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1.1 INTRODUCTION

In the State of Wisconsin the inventory summaries and demographics of structural highway assets can be found in the Wisconsin Department of Transportation (WisDOT) Bureau of Structures (BOS) Annual Bridge Report.

http://dotnet/hwystructures/development/index.htm

The current standard design life of a bridge is 75 years, while many older structures were designed with shorter life spans. An increasing number of Wisconsin's structures have approached or surpassed their design lives. As these structures continue to age, inspection and maintenance become increasingly important. The structure inspector fills the vital role of assessing the condition of these structures and recommending maintenance actions.

All of the structures discussed in this manual have the potential to cause damage and fatalities if they fail. The collapse of the Silver Bridge over the Ohio River between Point Pleasant, West Virginia and Kanauga, Ohio on December 15, 1967, killed forty-six people and injured nine. The investigation following the Silver Bridge collapse showed the lack of regular bridge condition and safety inspections. There was no national standard for bridge inspection or inspection interval. As a result, Congress created the National Bridge Inspection Standards (NBIS) in 1971 under the 1968 Federal Aid Highway Act. This Act requires that all bridges on public roads be inspected at regular intervals, not to exceed 24 months. The NBIS are federal guidelines pertaining to bridge inspection interval, inspector qualifications, report formats, and inspection and rating procedures. These standards were created in an effort to make bridge inspections thorough and consistent nationwide. The NBIS are minimum standards, and states may elaborate on these guidelines to clarify them or to make them more stringent. The NBIS led to the advent of the National Bridge Inspection Program (NBIP). Prior to 1971, many states did not have formal bridge inspection programs. Therefore, the NBIP mandated that all states maintain an up-to-date inventory of all bridges over 20 feet in span and inspect them at regular intervals using the NBIS criteria.

Since 1971, bridge inspection and structure inspection in general has improved. Each state currently has a bridge inspection program that conforms to the federal standard. In addition, agencies are now required to adopt Element Level inspection to aid in more accurately identifying and determining structural deficiencies which will result in more effectively appropriating funds. As a result, the number of catastrophic bridge failures in recent years has been reduced due to reduced structure inspection intervals, inspection thoroughness, and routine structure maintenance.

Currently, structures such as high mast light towers, traffic sign/signal support structures, shorter span bridges and box culverts, retaining walls, and noise barriers are not under the umbrella of the NBIP or the NBIS. It is becoming increasingly evident, in the shadow of several failures, that these structures also require routine inspection and maintenance. Many states, including Wisconsin, have established inspection programs for many of these ancillary structures.

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1.2 INSPECTION PROGRAM OVERVIEW

1.2.1 Introduction

The goal of the Wisconsin Department of Transportation (WisDOT) Structures Inspection Program is to compile accurate inventory and condition data, along with the functional, structural, maintenance, and safety conditions of transportation-related structures in the state in order to provide a safe and economically prudent infrastructure. The information in these records is gathered through thousands of structure inspections conducted at regular intervals. Standardized and consistent methods must be used to inspect and report on these structures if the program database is to be of any substantial use. Without a standardized system of inspection methods, inspection intervals, and inspection reporting systems, the likelihood of vital information being overlooked or misreported increases dramatically. An inaccurate assessment or an overlooked problem can result in increased long-term maintenance costs and the consequences associated with failure of the structure. Therefore, the primary aim of this program is to provide the structure inspector with a framework that promotes consistency and uniformity in the methods used to inspect and document the condition of highway structures throughout the state.

1.2.2 Program Summary

The Structure Inspection Program operates under the directives of the Federal Highway Administration (FHWA) and the WisDOT Bureau of Structures (BOS). The mission of the program is three-fold:

1. Ensure public safety.

2. Provide the most efficient use of resources for maintaining the serviceability of Wisconsin's transportation-related structures.

3. Maintain compliance with all federal and state laws, rules, and policies.

The Structure Inspection Program in Wisconsin is composed of three separate programs: bridges, tunnels, and ancillary structures.

1.2.2.1 Bridge Inspection Program

The WisDOT Bridge Inspection Program is federally mandated and has been in effect since 1971. The policies of the bridge inspection program are based upon the National Bridge Inspection Standards (NBIS) which are published in the Code of Federal Regulations, 23 CFR 650, subpart C. Bridge Inventory data inspection reports and records are warehoused by WisDOT in an electronic database, the Highway Structures Information System (HSIS). Inventory and inspection data from HSIS for all bridges open to public vehicular traffic in Wisconsin are compiled and forwarded to the FHWA in a prescribed format on an annual basis. This submittal to FHWA includes element level data for those bridges requested by FHWA. Refer to the other portions of Part 2 of this Manual for further specific discussions on bridge inspections.

23 CFR Part 650 defines a bridge as:

"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. Refer to figures 1.2.2.1 – 1 and 1.2.2.1 – 2 for examples on measurement used to define a bridge.

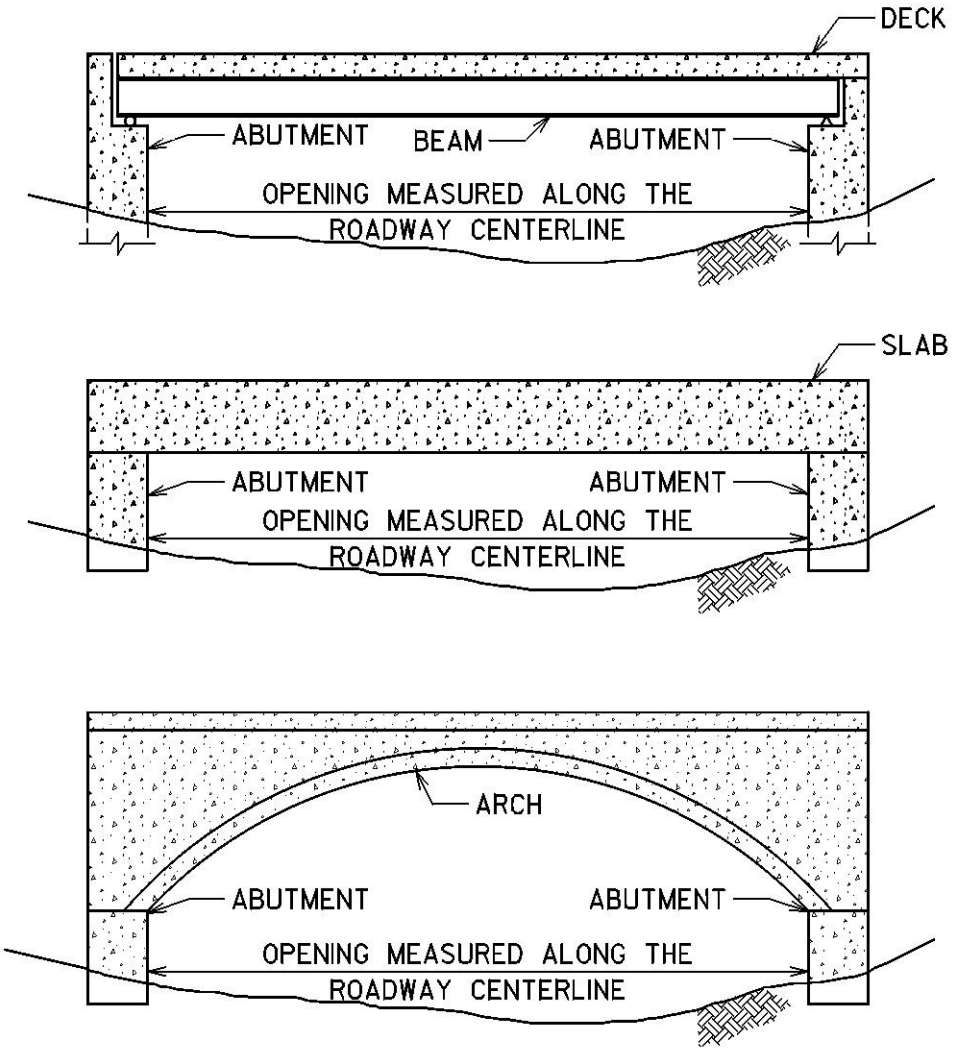


Figure 1.2.2.1-1: Measurement for Definition of a Bridge.

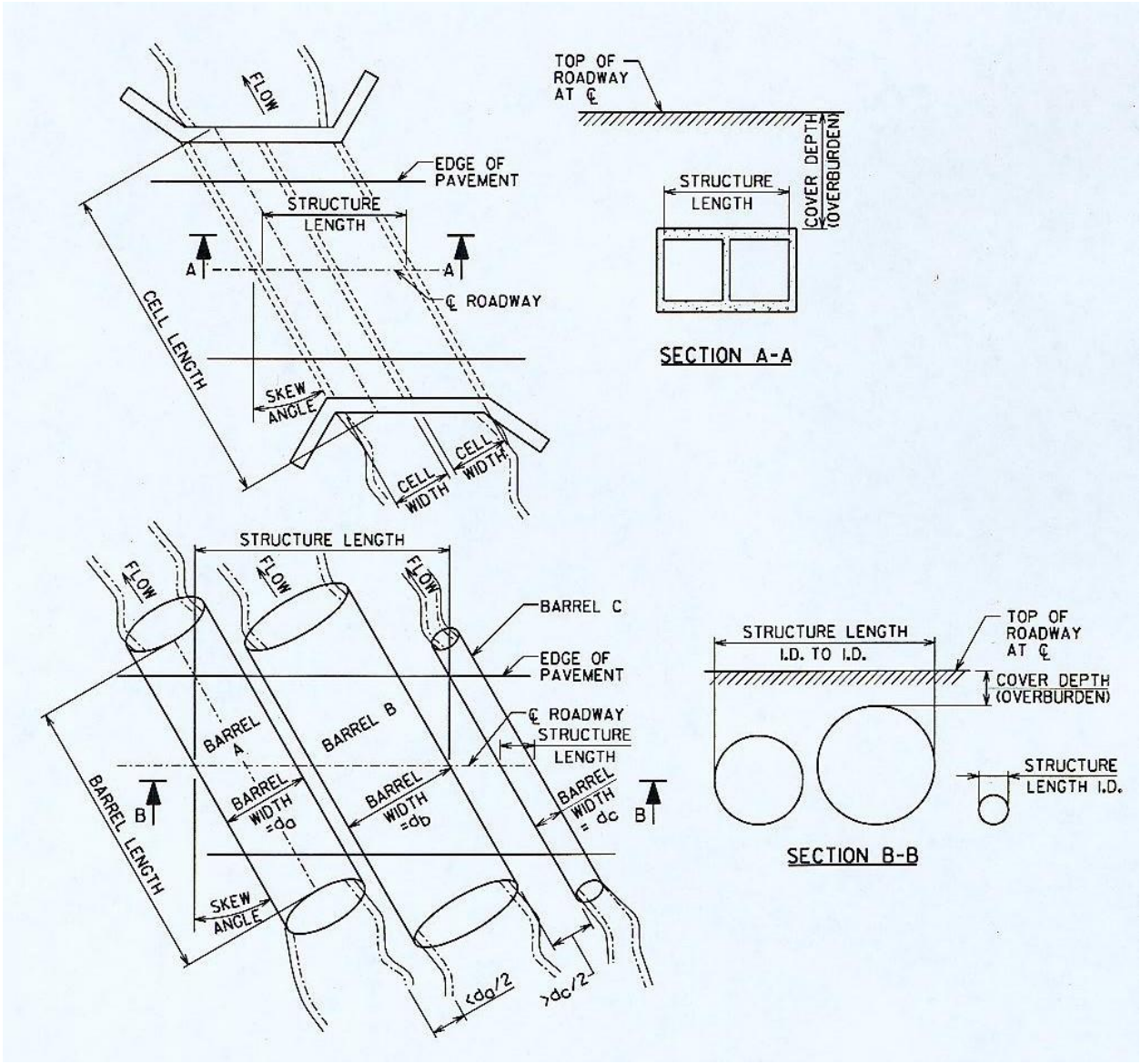


Figure 1.2.2.1-2: Measurement for Definition of a Culvert Structure.

