cofferdam, bulkhead, etc.; also known as sheeting.

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 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.

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 unrestrained supports near its ends.

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.

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.

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.**

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 sawmill.

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 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, or to monitor special details or unusual characteristics of a bridge that does not necessarily have defects.

special 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 and routine permit loads, typically authorized for single or limited trips.

specifications - a detailed description of requirements, materials, tolerances, etc., for construction which are not shown on the drawings; also known as specs.

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.

state - any of the 50 States, the District of Columbia, or Puerto Rico.

state transportation department - that department, commission, board, or official of any State charged by its laws with the responsibility for highway construction. (23 U.S.C. 101(a))

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**.

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.

strain - the change in length of a body produced by the application of external forces, measured in units of length; this is the proportional relation of the amount of change in length divided by the original length.

strand - a number of wires grouped together usually by twisting.

Strategic Highway Network (STRAHNET) - a network of highways which are important to the United States' strategic defense policy and which provide defense access, continuity, and emergency capabilities for defense purposes.

Strategic Highway Network (STRAHNET) Connectors - Highways which provide access between major military installations and highways which are part of the Strategic Highway Network.

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 reversal - change of stress type from tension (+) to compression (−) or vice versa.

stress-laminated timber - consists of multiple planks mechanically fastened 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 review - A review by a person qualified to evaluate the field-observed conditions and make a determination of the impacts of the conditions on the performance of the bridge member or entire bridge. Structural reviews may include a review of the field inspection notes and photographs, review of as-built plans, or analysis as deemed appropriate.

structural shapes - the various types of rolled iron and steel having flat, round, angle, channel, “I”, “H”, “Z” and other cross-sectional shapes adapted to heavy construction.

structural stability - the ability of a structure to maintain its normal configuration, not collapse or tip in any way, under existing and expected loads.

structural tee - a tee-shaped rolled member formed by cutting a wide flange longitudinally along the centerline of web.

structure - something, such as a bridge, that is designed and built to sustain a load.

strut - a member acting to resist axial compressive stress; usually a secondary member.

stub abutment - an abutment within the topmost portion of an embankment or slope having a relatively small vertical height and usually pile supported; stub abutments may also be founded on spread footings.

subgrade - natural earth below the roadway pavement structure.

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.

supported bridge - a bridge with temporary shoring, supports, repairs, or supplemental members that are installed to keep the bridge open despite deficiencies in the permanent structure, pending future repairs or replacement.

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.

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.

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.

system redundancy - a redundancy that exists in a bridge system without load path redundancy, such that fracture of the cross section at one location of a primary member will not cause a portion of or the entire bridge to collapse.

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**.**

tape measure - a long, flexible strip of metal or fabric marked at regular intervals for measuring.

team leader - the on-site, nationally certified bridge inspector in charge of an inspection team and responsible for planning, preparing, performing, and reporting on bridge field inspections.

tee beam - a rolled steel section shaped like a "T"; reinforced concrete beam shaped like the letter "T".

temporary bridge - a bridge which is constructed to carry highway traffic until the permanent facility is built, repaired, rehabilitated, or replaced.

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/thru 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 used to transmit tensile stress; also known as tie bar.

tied arch bridge - a variation of the through arch bridge in which the horizontal reactions are transferred through a tie, which connects the ends of the arch together, and into the foundations.

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.

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.**

traveled way - the portion of roadway for the movement of vehicles, exclusive of shoulders.

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.

truss panel - see **panel.**

tunnel - an underground passage, open to daylight at both ends. See National Tunnel Inspection Standards (23CFR650 Subpart E).

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

ultimate strength - the highest stress that a material can withstand before breaking.

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 bridge inspection diver - the individual performing the inspection of the underwater portion of the bridge.

Underwater Bridge Inspection Manual – a comprehensive FHWA manual on the procedures and techniques for underwater bridge inspection. This manual is available at the following URL: (*www.fhwa.dot.gov/bridge/nbis.cfm*). This manual may be purchased from the Government Publishing Office, Washington, DC 20402 and from National Technical Information Service, Springfield, VA 22161.

underwater 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 - **i**nspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water or by wading or probing, and generally requiring diving or other appropriate techniques.

unknown foundations - foundations of bridges over waterways where complete details are unknown because either the foundation type and depth are unknown, or the foundation type is known, but its depth is unknown, and therefore cannot be appraised for scour vulnerability.

uniform load - a load of constant magnitude along the length of a member.
unit stress - the force per unit of surface or cross-sectional area.