1.2.2.2 Ancillary Structures Inspection Program

There currently is no national standard for the inspection of ancillary highway structures. WisDOT recognizes that neglecting the inspection of ancillary highway structures can result in increased maintenance costs and pose a safety risk to the traveling public. As a result, WisDOT has created a program for the routine inspection and maintenance of State-owned ancillary highway structures. Refer to Part 4 of this Manual for further specific discussions on the inspection of ancillary highway structures.

1.2.2.3 Tunnel Inspection Program

The WisDOT Tunnel Inspection Program (WiTIP) is federally mandated and was developed in 2021. The policies of the tunnel program are based upon the National Tunnel Inspection Standards (NTIS). Tunnel inventory data, inspection reports and records are warehoused by WisDOT. Inventory and inspection data for all publicly owned tunnels in Wisconsin are compiled and forwarded to the FHWA in a prescribed format on an annual basis. Refer to the other portions of Part 6 of this Manual for further specific discussions on tunnel inspections.

1.2.3 Program Leadership and Organization

There are several levels of administration or management for the statewide structure inspection program. WISDOT BOS, more specifically the Chief Structures Maintenance Engineer, is charged with administering the Statewide Structure Inspection Program. Therefore, the Chief Structures Maintenance Engineer also holds the title of Statewide Structure Inspection Program Manager. The Assistant Statewide Program Managers (ASPM), Region Program Managers (RPM), and County Program Managers (CPM) follow under the leadership of the Statewide Structure Program Manager (SPM).

The organization of the state structure inspection program is shown in Figure 1.2.3-1 and described in detail later in this chapter.

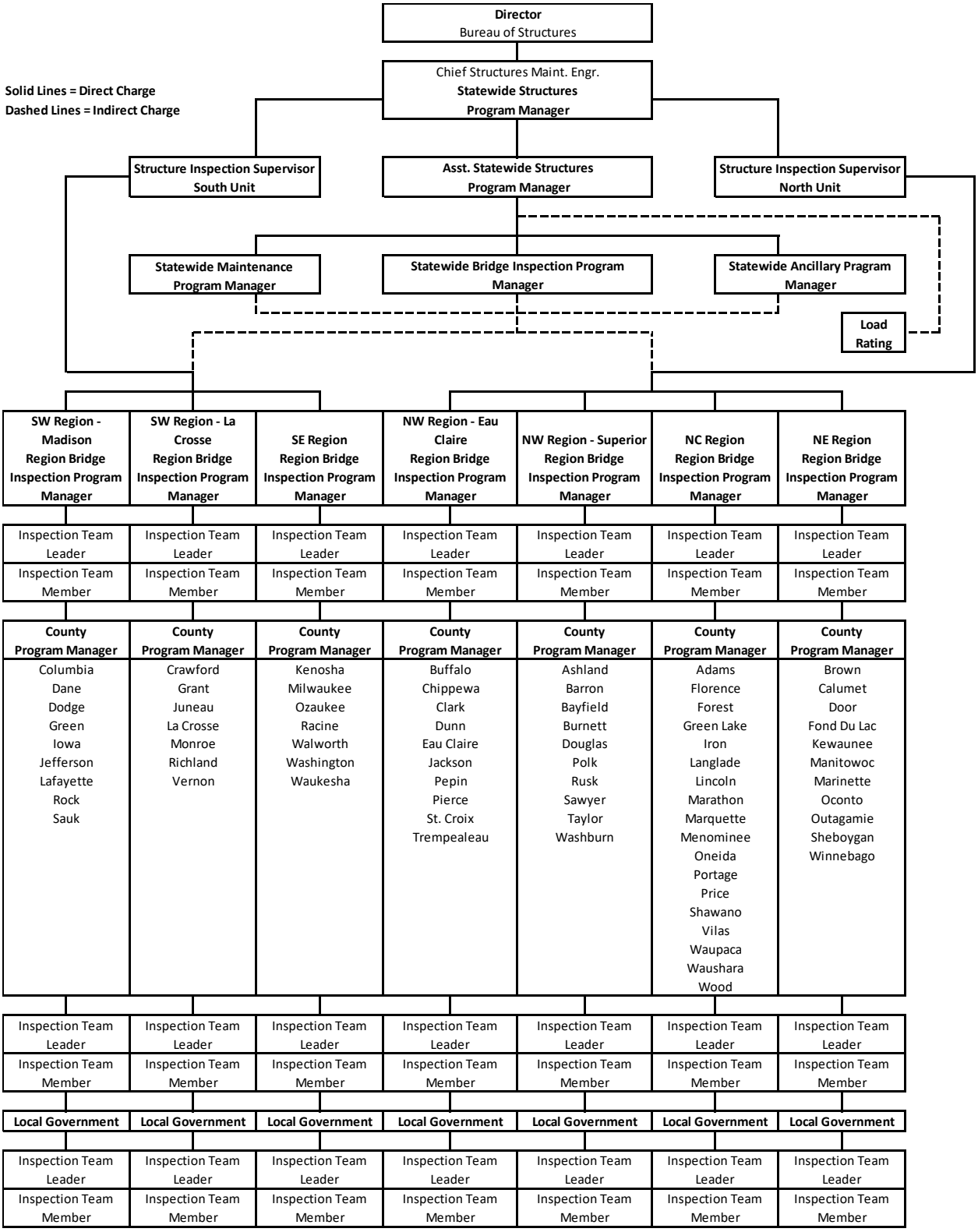


Figure 1.2.3-1: Structure Inspection Program Organization.

1.2.3.1 Management Qualifications

For the safety of the public, it is important that qualified personnel inspect Wisconsin's transportation facilities. Structure inspectors are required to render judgments pertaining to the safety and integrity of the structures they inspect. Inspection program managers make important decisions ranging from suggestions regarding the allocation of scarce rehabilitation dollars to the decision to close a major structure. Therefore, it is important that inspectors and program managers are highly trained and adept individuals who understand the mechanics, behavior trends, and economics of a wide variety of structure types. WisDOT has adopted strict criteria for the qualification of structure inspectors and inspection program managers. In addition, all structure inspectors and inspection program managers are expected to be thoroughly familiar with this Manual.

The rest of this chapter primarily deals with the bridge inspection program. For more information on the Ancillary Program, see Part 4. For more information on the Tunnel Program, see Part 6.

The SPM, ASPM, CPM and RPM must have the following minimum qualifications (shown also in Figure 1.2.3.1-1): as detailed in NBIS 23 CFR Part 650 National Bridge Inspection Standards Section 650.309 (a)

1. Either a registered Wisconsin licensed professional engineer or have ten-years of bridge inspection experience that includes a minimum:

    a. 200 or more bridge inspections over a 10-year period and 5 years of other bridge experience.

    OR

    b. 400 or more bridge inspections over a 10-year period.

2. Complete an FHWA-approved comprehensive bridge inspection training course as described in 23 CFR Part 650 Section 650.309(h) and score 70 percent or greater on an end-of-course assessment. The National Highway Institute (NHI) course "Safety Inspection of In-Service Bridges" (National Highway Institute (NHI) Course Number 130055), or the course "Safety Inspection of In-Service Bridges for Professional Engineers" (NHI) Course Number 130056) are approved courses;

3. Complete a cumulative total of 18 hours of FHWA approved bridge inspection refresher training over each 60-month period.

4. Most recent annual WisDOT Structure Inspection Refresher Training.

5. Maintain documentation supporting the satisfaction of the above requirements.

)**LEVEL**	**QUALIFICATIONS**	**COMMENTS**
SPM		WisDOT Policy & NBIS
ASPM		

RPM CPM	Registered Wisconsin licensed professional engineer, Or have ten-years of bridge inspection experience as described above; Successfully complete NHI 130055 “Safety Inspection of In-Service Bridges” or NHI 130056 “Safety Inspection of In-Service Bridges for Professional Engineers”; Complete a cumulative total of 18 hours of FHWA approved bridge inception refresher training over each 60-month period; Maintain documentation supporting the satisfaction of the above requirements.	

Figure 1.2.3.1-1: Program Manager Qualifications

Statewide Inspection Program Manager

The Statewide Program Manager (SPM) is responsible for the organizational unit management of structure inspections, inventory, and reporting. The SPM is responsible for the overall supervision of the inspection teams in the field. The SPM advises on technical issues concerning problems or deficiencies discovered during the inspection. The SPM also assists the Assistant Statewide Program Manager (ASPM) or Inspection Team Leader (ITL) to determine what, if any, maintenance or repair actions are appropriate. Decisions regarding load posting and long-term bridge closures require the approval of the SPM or delegate.

A sound background in structure inspections, rehabilitation, and maintenance is required for the SPM to be an efficient and effective manager. On occasion, specialized knowledge and skills in fields such as structural design, construction, mechanical systems, electrical systems, soils, construction materials, and emergency repair techniques will be required.

The SPM is the liaison between the FHWA and WisDOT. The SPM is responsible for ensuring that WisDOT complies with Federal directives regarding structure inspection and maintenance. This includes making sure that all structures are inspected at the proper intervals and that the state structure files are kept up-to-date and accurate. This person is responsible for the Leadership and overall management of the statewide structure inspection and inventory program, statewide structure load posting program, statewide structure maintenance program, and statewide training of structure inspectors. The SPM has overall responsibility for personnel supervision; scheduling of structure inspections, maintenance, securing inspection and repair specialists; and scheduling the use of WisDOT-owned underbridge access machines. The responsibilities of this position that pertain directly to structure inspection and maintenance are described in more detail below:

1. Program leadership and management of the statewide structure inspection, inventory, and maintenance programs.

a. Monitor federal structure inspection requirements and recommend policy changes to assure that Wisconsin's program complies with all federal directives.

b. Ensure that inspection data is uploaded to centralized data file within mandated timeframes.

c. Review and confirm that all structures in the state are inspected at an interval and by a method consistent with the National Bridge Inspection Standards, Section 84.17 SS, and Administrative Rule Trans. 212.

d. Leadership and the oversight of quality assurance reviews of region and local (County) inspection program operations and bridge inspectors. Coordinate with federal, state, and local governmental agencies.

e. Leadership and management of special inspection and in-depth inspection programs for structures with nonredundant steel tension members (NSTM), underwater members or unique or special features requiring additional attention during inspection to assure the safety of such structures.

f. Conduct annual inspections of state border bridges in company with the respective state's personnel and region offices to determine required maintenance efforts or other courses of action and then lead the effort to accomplish Wisconsin's portion of the required maintenance or action.

g. Negotiate and coordinate updates to border bridge agreements with bordering states on the inspection, maintenance and repair of border bridges. Responsible for budgeting, tracking, and invoice approval for work performed by bordering states on Border Bridges.

h. Manage and provide oversight of the moveable bridge aid programs.

2. Program leadership and management of the statewide load posting program.

   a. Oversee monitoring of the signing of bridges statewide that require load posting.

   b. Review and confirm that load posted structures receive required inspections as required by federal and state laws, rules, and policy.

   c. Oversee quality assurance reviews with personnel from region offices and local units of government to assure that proper signing is in place for structures that require load posting.

3. Leadership and management of related programs.

   a. Coordinate and provide technical support for the inspections and repairs of the Merrimac Ferry to ensure safe and available service to the traveling public.

   b. Act as liaison between the Bureau of Transit, Local Roads, Railroads and Harbors and the BOS to accomplish the state-owned railroad bridge inspections.

c. Work with staff and regions to furnish inspectors to accomplish the above.

d. Assist Regional Program Manager (RPM) in arranging vehicles and equipment as necessary for additional testing of railroad bridges.

e. Manage a technological transfer program for inspection of bridges and ancillary structures on the state-owned short line railroads.

f. Coordinate and provide technical support for the inspection and certain maintenance activities involved with weigh scale buildings, pits, and platforms.

4. Training of statewide bridge inspection, maintenance, and repair specialists to perform their duties.

   a. Develop, monitor, and update training programs for state, county, city, village, town, and private consultant structure inspectors in bridge inspection, maintenance, and repair techniques.

   b. Arrange or conduct structure inspection, maintenance, and repair training programs throughout the state.

   c. Assist the region offices in giving refresher structure inspection and maintenance training programs.

5. Leadership and management of bridge inspection, maintenance, and repair personnel.

   a. Manage structure inspectors, structure inspection specialists, and structure repair specialists to meet the needs of the statewide structure inspection and repair program.

   b. Manage the WisDOT-owned under-bridge access equipment to assist in the inspection, maintenance, and repair of structures statewide.

   c. Manage inspection and repair activities to assure proper staffing by region and county personnel. Retain the services of private consultants or contractors to supplement region or county staff, as needed, to perform specialized inspection, testing or repair techniques.

   d. Provide budget and training for personnel on proper access, equipment operation, and safety procedures.

6. Leadership in the determination, formulation, and administration of programs and policies.

   a. Develop, implement, and evaluate policies, standards, procedures, and programs.

   b. Analyze federal and state legislation, administrative rules, and national and industry standards for incorporation in programs and policies.

c. Recommend the revision of legislation and participate in new legislation development.

7. Leadership and overall responsibility for prompt, decisive, and effective responses to emergencies (e.g., floods, major bridge damage, and bridge failures).

8. Determination, development, and management of inspection and maintenance budgets for the Structures Maintenance Section within BOS.

Assistant Statewide Program Manager

The Assistance Statewide Program Manager (ASPM) assists the SPM in the management and oversight of the state's inspection program. The ASPM is a statewide expert for bridge safety inspections; providing alignment, uniform interpretation, and application of the requirements to ensure compliance with State and Federal Highway Administration (FHWA) regulations. General responsibilities include tracking statewide compliance with FHWA Bridge Inspection Program Inspection Program metrics, participating in field reviews and file audits, assisting in the development and implementation of Plans of Corrective Action, providing reports to FHWA, assisting with policy development, and managing consultant projects related to inspection services.

The ASPM assists in management of the state inspection program which includes working with region, and local (county) programs for compliance with federal and state laws and policies regarding structure inspection. The responsibilities of this position that pertain to structure inspection are described in more detail below:

1. Program leadership and management of the statewide structure inspection, inventory, and maintenance programs.

   a. Assist with statewide oversight of state and local agency bridge inspections by performing monthly, quarterly, and annual data checks, and coordinate with bridge owners and bridge inspectors to address deficiencies.

   b. Serves as a statewide technical resource for complex, non-redundant, fracture critical, fatigue sensitive, underwater, damage, in-depth, and scour monitoring inspections.

   c. Leads the implementation of NSTM, In-depth, and Special inspection requirements and providing state, local agency, and private bridge owners with the support and guidance necessary to achieve compliance.

   d. Administer bridge inspection consultant contracts and perform project management functions.

   e. Possess and maintain Team Leader (TL) and NSTM TL status.

   f. Evaluate and approve a potential Team Leader's overall bridge inspection experience.

2. Assist with program management of the structures inspection, inventory, and maintenance programs.

a. Monitor region and staff performance of structure inspection procedures and methods to assure policy compliance and statewide continuity.

b. Assist with and if necessary lead quality assurance reviews of region and local (county) program inspection programs. Coordinate with federal, state, and local governmental agencies.

c. Support Regional Program Managers (RPM) in the oversight of the inspection of local structures including recommendations for posting load restrictions, closing of structures, replacement, and interval of inspections.

d. Coordinate the relationships between the inspection program, bridge management engineer, and the structure replacement, structures maintenance, and structure asset management programs.

e. Innovation: Analyze and evaluate new methods and equipment for incorporation in the inspection procedures.

f. Provide direction and oversight on which structures should receive an in-depth inspection, underwater inspection, or other special inspection procedures.

g. Manage the consultant contracts for structure inspections related needs.

h. Manage the inspector certification program; assigning inspector numbers and maintaining credentials on bridge inspectors.

i. Statewide technical resource for routine, NSTM, special, underwater, damage, in-depth, and scour monitoring inspections.

3. Planning and directing the establishment and implementation of policies, programs, manuals, training, and services for the region and local structures maintenance and inspection programs.

4. Leadership and delegation of prompt, decisive, and effective responses to emergencies (e.g., floods, major bridge damage, and bridge failures).

5. Provide assistance and recommendations in the development and management of inspection and maintenance budgets.

6. Provide recommendations to the SPM on inspection and maintenance budget issues for the Structures Inspection & Repair Unit.

Region Program Manager

The WisDOT Region Program Managers (RPM) are responsible for all state-owned bridges in their respective regions. The RPM has oversight responsibility for all bridges; including county bridges, city bridges, village bridges, and township bridges in their geographic region. County Program Managers (CPM) report to the RPM on matters related to structure inspection policy and procedures. The RPM is responsible for the following items:

General

1. Wisconsin statutes delegate the Inspection program manager responsibilities for locally owned bridges to county program managers where the county program manager meet the qualifications for inspection program managers as described previously or their designee.

2. Inspection: the inspections of all state-owned bridges are scheduled and coordinated by the RPM. The RPM is responsible for the Quality Control (QC) of inspections on state owned bridges. The RPM is responsible for the oversight of the County inspection programs in their geographic region and monitoring the completion and timeliness of local bridge inspections.

3. Maintenance: All maintenance required on state-owned bridges is arranged by the region staff working with the RPM, typically through the counties. All maintenance items required on locally-owned bridges are the responsibility of the bridge owner and should be acted on in a timely manner as identified by the RPM in consultation with the SPM. If critical, serious or safety related maintenance items on locally-owned bridges are not dealt with in a timely manner, the region has authority to order the county to instigate the necessary repairs or close the bridge entirely or partially to traffic.

State-Owned Bridges

1. Maintain up-to-date and accurate inventory and inspection data in HSIS for all state-owned bridges in their region.

2. Coordinate and assure compliance with structure inspection policies and procedures for all state-owned bridges in their region. As part of this process, the RPM shall:

    a. Ensure that all bridges under their jurisdiction are receiving timely and appropriate inspections by qualified personnel.

    b. Ensure all structure inspection results are entered in accordance with section 1.2.5.8.

    c. Ensure that all state-owned bridge inspection results are entered into WisDOT Highway Structures Information System (HSIS).

    d. Ensure Load posted verification forms are entered into HSIS for any new or updated posting signage.

3. Coordinate and ensure compliance with bridge replacement and rehabilitation procedures for all state-owned bridges in the region. For this process, the RPM, or their designee, shall determine which bridges are eligible for replacement or rehabilitation, and prioritize those bridges.

4. Budget, Schedule, and Coordinate maintenance activities on state owned structures. Institute Bridge Preservation activities as directed by Bridge preservation policies.

5. The region shall submit an annual update of its structure inventory (Completion list) to the Statewide Structure Management Engineer in the Development Section of BOS

denoting new structures and structures no longer in service, as well as construction work completed on bridges.

Locally-Owned Bridges

1. The region does request information from county managers, and those county managers are required to fulfill such requests in a timely manner as defined by the RPM in consultation with the SPM.

2. Keep the SPM and ASPM informed of changes to county Highway commissioners and CPM's

3. Monitor timeliness of local structure inspections through automated scheduling reports and coordinate with the CPM to ensure inspections are getting completed and inspected on time. Keep the SPM and ASPM informed of the status and reasoning for inspection that are late or past-due. As part of this process, the RPM shall:

    a. Ensure that all bridges are receiving timely and appropriate inspections by qualified personnel.

4. Ensure all structure inspection results are entered in accordance with section 1.2.5.8.

5. Serve as the appeals administrator should a county not agree with a bridge inspection report or its recommendations.

6. Review county program manager qualifications. In counties where the County Commissioner does not meet the criteria for program managers, as set forth in Figure 1.2.3.1-1, the RPM must have on file a written agreement with the county commissioner that details and delegates the administrative responsibilities for the county program to a qualified program manager. Forward a copy of this agreement to the SPM.

7. Maintain a schedule of and assist with regular quality assurance (QA) evaluations of the county programs. See Section 1.2.6 for details and information on the QA Program.