Unmanned Aerial Systems (UAS) - commonly referred to as drones, advanced equipment carrying high-definition cameras or thermal imaging cameras to aid in bridge inspection.

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 lift bridge - a bridge in which the span moves up and down while remaining parallel to the roadway.

vibration - the act of vibrating concrete to compact it.

voided slab - a precast concrete deck unit cast with cylindrical voids to reduce dead load.

voids - an empty or unfilled space in concrete.

W

washer - a small metal ring used beneath the nut or the head of a bolt to distribute the load or reduce galling during tightening.

water-cement ratio - the weight of water divided by the weight of Portland cement in concrete; this ratio is a major factor in the strength of concrete.

waterproofing membrane - an impervious layer placed between the wearing surface and the concrete deck, used to protect the deck from water and corrosive chemicals that could damage it.

waterway 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.

weep hole - 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 joint - a joint in which the assembled elements and members are connected by welds.

welding - the process of making a welded joint.

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 toe - particularly in a filet weld, the thin end of the taper furthest from the center of the weld cross section.

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.

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.

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.

BIBLIOGRAPHY

1. AASHTO. *AASHTO Guide Manual for Bridge Element Inspection, 2nd Edition*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2019. (see 74 FR 68377-68379 at 68379 (May 6, 2022) (23 CFR §§ 650.315, 317) and is regulatory)
2. AASHTO. *AASHTO LRFD Bridge Design Specifications*, Customary U.S. Units, 9th Edition. Washington, D.C.: American Association of State Highway and Transportation Officials, 2020.
3. AASHTO. *The Manual for Bridge Evaluation, 3rd Edition.* Washington, D.C.: American Association of State Highway and Transportation Officials, 2018. (see 74 FR 68377-68379 at 68379 (May 6, 2022) (23 CFR §§ 650.317) and is regulatory)
4. AASHTO. *Movable Bridge Inspection, Evaluation, and Maintenance Manual, 2nd Edition*. Washington, D.C.: American Association of State Highway and Transportation Officials, 2017.
5. AASHTO. *Guide Specifications for Fracture Critical Non-redundant Steel Bridge Members*. Washington, D.C.: American Association of State Highway and Transportation Officials, 1978.
6. AASHTO-PCI. Precast/Prestressed Beam Shapes and Section Properties. (https://www.pci.org/BridgeDesign)
7. ADC. *International Consensus Standards for Commercial Diving and Underwater Operations*. 6.2 Edition. Houston, TX.: Association of Diving Contractors International, Inc., 2016.
8. AFPA. *National Design Specifications: Design Values for Wood Construction.* Washington, D.C.: American Forest & Paper Association, American Wood Council, 2005.
9. AISC. *Steel Construction Manual. 15th Edition*. American Institute of Steel Construction, 2017.
10. ANSI/ACDE. *American National Standard for Divers – Commercial Diver Training – Minimum Standard.* American National Standards Institute, Inc., 2009.
11. ArDOT. *I-40 Hernando DeSoto Bridge Emergency Repair and Inspection After Action Report*, 2021. (https://www.ardot.gov/wp-content/uploads/2021/11/ARDOT-After-Action-Report-I-40-MS-Rvr-Bridge.pdf)
12. ASTM. *ASTM E23-18 Standard Test Methods for Notched Bar Impact Testing of Metallic Materials*. West Conshohocken, Pennsylvania. ASTM International. 2018.
13. Beer, F.P. and E.R. Johnston, Jr., John T. Dewolf. *Mechanics of Materials, 3rd Edition with Tutorial CD.* New York: McGraw-Hill, 2001.
14. Blodgett, O.W. *Design of Welded Structures.* Cleveland, Ohio: The James F. Lincoln Arc Welding Foundation, 1966.

15. Cassady, S. *Spanning the Gate: The Golden Gate Bridge.* Mill Valley, California: Squarebooks, 1986.

16. FAA. *Remote Pilot – Small Unmanned Aircraft Systems (Certification and Recurrent Knowledge Testing) Airman Certification Standards*. FAA-S-ACS-10A. Washington, D.C.: United States Department of Transportation. 2018.