County Program Manager

Wisconsin statute places primary responsibility on local authorities for the inspection of bridges within their respective jurisdictions. WisDOT expects the county commissioners to perform, or cause to be performed, all inspection and load posting responsibilities for local owned structures on county, city, village, and town roads in their county. The responsibilities of the county inspection program shall be set forth in writing with a copy send to WisDOT. The county commissioner may be the county program manager (CPM) for structure inspections. The county manager is responsible for the following items and must meet the qualifications set forth previously for Inspection program manager. If a county commissioner does not meet the qualifications, then he/she shall make a formal, written agreement with a qualified individual to serve as county inspection program manager. The county commissioner shall file a copy of this agreement with the RPM, who will forward a copy to the SPM.

In the event a local authority desires to perform those responsibilities historically performed by a county program manager for bridges within its jurisdiction, the local authority must work with that county's Highway Commissioner to allocate those responsibilities between the local

authority and the county. Refer to the section below on the City, Town, and Village Manager responsibilities for additional information.

1. WisDOT does request information from county program managers. County program managers are required to fulfill such requests in an efficient and timely manner.

2. Maintain an up-to-date library of all locally-owned bridge records for the county. The county program manager shall coordinate and assure compliance with structure inspection procedures for all locally-owned bridges in the county. As part of this process, the county program manager shall:

    a. Notify each local unit of government, in writing, that all local bridges shall be inspected at an interval as defined in section 1.3.3.3 of the Structure Inspection Manual.

    b. Ensure that all bridges are receiving timely and appropriate inspections by qualified personnel.

    c. Ensure all structure inspection results are entered in accordance with section 1.2.5.8. Provide guidance and assistance as necessary to the local units of government.

3. Provide for Quality Control (QC) of the inspection program in the county.

4. Coordinate and ensure compliance with bridge replacement and rehabilitation procedures for all locally-owned bridges in the county. For this process, the county program manager, with the approval of the county commissioner, shall:

    a. Notify each local unit of government, in writing, of the eligible local bridges and ask that they priority rate these bridges.

    b. Meet with each local unit of government for eligible bridges to discuss their bridge stewardship responsibilities.

5. Serve as the appeals administrator should a local unit of government not agree with a bridge inspection report or its recommendations.

6. Coordinate and assure compliance with TRANS 212.10 requirements for load posted or closed locally-owned bridges in the county. Ensure current load posted verification forms are entered into HSIS when posting signage is erected or updated.

City, Town, and Village Manager

Wisconsin statute places primary responsibility on local authorities for the inspection of bridges within their respective jurisdictions, WisDOT expects the county in which those bridges are located to perform those inspection and load posting responsibilities. In the event a local authority desires to perform those responsibilities historically performed by a county program manager for bridges within its jurisdiction, the local authority must work with that county's Highway Commissioner to allocate those responsibilities between the local authority and the county. Such allocation of responsibilities shall be set forth in writing with a copy sent to WisDOT. Any individual assigned by the local authority to perform those locally allocated

responsibilities must meet all applicable qualifications set forth previously for Inspection program managers. Any reference within this Manual to a county program manager, county inspection program manager, or CPM, shall also refer (where applicable) to the qualified individual assigned by a local authority to perform such locally allocated responsibilities.

Load Rating Engineer

The load rating engineer is an individual responsible for determining and maintaining current and accurate load carrying capacity for a bridge through data obtained from existing plans and pertinent condition and defect information provided from the inspection report. Responsibilities include reviewing inspection reports for changed conditions that warrant revisions to load ratings on file through automated reporting, revising load ratings where required, creating load rating files for new bridges, and ensuring that the findings from load ratings are implemented.

Qualifications

The minimum qualifications for a Load Rating Engineer are established in the 23 CFR Part 650 National Bridge Inspection Standards, Section 650.309 (d): which states:

Load ratings must be performed by, or under the direct supervision of, a registered professional engineer.

1.2.3.2 Field Inspection Team Qualifications and Responsibility Attributes

The field inspection team consists of the personnel who are in the field doing the majority of the hands-on inspection work. These personnel typically include Inspection Team Leaders (ITL) and Inspection Team Members (ITM), both of which are explained in detail below. County, Region and Statewide Program Managers are automatically qualified to do field inspection as an ITL because the job requirements for those positions encompass the job requirements for the ITL, as described below. As an ITL, the Program Manager must meet the qualification stated above in section 1.2.3.1.

For Sign/Signal Structure Inspection Team Leaders, Including High Mast Light (HML) poles, see SIM 4.1.1 for TL qualifications.

Inspection Team Leader (ITL)

The ITL is responsible for leading the structure inspection team and planning, preparing, and performing structure inspections. The ITL is ultimately responsible for reviewing the inspection report or form and signing it. The ITL is also responsible for the content of any written inspection report and serves to ensure the quality of any inspections they lead. The ITL shall be familiar with this Manual and preferably have a strong background in such areas as structural engineering, structure behavior trends, bridge maintenance, and rehabilitation techniques. The ITL is also responsible for the general safety of the work site. Safety items can include obtaining and monitoring any required traffic control, ensuring each ITM complies with safety procedures, proper use of access equipment, and more. There must be at least one ITL at the structure site at all times during each field inspection. A more complete discussion of the duties and responsibilities of the ITL are in Section 1.4.2.

Federal and State laws and rules define the qualifications and duties of the inspection team leader. People who sign inspection reports without meeting those qualifications, or who sign reports without being at the structure site and participating in the inspection as defined in the American Association of State Transportation and Highway Officials (AASHTO) Manual for Bridge Evaluation, may be subject to prosecution for forgery or fraud under Wisconsin Statutes §§943.38 and 943.39 or other applicable state or federal laws.

ITL Qualifications

A bridge inspection team leader (TL) for all bridge inspection types must meet the qualifications of 23 CFR Part 650 Section 650.309 (b) and WisDOT experience requirements. Additional requirements exist for TL for NSTM, Underwater Dive, and Complex bridge inspection team leaders.:

The methods to qualify as a bridge inspection TL are as follows:

1. Pass the FHWA approved National Highway Institute (NHI) comprehensive bridge inspection training course (NHI #130055 or #130056).
2. Meet at least one of the following:
   a. Wisconsin PE and 6 months bridge inspection experience that includes a minimum:
      i. 20 or more bridge inspections over the last 3 month to 3-year period and 3 months of other bridge experience
      OR
      ii. 40 or more bridge inspections over the last 6-month to 3-year period
   b. Bachelor's degree in engineering from an ABET accredited college or university, a passing score on the Fundamentals of Engineering Exam and 2 years bridge inspection experience that includes at a minimum:
      i. 40 or more bridge inspections over the last 1-5-year period and 1 year of other bridge experience
      OR
      ii. 80 or more bridge inspections over the last 2-5-year period
   c. Associates degree in engineering or engineering technology from an ABET accredited college or university and 4 years bridge inspection experience that includes a minimum:
      i. 80 or more bridge inspections over the last 2–8-year period and 2-years of other bridge experience
      OR
      ii. 160 or more bridge inspections over the last 4–8-year period
   d. 5-years or more bridge inspection experience that includes a minimum:
      i. 100 or more bridge inspections over the last 2.5–10-year period and 2.5 years of other bridge experience
      OR
      ii. 200 or more bridge inspections over the last 5–10-year period
3. Complete a cumulative total of 18 hours of FHWA-approved bridge inspection training over each 60-month period. Submit together all refresher training certificates in one PDF document on WisDOT website.
4. Most recent annual WisDOT Structure Inspection Refresher Training.

NSTM, Underwater Dive, and Complex Bridge Inspection Team Leader Additional Requirements

- *Non-redundant steel tension member (NSTM)* Team Leader must have taken and passed the NHI NSTM inspection training course. NSTM TL's who have taken and passed NHI 130078 Fracture Critical training do not have to complete the new NSTM NHI course to remain NSTM TL.

- *SRM/IRM* - Team Leader must have NSTM qualification to inspect these members.

- *Underwater Dive* Team Leader must have taken and passed the NHI Underwater Bridge Inspection training course. Underwater TL's who have taken and passed NHI 130091 Underwater Bridge Inspection/Underwater Inspection of Bridges do not have to complete the current Underwater NHI course to remain underwater TL.

- *Complex Bridge* in Wisconsin is classified as Moveable or Cable Stay bridges. *Moveable bridge inspection TL* must meet the qualifications outlined in SIM 3.1.3. *Cable stayed bridge inspection TL* must be a qualified TL for in-service bridge inspections and NSTM TL qualified.

Inspection and Other Bridge Experience

An individual's inspection experience from any state must include active participation in bridge or small bridge* inspections in a team member role using NBIS procedures provided it develops the skills necessary to properly perform an NBIS bridge inspection. Documentation of the inspection experience must be provided. Provide a list of the structure IDs with month/year of inspection. When a list of structure ID with month/year is not available, provide an affidavit from the NBIS team leader affirming and explaining the inspection experience or other acceptable documentation showing proof of inspection experience.

*Inspection of Small Bridges (WisDOT defined C type structures) following NBIS inspection procedures may be used for up to 50% of the required inspection experience. Documentation of the inspection experience must be provided.

Other bridge experience includes bridge design, bridge load rating, bridge maintenance, and bridge construction experience. An affidavit or letter of validation must be provided from a peer or supervisor directly involved with the experience of the applicant. Railroad bridge inspections do not count towards inspection experience but does count as other bridge experience.

Evaluation of an individual's other bridge experience for compliance with the requirements for a team leader will consider the following:

a. The relevance of the individual's actual experience, i.e., has the other bridge experience enabled the individual to develop the skills needed to lead properly a bridge safety inspection.
b. Exposure to the problems or deficiencies common in the types of bridges being inspected by the individual.
c. Complexity of the structures being inspected in comparison to the knowledge and skills of the individual gained through their prior bridge experience.
d. The individual's understanding of the specific data collection needs and requirements.

e. Demonstrated ability to lead bridge safety inspections.
f. The level of oversight and supervision demonstrated by the individual in prior experience.

Supplemental Qualification Guidance

WisDOT may consider an inspector's qualifications earned from another state DOT. WisDOT may factor equivalent experience when the inspector's qualifications relate to complex and/or signature bridges. Meeting one or more of the following may be considered as experience for an inspection team leader (TL).

1. Inspector reciprocity: WisDOT may consider an inspector's TL status obtained from another state DOT if the qualification or examination process is acceptable to WisDOT. The determination will be on a case-by-case basis if granting Wisconsin TL status.

2. Inspector experience with complex or signature bridge(s): WisDOT may consider inspection team member involvement with complex or signature bridges to be equivalent to a portion or all of the required inspection participation experience listed under SIM 1.2.3.2. Inspector must submit inspection experience explaining in detail the involvement with the structure inspection of complex or signature bridges, specifically related to the team member role using the NBIS procedures. The determination will be made on a case-by-case basis when granting TL status.

TL Application

TL applicants need to fill out DT2001 and will require:

1. The TL or PM applicant must provide their full name, home address, telephone number, email address, current employer, employer address and employer telephone number.
2. Copies of completion certificates from bridge inspection training courses. If certificates are not available, the PM will conduct an interview and share information with FHWA division office for concurrence.
3. A list of inspected bridges with corresponding month/year. When lists of inspected bridges is not available, provide an affidavit from NBIS team leader validating the inspection and other bridge experience,
4. A description of other bridge experience including time periods with dates and Name and Telephone # for reference during each period of work. DT2001 Applications and credentials shall be submitted on our website.

ITL Desirable Physical Attributes

1. Uncorrected visual acuity of at least 20/40 based on the Jaeger Chart or have a corrected visual acuity of at least 20/40 based on the Jaeger Chart with the mandatory use of a corrective device (glasses, contact lenses, etc.) during the course of any inspection field work.

2. A bridge inspector should be able to perform the following physical tasks:

   a. Walk on riprap and steep slopes;

b. Climb over fences;

c. Work comfortably at heights;

d. Work comfortably in confined spaces;

e. Work comfortably close to live traffic;

f. Work comfortably in or near water; and

g. Perform other similar field tasks.

Inspection Team Member (ITM)

This individual assists the ITL in the field. It is expected that this person, at a minimum, is familiar with appropriate parts of this Manual and has a competency level sufficient to follow the directives of the ITL. To ensure competency, all ITMs should be encouraged to take the two-week FHWA "Safety Inspection of In-Service Bridges" course. ITMs are essentially apprentices and should have the goal of becoming ITLs. ITLs and supervisors should provide appropriate training and guidance to assure the ITMs progress toward this goal. ITMs do not have the authority to sign inspection forms and should never do so. However, ITMs shall have their name entered on the inspection form under team members to document their participation and experience. They shall also maintain a log of their experience for future reference.

ITM Qualifications

1. Have the competency and ability to carry out the duties assigned by the ITL (see Figure 1.2.3.2-1).

ITM Desirable Physical Attributes

1. Uncorrected visual acuity of at least 20/40 based on the Jaeger Chart or have a corrected visual acuity of at least 20/40 based on the Jaeger Chart with the mandatory use of a corrective device (glasses, contact lenses, etc.) during the course of any inspection field work.

2. A bridge inspector should be able to perform the following physical tasks:

   a. Walk on riprap and steep slopes;

   b. Climb over fences;

   c. Work comfortably at heights;

   d. Work comfortably in confined spaces;

   e. Work comfortably close to live traffic;

   f. Work comfortably in or near water; and

   g. Perform other similar field tasks.

Underwater Bridge Inspection Diver

All diving operations shall be conducted in accordance with all applicable federal and state regulations. Occupational Safety and Health Administration (OSHA) 29 CFR 1910 Subparts T and Y, as well as USCG 46 CFR 197.200, mandate training qualifications for all members associated with a diving activity. These federal regulations mandate diving and emergency management training (CPR and First Aid) requirements. The National Bridge Inspection Standards states the training requirements for underwater bridge inspection divers.

Qualifications

In addition to diving and emergency management training, structure inspection training and qualifications shall be the same for underwater inspections as for above water inspections. However, the team leader shall be a certified diver and be able to perform the underwater inspection unless an "unmanned" method ensures a sufficient level of certainty. Furthermore, the team leader shall be on-site at all times and partake in at least 50 percent of the diving to ensure proper detection and assessment of defects.

Team members for underwater inspections shall have training and experience necessary to perform assigned tasks in a safe and healthful manner. It is preferable that all team members be certified divers since each may be called upon to act in a variety of roles including standby rescue diver, in an emergency. In accordance with OSHA regulations, the dive team should consist of at least three individuals.

An underwater bridge inspection dive team leader must meet the Inspection Team Leader qualifications as identified earlier in this section and successfully complete an FHWA approved underwater bridge inspection diver training course as described in 23 CFR Part 650 NBIS Section 650.309 (e).

Specialists

Non-Destructive Evaluation (NDE)

Non-Destructive Testing (NDT) personnel shall be qualified in accordance with nationally recognized NDE personnel qualifications practice or standards such as ANSI/ANST-CP-198, SNT-TC-1A, MIL STD 410, NAS-410 or a similar document. The practice or standard used and the applicable revision shall be specified in the contractual agreement between the structure owner and the NDE inspector.

Consultant, Contractor, and Subcontractor Personnel

A. NDE Requirements

Consultants, contractors, and their subcontractors performing NDE [magnetic particle testing (MT), liquid (dye) penetrant testing (PT), ultrasonic testing (UT),etc.] on bridges or ancillary structures as defined in the Wisconsin Structure Inspection Manual shall submit their written practice and personnel certifications for review prior to being allowed to solicit or perform these activities.

Consultants, contractors, and their subcontractors performing NDE (visual inspection) on bridges or ancillary structures, as defined in the *Wisconsin Structure Inspection Manual,* shall:

1. have taken and passed the NHI 80-hour course (Safety Inspection of In-service Bridges) or NHI 1 week bridge inspection course for PE's or

2. be certified as an American Welding Society Certified Welding Inspector (AWS/CWI) or

3. Submit their written practice and certifications documenting the equivalent training that qualifies their personnel to perform visual inspection on structures in the State of Wisconsin. This option must be approved by the DOT's Statewide Program Manager for approval prior to performing any work.

State, County, and Local Units of Government

A. NDE Requirements

State, county, and local units of government performing NDE (MT, PT, UT, etc.) shall be certified under the State of Wisconsin DOT's written practice or submit their written practice and personnel certifications for review prior to performing NDE (MT, PT, UT, etc.).

State, county, and local units of government performing NDE (visual inspection) on bridges or ancillary structures as defined in the *Wisconsin Structure Inspection Manual* shall:

1. have taken and passed the NHI 80-hour course (Safety Inspection of In-service Bridges) or NHI 1-week In-Service Bridge Training for PE's or

2. be certified as an American Welding Society Certified Welding Inspector (AWS/CWI) or

3. Submit their written practice and certifications documenting the equivalent training qualifying their personnel to perform visual inspections on structures within their jurisdiction. This option must be approved by the Statewide Program Manager for approval prior to performing any work.

<u>Movable Structures</u>

Movable structures function with large complex pieces of machinery, hydraulic, and electrical equipment. Each part of a movable structure has a relationship to, and must interact with, many other parts. In order for a movable structure to operate efficiently and to provide a long and serviceable life, all parts of the structure must be in alignment and receive proper maintenance. Proper maintenance requires that all functional systems (including electrical, mechanical, hydraulic, and structural elements related to the machinery) should be inspected and evaluated by personnel who are sufficiently experienced in that particular discipline.

The personnel who perform inspection of movable structures are required to have additional experience and training beyond just that for structural inspections. The inspection team may include separate structural, electrical, mechanical, and hydraulic Lead Inspectors. A single individual may serve as Inspection Team Leader and/or Lead Inspector in more than one of

these disciplines if the individual possesses the necessary level of expertise. It is necessary that each Lead Inspector have the experience in their specific discipline to meet those qualifications comparable with the Inspection Team Leader for structural inspections. However, an experienced tradesman or engineer with a background in electricity or machinery may qualify as a Lead Inspector so long as their qualifications are adequate for the type of structure and approved by the Inspection Team Leader or Inspection Program Manager. Each Lead Inspector must supervise and monitor the work performed by anyone assisting in the efforts of their discipline. Likewise, the Inspection Team Leader must supervise and coordinate the Lead Inspectors and all team members. More specifics are documented in Part 3 of this manual.

1.2.4 Official Bridge Files

The Official Bridge File for documentation on highway bridges within Wisconsin is the Highway Structures Information System (HSIS). HSIS is supplemented with other documents located in the bridge owner's office (WisDOT's central and regional, county highway departments, or municipalities). The official copy of inspection documentation produced on or after April 1, 2016, is stored in HSIS. Inspection documentation in HSIS dated before April 1, 2016 are copies of the official inspection report originating from the bridge owner's office. Other historical documentation may be stored in either HSIS or the bridge owner's office.

The National Bridge Inspection Standards require documents/components of the bridge files must be maintained for as long as the bridge exists. Whenever possible, documents should be stored electronically in HSIS.

Documents stored in bridge owner's office must be saved in a designated accessible Bridge File. The Bridge File can be a folder in a file drawer in the bridge owner's office or an electronic copy stored on a bridge owner's electronic storage system. Electronic files can be scanned copies of paper documents or an original electronic file. The Bridge File must be maintained/saved, either paper or electronic, for as long as the bridge exists.

Section 2 of *AASHTO's Manual for Bridge Evaluation (MBE)* identifies a wide range of components that should make up the bridge file. A list of applicable significant bridge file components, which is a subset of the list in the MBE, is identified in the *NBIS Metrics for the Oversight of the National Bridge Inspection Program,* Metric #15: Inspection procedures – Bridge File:

a. Inspection reports

b. Waterway information – channel cross-sections, soundings, stream profiles

c. Special inspection procedures and requirements for nonredundant steel tension member (NSTM), complex, in-depth, and underwater dive inspections

d. Load rating documentation

e. Posting documentation

f. Critical findings and actions taken

g. Scour assessment

h. Scour Plan of Action (POA)

i. Inventory and evaluation data

j. Significant correspondence which includes agreements

k. Maintenance records

Components of the official bridge file located in HSIS include:

a. Copies of inspection reports from 2004 to April 1, 2016.

b. Official inspection reports from April 1, 2016 to present stored electronically.

c. Structure plans, where available (original construction, as-built, repairs, rehabilitation)

d. Structure and hydraulic design computations from 2004 to present

e. Nonredundant steel tension member (NSTM) diagram

f. Critical Findings and responses

g. Load ratings and bridge posting documentation

h. Scour plan of action (POA)

i. Inventory data

j. Special inspection procedures

Components that generally remain in the local offices include:

a. Inspection reports prior to 2004

b. Structure and Hydraulic design computations where available

c. Historical waterway information where available

d. Limited significant correspondences (ownership, inspection & maintenance responsibility)

e. Scour assessment documentation, where not part of the structure design computations

For premanufactured culvert type structures, include all the following that are available in HSIS:

- As-built plans
    - Plan/profile sheets

- Construction detail sheets

- Special provisions that apply to the structure
- Manufacturers product data sheets
- Shop drawings
- Calculations

Highway Structure Information System (HSIS) is maintained by the Statewide Structure Management Engineer for all bridges and ancillary structures in Wisconsin. This database system consists of element level data, an electronic directory of supplemental information files (approved format such as .pdf, .doc, etc.), and inventory data utilized to create the National Bridge Inventory (NBI) File sent annually to FHWA. The data and files in this system are maintained by the regional and county (local) inspection Program Managers. Currently FHWA requires that all states maintain an NBI file that contains bridge inventory and inspection data in the format described in the current edition of the FHWA Specifications for the National Bridge Inventory (SNBI) for all bridges in the state. FHWA requires all States to collect and submit element data as provided by the current edition of the *AASHTO Manual for Bridge Element Inspection.*

Each year, the Statewide Structure Management Engineer uses inventory and inspection data in HSIS to create the NBI file. The Statewide Structure Management Engineer is the primary contact for data submittal and coordination with the FHWA, WisDOT regions, and counties.