17. FHWA. *Code of Federal Regulations Part 650, Title 23, Subpart C.* Washington, D.C.: United States Department of Transportation, 2022. (https://www.ecfr.gov/current/title-23/chapter-I/subchapter-G/part-650)

18. FHWA. *Bridge Inspection Techniques for Nonredundant Steel Tension Members (NSTMs).* Course FHWA-NHI-130078, 2022.

19. FHWA. *Manual on Uniform Traffic Control Devices for Street and Highways.* Washington, D.C.: United States Department of Transportation, 2009. (MUTCD is regulatory, see 23 CFR §§ 655.601(d)(2)(i) and 603(a))

20. FHWA. *Bridge Inspection Nondestructive Evaluation Seminar (BINS).* Course FHWA-NHI-130099A, 2010.

21. FHWA. *Bridge Inspection Refresher Training.* Course FHWA-NHI-130053S, 2022.

22. FHWA. *Bridge Inspector's Training Manual 70.* Washington, D.C.: United States Department of Transportation, 1979.

23. FHWA. *Bridge Inspector's Training Manual 90.* Washington, D.C.: United States Department of Transportation, 1979, 1991, Revised 1995.

24. FHWA. *Culvert Design.* Course FHWA-NHI-135056, 2010.

25. FHWA. *Culvert Inspection Manual (Archived).* FHWA-IP-86-2, Washington, D.C.: United States Department of Transportation, 1986.

26. FHWA. *Design and Construction Guidelines for Geosynthetic Reinforced Soil Abutments and Integrated Bridge Systems.* Publication No. FHWA-HRT-17-080. Washington, D.C.: United States Department of Transportation, 2018.

27. FHWA. *Design and Evaluation of Steel Bridges for Fatigue and Fracture*. Course FHWA-NHI-130122, 2016.

28. FHWA. *Engineering Concepts for Bridge Inspectors.* Course FHWA-NHI-130054, 2011.

29. FHWA. *Forensic Evaluation of Prestressed Box Beams from the Lake View Drive over I-70 Bridge: Final Report.* Report No. FHWA-PA-2006-017-EMG002. Lehigh University, Bethlehem, PA: United States Department of Transportation, 2006.

30. FHWA. HDS-5 (Hydraulic Design Series No. 5), *Hydraulic Design of Highway Culverts.* 2005.

31. FHWA. HDS-6 (Hydraulic Design Series No. 6), *River Engineering for Highway Encroachments: Highways in the River Environment.* 2001.

32. FHWA. HEC-18 (Hydraulic Engineering Circular No. 18), *Evaluating Scour at Bridges, 5th Edition.* 2012.

33. FHWA. HEC-20 (Hydraulic Engineering Circular No. 20), *Stream Stability at Highway Structures, 3rd Edition.* 2001.

34. FHWA. HEC-23 (Hydraulic Engineering Circular No. 23), *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance, 3rd Edition.* 2009.

35. FHWA. InfoTechnology. Bridge Non-Destructive Evaluation Technologies. (https://infopave.fhwa.dot.gov/infotechnology/bridge/)

36. FHWA. *Lab & Field Testing of AUT Systems for Steel Highway Bridges*. Publication Number: FHWA-HRT-04-124, 2005. (https://www.fhwa.dot.gov/publications/research/infrastructure/structures/04124/02.cfm)

37. FHWA. Memorandum. *Hoan Bridge Failure Investigation*. Washington, D.C.: United States Department of Transportation, 2001.

38. FHWA. Memorandum. Inspection of Nonredundant Steel Tension Members, Washington, D.C.: United States Department of Transportation, 2022.

39. FHWA. Memorandum. *Non-Destructive Testing of Fracture Critical Members Fabricated from AASHTO M244 Grade 100 (ASTM A514/A517) Steel*. Washington, D.C.: United States Department of Transportation, 2021.

40. FHWA. National Bridge Inventory. https://www.fhwa.dot.gov/bridge/nbi/ascii.cfm

41. FHWA. *Nondestructive Evaluation Fundamentals for Bridge Inspection (Web-based).* Course FHWA-NHI-130111, 2020.

42. FHWA. *Report on the Diagnosis, Prognosis, and Mitigation of Alkali-Silica Reaction (ASR) in Transportation Structures.* Report No. FHWA-HIF-09-004, 2010.