1.2.5 WisDOT Bridge Inspection Quality Contol Program

1.2.5.1 Purpose

The NBIS Metrics for Oversight of the National Bridge Inspection Program, Metric #20: Inspection Procedures – QC/QA states:

1. Systematic quality control (QC) and quality assurance (QA) procedures are used to maintain a high degree of accuracy and consistency in the inspection program.

2. QC/QA procedures include periodic field review of inspection teams, periodic refresher training requirements, and independent review of inspection reports and computations.

The accuracy and consistency of inspection data is crucial to the entire bridge management operation as it lays the foundation for any bridge management systems. The data gathered through the Bridge Inspection Program aids in making decisions for needed maintenance, repair, rehabilitation, and replacement as well as improving design for new bridges. Furthermore, quality inspection and documentation practices will help to uphold the WISDOT commitment to the safety of the traveling public.

In order to maintain the accuracy and consistency of structure inspections and structure inspection reports, inspection programs are required to have appropriate quality control and quality assurance (QC/QA) measures. Typical QC procedures will include a centralized qualification list for inspectors, the online data management software for bridge information

(HSIS), and Quality Control Best Practices to provide a review or validation process. Furthermore, WisDOT BOS, each region, and each county shall conduct QC practices of their inspection program as outlined in this section.

Private bridge owners are not subject to the NBIS and do not fall under WisDOT oversight. They are encouraged but not required to perform inspections that comply with the NBIS. Railroad structures fall under the oversight of the Federal Railroad Administration.

The purpose of the Quality Control Program is to ensure uniformity and consistency in inspector training and qualification, completeness and accuracy of inspection reports and structure data, and timeliness in adherence to inspection interval requirements.

1.2.5.2 Components

The Quality Control Program within WisDOT's Bridge Inspection Program has numerous components, which cover the requirements WisDOT has established for both region and county programs.

1. Training Courses

   WisDOT hosts several training courses for inspectors to learn new information or keep up-to-date with current practices. Inspectors from the state, region, and county level, as well as consultants, all have access to these trainings that are put on by WisDOT, as needed.

2. Inspector Qualification Records

   WisDOT's BOS maintains a system which houses the qualification records for all current and past inspectors that inspect within the State of Wisconsin. This record includes training completion certificates, experience histories, letters of recommendation, and licenses.

3. Data Management System

   The Highway Structures Information System (HSIS) is the online database that contains bridge inventory and inspection information along with construction history and pertinent bridge related documents (such as bridge plans). The software is capable of performing data checks as well as running queries of its data to aid in achieving compliance for inspection interval metrics.

4. Inspection Operations

   Inspection programs shall have a process in place to ensure that inspectors are sufficiently equipped to properly inspect all necessary structures on time. While this process can vary in form, its function is vital to maintaining an accurate bridge management system.

5. Inspection Interval Compliance Procedure

   WisDOT has a procedure in place for actions to be taken by the Regional Program Managers and BOS as it relates to the inspection entry timeline.

6. Inspection Meetings

Inspection meetings on a regular basis can be utilized to discuss emerging issues or concerns with the inspection program or as a refresher of the inspection program.

7. Quality Control Best Practices

   Inspection programs shall utilize some form of inspection report review and validation in order to ensure quality and completeness. Methods for accomplishing this component can vary from program to program.

8. Quality Control DT Form

   Every Program will be required to have on-file a standardized Quality Control form (WisDOT DT2002). This form will detail the program's staff, regular bridge meetings, review methods, and bridges (listed by inspection type).

Each component highlighted above will be discussed in further detail in the following sections.

1.2.5.3 Training Courses

There are several training courses offered by WisDOT that are required for inspectors as part of the qualification process to perform certain inspections. While WisDOT may offer these courses in order to maintain a sufficient pool of qualified inspectors, it remains the inspector's responsibility to ascertain the necessary training. A listing of future courses offered by WisDOT can be found on the Maintenance & Inspection Training & Tools webpage (http://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/strct/inspection-training.aspx). The courses typically offered by WisDOT are described below.

Safety Inspection of In-Service Bridges
This is a National Highway Institute (NHI) standard course 130055 lasting 2-weeks that is required for all Inspection Team Leaders and Program Managers. This comprehensive course, based on the 2012 FHWA "Bridge Inspector's Reference Manual (BIRM)", covers AASHTO's element-level inspection approach as well as critical finding procedures. This training course is historically hosted by WisDOT when there is enough interested participants. Individuals looking to take this training should coordinate with WisDOT to learn about options that exist in any given year. A modified version of this course NHI 130056 is also offered for Professional Engineers and only takes 1-week instead of the 2-week option.

WisDOT Inspector Refresher Training
Wisconsin certified bridge inspectors are required to complete a cumulative total of 18 hours of FHWA approved bridge inspection refresher training over each 60-months period. Copies of completion certificates must be emailed to WisDOT by emailing to Structure-Inspection@dot.wi.gov.

Additionally, Wisconsin certified bridge inspectors are required to complete all WisDOT bridge inspection program-based refresher training. The method for an inspector to complete WisDOT refresher training will depend on method of training delivery and the subject of the training – in person, webinar, or review of electronic media.

Bridge Inspection Techniques for Nonredundant Steel Tension Members (NSTM)

Inspectors that wish to complete NSTM Inspections in the state of Wisconsin are required to complete this NHI standard course. Inspectors will be taught the concepts of NSTM Member identification, failure mechanisms, fatigue, Non-Destructive Evaluation (NDE) methods, and documenting inspection procedures. WisDOT offers this course as needed.

Underwater Bridge Inspection
Inspectors that wish to complete Underwater Dive Inspections in the state of Wisconsin are required to complete this NHI standard course. Inspectors will be taught the methods of underwater inspection, underwater material deterioration mechanisms, deterioration inspection techniques, and scour inspection techniques. WisDOT offers this course as needed.

Other Specialized or Field Specific Training
WisDOT may offer additional training in specific fields or for specialized tasks, as needed.

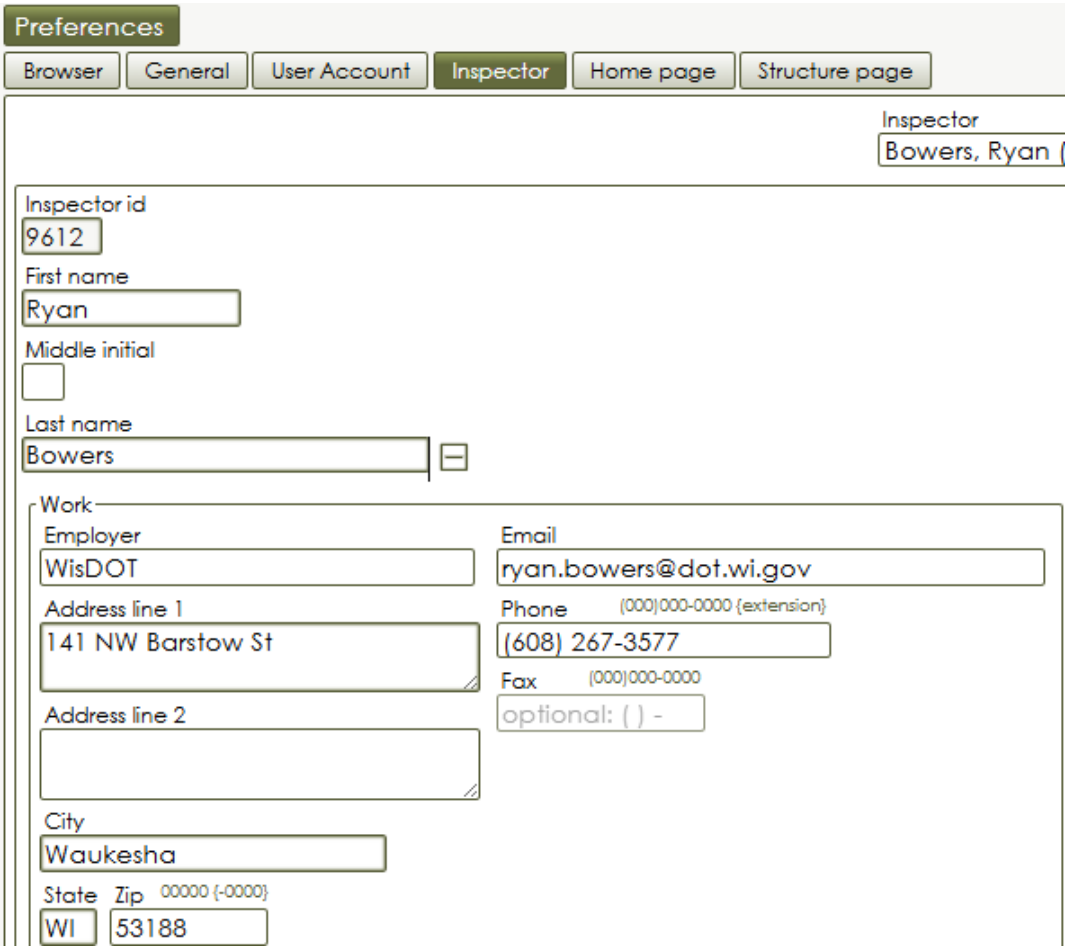
A complete listing of all NHI courses can be found on their website: www.nhi.fhwa.dot.gov

1.2.5.4 Inspector Registry

The National Bridge Inspection Standards (NBIS) requires state DOT's to maintain inspector contact information in a registry of state certified bridge inspectors. WisDOT maintains the list through the Highway Structures Information System (HSIS). All Wisconsin certified inspectors must provide and update the following information within HSIS to remain certified and be able to enter inspection data.

- Email address
- Employer name
- Work address
- Telephone number
- Additional information can be provide
    - Home address
    - Home telephone number

In HSIS, inspector contact information can be entered in HSIS on under **Preferences** and the **Inspector** tab as shown in the picture below.



An annual email will be sent out by the SPM to all certified inspectors to review and update inspector contact information in HSI.

1.2.5.5 Inspector Qualification Records

Inspector ID numbers allow an inspector to enter an inspection into the HSIS database and are granted only to qualified individuals. Those seeking to acquire such a number must first submit a Qualifications Record form (WisDOT DT2001). This form requires the applicants to verify their qualifications mainly through training course completion certificates, license numbers, experience histories, and letters of recommendation. See the Inspector Application and Credentials webpage for submitting this information and other requirements.