43. FHWA. *Safety Inspection of In-Service Bridges.* Course FHWA-NHI-130055S, 2022.

44. FHWA. *Safety Inspection of In-Service Bridges for Professional Engineers*. Course FHWA-NHI-130056S, 2022.

45. FHWA. *Specifications for the National Bridge Inventory (SNBI).* Report No. FHWA-HIF-22-017 Washington, D.C.: United States Department of Transportation, 2022. (https://www.fhwa.dot.gov/bridge/snbi/snbi_march_2022_publication.pdf)

46. FHWA. *Stream Stability and Scour at Highway Bridges for Bridge Inspectors.* Course FHWA-NHI-135047, 2009.

47. FHWA. Technical Advisory. *Inspection of Fracture Critical Bridges Fabricated from AASHTO M270 Grade 100 (ASTM A514/A517) Steel*. Washington, D.C.: United States Department of Transportation, 2011. (https://www.fhwa.dot.gov/bridge/t514032.pdf)

48. FHWA. *Underwater Bridge Inspection.* Course FHWA-NHI-130091, 2010.

49. FHWA. *Underwater Bridge Inspection.* (Reference Manual for NHI Course No. 130091.) Publication No. FHWA-NHI-10-027, 2010.

50. FHWA. *Underwater Inspection of Bridge Substructures Using Imaging Technology, Report FHWA-HIF-18-049*. FHWA, 2018.

51. Frederick, K., and Young, A. *Mianus River Bridge Collapses – Today in History: June 28*. (https://connecticuthistory.org/mianus-river-bridge-collapses-today-in-history/)

52. Griggs, Frank Jr. *Joseph B. Strauss Bascule Bridge*, Structure Magazine – Historic Structures, 2019. https://www.structuremag.org/?p=14194

53. International Fatigue and Fracture Conference, Lehigh University Presentation, 2009.

54. McBriarty, Patrick T. *Chicago River Bridges*. Library of Congress Cataloging-in-Publication Data. 2013.

55. MNDOT. *I-35W St. Anthony Falls Bridge*. (https://www.dot.state.mn.us/i35wbridge/index.html)

56. Murphy, Robin R., et. al. *Robot-Assisted Bridge Inspection after Hurricane Ike.* 2009.

57. NCDOT. *Inspection Manual.* Pages 43-45, North Carolina Department of Transportation.

58. NCHRP. *Legal Implications of Highway Departments' Failure to Comply with Design, Safety, or Maintenance Guidelines.* Research Results Digest No. 129, 1981.

59. NCHRP. *Liability of State Highway Departments for Defects in Design, Construction, and Maintenance of Bridges.* Research Results Digest No. 141, 1983.

60. NCHRP. *Pot Bearings and PTFE Surfaces.* Research Results Digest No. 171, 1989.

61. NCHRP. *Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features.* 1993.

62. NCHRP. *Report 354: Inspection and Maintenance of Bridge Stay Cable Systems*. 2005.

63. NCHRP. *Report 503: Application of Fiber Reinforced Polymer Composites to the Highway Infrastructure.* 2003.

64. NCHRP. *Report 534: Guidelines for Inspection and Strength Evaluation of Suspension Bridge Parallel Wire Cables.* 2004.

65. NCHRP. *Report 564: Field Inspection of In-Service FRP Bridge Decks*. 2006.

66. NHI. National Highway Institute, (www.nhi.fhwa.dot.gov)

67. NTSB. *Highway Accident Report – Collapse of I-35W Highway Bridge*. Washington D.C., 2008.

68. ODOT. *Bridge Inspection Manual.* Section 8.2, Oregon Department of Transportation.

69. OSHA. Occupational Safety and Health Administration, www.osha.gov

70. PennDOT. Publication 100A, *Bridge Management System 2 (BMS2) Coding Manual.* Pennsylvania Department of Transportation, 2022.

71. PennDOT. Publication 238, *Bridge Safety Inspection Manual.* Pennsylvania Department of Transportation, 2021.

72. PennDOT. *Refresher Course*. Pennsylvania Department of Transportation, 2022.

73. Society of Professional Rope Access Technicians. *Certification Requirements for Rope Access Work*. Wayne, Pennsylvania. 2019.

74. Spivey, Justin M. "Pittsburgh, Fort Wayne & Chicago Railway, Calumet River Bridge" *Historic American Engineering Record.* Library of Congress. Washington D.C. 2018.

75. Timber Bridge Information Resource Center. *Crossings*. Issue 1, 1990.

76. Tonias, Demetrios E., Stuart Chen, Richard Garrabrant. *Bridge Engineering, 2nd Edition*. New York: McGraw-Hill, 2002.

77. WSDOT. *Washington State Bridge Inspection Manual.* Washington State Department of Transportation, 2010.

78. WVDOT. Silver Bridge. (https://transportation.wv.gov/highways/bridge_facts/Modern-Bridges/Pages/Silver.aspx)

This page intentionally left blank.