BOS will review the documentation provided and make a decision on whether to accept the applicant and assign them an Inspector ID number. Follow-up on the experience history may be performed. Successful applicants will be emailed a cover letter informing them of their acceptance and their unique Inspector ID number. Once an ID number is assigned, HSIS permissions is assigned to the ID number. More specifics on the permissions within HSIS will be discussed in the following section.

All of the documents associated with this process will be stored by BOS in its filing system. This system is to be maintained under the direction of the SPM.

Qualifications required for Team Leaders and Load Raters are summarized in Figure 1.2.3.2-1, which was presented earlier in this chapter.

1.2.5.6 Data Management

WisDOT utilizes its in-house software, the Highway Structures Information System (HSIS), as its tool for data management. HSIS performs many tasks in the Quality Control process, the more important of which will be enumerated below.

Inventory Data

HSIS houses the structure inventory data, construction history, structure plans, and rating files. All of this information is easily navigable through a tab hierarchy, as the software was designed to be used on a mobile platform.

Inspection Data and Checks

HSIS maintains an inspection history for each structure, starting in the late 1990s. Inspections are entered through the interface of the database within a standardized inspection form. Additionally, the system houses fields for inspection intervals associated with each inspection type required for a given structure. Most importantly, for QC, however, are the data checks performed by HSIS during inspection entry.

One check relates back to the assignment of an inspector ID. Once the new inspector ID has been assigned in HSIS, specific permissions will be assigned to the inspector ID. This will only allow the user to access certain fields and change data specific to a particular program. Furthermore, the permissions can be categorized by the training and special inspection qualifications associated with the individual inspector. This is to say that certain IDs will have permissions to enter Routine bridge inspections versus NSTM bridge inspections versus Underwater Dive bridge inspections. This feature ensures that only those individuals that have been deemed qualified by BOS bridge inspection staff are allowed to create a certain type of inspection.

Another check affects the ability of the inspector to complete the inspection being entered. HSIS will check to ensure that all necessary fields are filled with information prior to allowing inspection report completion. It will also check to make sure appropriate data rules are followed. Element quantity fields will even be checked for accuracy as they relate to the sum of defects associated with the given element. For example, material defect quantities cannot add up to more than the total quantity for that element.

Additionally, HSIS houses a series of over 50 data checks; all of which ensure that NBI, Element, SI&A, and Inspection data all conform to appropriate submission standards.

Automated Reports

Another important feature of HSIS is the ability to run queries of the inventory and inspection data. Over the years, certain queries have become more common and were written into the system as reports. These reports can be set up to automatically query data and email results to particular users on set intervals or upon changing of certain data fields. Three examples will be described below.

Advanced Planning is necessary to avoid inspections being completed past the due date. Bridge Owners and Inspection Program Managers are subscribed to a distribution list for bridge inspection scheduling. This report distributed automatically on the first of each month. This monthly scheduling report that is maintained by BOS staff queries for any bridges due for inspection within the next 4 to 6 months (depending on type of inspection) or

any bridges that are overdue for an inspection. The information is displayed chronologically by structure under STRUCTURE_ID column. The user can Filter and sort to display what they want. Under the STATUS column one can find the status of the inspection. A few important ones are listed below with definition:

- Past-Due - When a due date has past and an inspection has not been created in HSIS. HSIS has no current inspection report started. This designation signifies an inspection hasn't been entered or has yet to be performed (and will be late).
- Missing – Structure is flagged requiring this type of inspection, but no inspection of that type exists. As an example, newly constructed bridges that are open to public traffic but have no routine inspection on file will be categorized with a "Missing" status.
- Late – The inspection has been completed in HSI and was performed after the due date.

Inspection Program Managers can also subscribe to a report which will send them an email upon completion of any inspection of a structure within their jurisdiction whose NBI Component value drops to a 4 or less. This tool allows for Program Managers to get an alert when structure condition is classified as poor in order to review inspection data and validate the inspectors concerns regarding condition. This can be setup by the user in HSIS under go-tools-sytem-subscriptions tab.

Load Rating Engineers within BOS are subscribed to a report for structures that may need to be rerated. This report queries for any structures that have had the rerate flag checked, have significant changes to NBI Component values, or have a material defect in Condition State 4 for certain primary elements. There is a section within the HSIS interface that allows Load Rating Engineers to communicate rerate findings back to the inspector.

1.2.5.7 Inspection Operations

Inspection Operations are the cornerstones for a successful and efficient inspection program. These operations include scheduling, preparation, procedure, and submission.

Scheduling is a programmatic approach to dividing up structure inspections over time and between inspectors. This approach is unique for each inspection program and plays an important role in establishing a good Quality Control Program.

Preparation is a set of efforts put forth prior to an inspection. Pre-inspection preparations should at least include the following:

- Assess equipment needs, traffic control needs, and access requirements
- Coordinate equipment reservations (such as Under Bridge Inspection Vehicles) and any necessary traffic control
- Review prior inspection reports, maintenance history, and bridge plans for background knowledge on the structure
- Delegate inspection duties amongst members of the inspection team, if applicable
- Ensure that inspection vehicle is properly stocked with appropriate equipment and reference manuals
- As a safety precaution, make someone else aware of the inspection plan should you be inspecting alone

Inspection procedure is an established protocol to be followed while on-site. Aspects of this operation include the following:

- Proper safety equipment is utilized by all inspection personnel
- Appropriate inspection tools and measuring devices are used
- Proper selection, documentation, and use of access equipment (including Under Bridge Inspection Vehicles)
- Maintain safe operating limits and work practices
- Ensure that established inspection procedures are being followed, especially for special inspections
- Ensure that all areas with defects/deterioration are appropriately documented, including any pertinent measurements

1.2.5.8 Inspection Submittal

Submission of inspection data must be entered, and the report completed in a timely fashion to ensure the required inspections have been performed. Inspectors shall create the inspection report in HSIS within 28 days after the month in which the field portion of the inspection is completed. The minimum information to create an inspection in HSIS is *Date, Team Leader, Inspection Type and Activity Types*. For example, any inspection performed in the month of July shall be created in HSIS no later than August 28th. The remainder of the inspection data must be entered, and the report completed in HSIS as soon as possible but within 3 months after the month when the field portion of the inspection is completed. WisDOT encourages programs to utilize tablet devices as an aid for a timelier entry of inspection reports.

Bridge Inspection Interval Late Tolerance
The inspection date is the last day of the field portion of the inspection and is expected to be completed during the month the inspection is due. Field inspections completed after the month in which it is due are considered late by WisDOT. The program manager for a late inspection or anticipated to be late inspection must provide email notification and an expected completion date to the next level inspection program authority as soon as possible. County and Local Inspectors shall notify the County PM who will notify the Region PM. State inspectors are to notify the Region PM who will notify BOS Structures Inspection Unit Supervisor. The Region PM will enter a grace date in HSIS for all inspections utilizing the late tolerance for the anticipated completion date.

An acceptable interval tolerance for late inspections is allowed by the NBIS which WisDOT has adopted. An inspection interval falling outside of the acceptable tolerances is considered noncompliant which results in the inspection program being noncompliant. The acceptable interval tolerances are as follows:

Acceptable Inspection Interval Tolerances	
Existing interval	Interval Tolerance
< 24 months	2 months *
≥ 24 months	3 months *

* After the month in which the inspection was due.

Inspections that are expected to exceed the tolerances require FHWA approval 3 months in advance of the inspection due date and must be documented in HSI.

Inspections that exceed the acceptable tolerances are rare and are reserved for unusual circumstances. These include but are not limited to natural disasters.

1.2.5.9 Inspection Interval Compliance Procedure

The Inspection Interval Compliance Procedure was established in an effort to ensure timely inspections and adherence to FHWA Metrics 6 through 10. This procedure occurs on a monthly cycle (around the first of the month) and lays out actions to be taken for past-due inspections. The expectation is that an inspection will be performed on the correct interval and entered in a timely fashion.

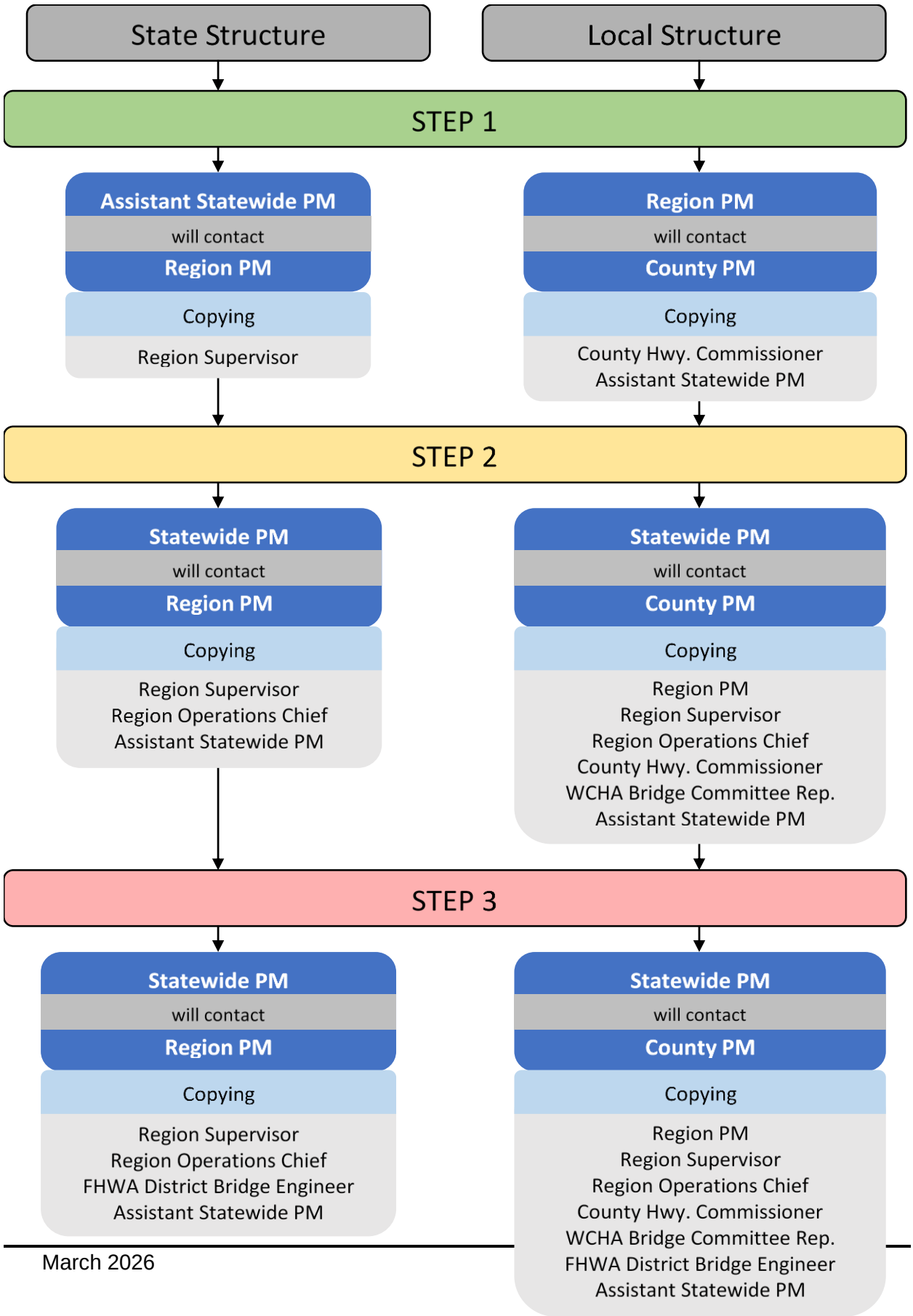
Past-due inspections are inspections that have exceeded the date of next required inspection and are not yet created in HSIS. For example, an inspection is two months past-due if it was required to be inspected in July and nothing is entered into HSIS as of September 1st.

The Inspection Interval Compliance Procedure is composed of three steps, with each subsequent step becoming more urgent. The level of urgency at each step is described below. After the step descriptions, there is a flowchart summarizing the contact tree for this procedure.

Step 1: Occurs for any inspection showing up as 2 months past-due. This step is an informal check-in in the form of either an email or a call asking about the completeness of the inspections in question. If the inspections are indeed complete, this information shall be passed on to WisDOT BOS staff. If the inspections in question are not yet complete, the need to perform the inspection should be enumerated.

Step 2: Occurs for any inspections showing up as 3 months past-due. This step will be a little more formal in that the contact will be documented with an email to be saved on file with WisDOT BOS. For State-owned structures, WisDOT will offer alternatives available in order to get the inspection complete should the Region not have the capabilities to do so. For Local-owned structures, WisDOT will remind the County of their responsibility, under WI Statute 84.17, to perform the inspections in question.

Step 3: Occurs for any inspections showing up as 4 months past-due. In order to maintain compliance for Metrics 6 through 10, WisDOT must not let an inspection be inspected over 4 months past its required inspection date. Therefore, this step is the final and most urgent step. For State-owned structures, should the Region not have the capability to do so, WisDOT BOS will cause it to be performed by the end of the 4th month. For Local-owned structures, WisDOT will send a Directive Letter to the County, copying Wisconsin County Highways Association (WCHA), citing WI Admin. Code Trans 212, directing the inspections in question to be performed.


State Structure
Local Structure
STEP 1
Assistant Statewide PM
will contact
Region PM
Copying
Region Supervisor
Region PM
will contact
County PM
Copying
County Hwy. Commissioner
Assistant Statewide PM
STEP 2
Statewide PM
will contact
Region PM
Copying
Region Supervisor
Region Operations Chief
Assistant Statewide PM
Statewide PM
will contact
County PM
Copying
Region PM
Region Supervisor
Region Operations Chief
County Hwy. Commissioner
WCHA Bridge Committee Rep.
Assistant Statewide PM
STEP 3
Statewide PM
will contact
Region PM
Copying
Region Supervisor
Region Operations Chief
FHWA District Bridge Engineer
Assistant Statewide PM
Statewide PM
will contact
County PM
Copying
Region PM
Region Supervisor
Region Operations Chief
County Hwy. Commissioner
WCHA Bridge Committee Rep.
FHWA District Bridge Engineer
Assistant Statewide PM

1.2.5.8 Inspection Meetings

WisDOT encourages each program to have inspection meetings, as necessary to keep up-to-date on current issues and practices within the inspection program. These meetings can consist of Bridge Owners, Program Managers, and Inspection Team Leaders gathering to discuss the state of either the Bridge Program or the Inspection Program. Alternatively, or complementary to these meetings, some Regions may host an inspection season refresher in the early spring which serves as a reminder of inspection requirements and practices for Bridge Owners and Program Managers. Lastly, WisDOT may send out an Update Seminar for inspectors to reaffirm some of the Wisconsin-specific practices as well as to disseminate new program policies. These forums offer great opportunities for involved parties to ask questions and maintain uniform approaches for inspection.

1.2.5.9 Quality Control Best Practices

Quality Control Best Practices are defined as activities which satisfy the "independent review of inspection reports" section within the criteria language for FHWA's Metric #20. The intent of these Best Practices is to have each program internally provide some level of review as to the completeness and accuracy of information being submitted on inspection reports. WisDOT provides several Best Practice options (outlined below), but it is the responsibility of each program to utilize their own form of inspection report review and/or validation.

Collaborative Inspections

A Collaborative Inspection is defined as an inspection performed in a team of 2 or more in which all inspectors are working together in performing the inspection of the structure. This also implies that each inspector will be evaluating the structure in its entirety and not that each inspector only inspects a portion of the structure. While the latter method can lower inspection times, it does not qualify as a Quality Control Best Practice. Collaborative Inspections include a Team Leader working alongside any individual with at least one of the following qualifications: Regional Program Manager, County Program Manager, another Team Leader, Team Member, or Bridge Owner. WisDOT's Under Bridge Access Inspection Staff can qualify towards this Best Practice.

Quality Control Inspections

A Quality Control Inspection is defined as an inspection performed by a Team Leader under the supervision of any individual with at least one of the following qualifications: Regional Program Manager, County Program Manager, or another Team Leader. This Best Practice differs from Collaborative Inspections in that the additional inspector is present simply to observe the practices and process of the Team Leader. They need not participate directly in the inspection of the structure, but should provide input on the acceptability of the Team Leader's inspection methods and results. WisDOT's Under Bridge Access Inspection Staff can qualify towards this Best Practice.

Inspector Rotation

Inspector Rotation is accomplished by alternating Team Leaders each inspection cycle for all or a portion of the program's inventory. This format allows for several different sets of eyes to view the structure during its lifespan. The additional sets of eyes will provide quality

checks on Inventory Data, Elements, Defects, and Component Ratings. This method is predicated upon the assumption that the same inspector does not inspect the same structure in consecutive cycles.

Independent Entry

Independent Entry is defined as having another individual enter an inspection for the Team Leader that performed the inspection. The individual entering the inspection must have at least passed the NHI "Safety Inspection of In-Service Bridges" course and must not have participated on the same inspection he/she is entering. This Best Practice is targeted at having a fresh set of eyes review the elements and data present on the report. Elements should make sense for the structure type being inspected and low NBI Component Ratings should be supplemented with sufficient documentation.

Inspection Report Review

Inspection Report Review is defined as an independent review of an inspection report by a Bridge Owner or Program Manager. The Program Manager reviewing an inspection report for which he/she was the Team Leader will not qualify under this Best Practice. Data to be reviewed includes but is not limited to: Inventory Data, Elements, Defects and Quantities, NBI Component Ratings, Documentation/Justifications, and Maintenance Actions. Inspection reports should also be reviewed for completeness. This review only signifies that the Bridge Owner or Program Manager has reviewed the inspection report for the above aspects. It does not, under any circumstance, signify that the Bridge Owner or Program Manager assumes responsibility for the accuracy or thoroughness of either the field inspection itself, the assessment of the structure's condition, or the documentation of the structure's condition.

Each program must have some form of review and validation of inspection reports, whether it is one of the Best Practices outlined above or another method that has worked in their program. While every inspection report does not need to be reviewed under a Best Practice, the program shall review enough reports during a cycle to ascertain that inspection reports being submitted are of sufficient quality.

1.2.5.10 Quality Control DT Form

The Quality Control DT2002 Form is a standardized form that lays out the scope and responsibilities of County and Region Inspection Programs. WisDOT requires that this form be updated and resubmitted to BOS no later than January 15th on the year the program is due for a Quality Assurance Review. The reviewing agency will inform the program by December 1st of the prior year if they are required to complete this document. Additionally, the form shall be updated and resubmitted when a new County Highway Commissioner or Program Manager is hired. Each program is responsible to keep this document-up-to-date as it should be a living document. Sections of the form are listed and described below.

Inspection Program Staff

Inspection Program Staff includes Bridge Owners, Highway Commissioners, Program Managers, Team Leaders, and Team Members that are directly involved in the inspection program. These individuals are the human resource component available to maintain the inspection program. The minimum amount of information needed in this section includes the individual's name, their role within the program, and their Inspector ID number, if applicable. If

consultants are hired by the program to fill a Program Manager role, the DT form shall be accompanied by a document detailing the scope of services that the consultant will provide to fulfill that role.

Inspection Meetings

Any bridge related inspection meetings, as detailed in section 1.2.5.8, can be documented under this section. These meetings highlight the programs dedication to maintain current and consistent inspection practices. Documentation should include an approximate interval for each meeting listed.

Inspection Scheduling Method

Each program will document the general method they use to schedule structure inspections, including the following criteria:

- when inspections are typically performed (general months and years)
- how and why inspections are delegated to specific inspectors

Quality Control Best Practices

Each program will document the methods they have historically used over the last inspection cycle for satisfying the "independent review of inspection reports" clause in the Metric #20 language. There are entries available for the program to write in methods not detailed as Best Practices in section 1.2.5.9. Each Best Practice documented on the form shall be accompanied by either a percentage of the inventory reviewed, the number of inspections reviewed, or the number of days spent performing reviews.

Bridges within Program Jurisdiction

Bridge lists will be attached to the DT form to help define the scope of each program. Note that these lists capture the program at a certain point in time and any future references should understand that the lists may not be final or complete. Counties and Regions should still maintain their own separate lists of structures that should be up-to-date at all times. One list shall be provided for each of the following inspection or activity types:

- 24-month Routine
- 12-month Routine
- NSTM
- Underwater Dive
- Underwater Profile
- In-Depth
- Movable
- Scour Critical

The minimum amount of information needed on these lists includes Structure ID and inspection interval. To help better identify bridges WisDOT recommends including columns for Feature On, Feature Under, Owner, and Municipality. Bridges may very likely show up on multiple lists.

This form will serve as an aid not only in the Quality Assurance Process but also in detailing roles and responsibilities to Owners and Program Managers.

1.2.6 WisDOT Bridge Inspection Quality Assurance Program

1.2.6.1 Purpose

The NBIS Metrics for Oversight of the National Bridge Inspection Program, Metric #20: Inspection Procedures – QC/QA states:

1. Systematic quality control (QC) and quality assurance (QA) procedures are used to maintain a high degree of accuracy and consistency in the inspection program.

2. QC/QA procedures include periodic field review of inspection teams, periodic refresher training requirements, and independent review of inspection reports and computations.

In order to maintain the accuracy and consistency of structure inspections and structure inspection reports, structure owners should implement appropriate quality control and quality assurance (QC/QA) measures. Typical QA procedures will include a field review performed by an independent agent and various performance metrics monitored by the WISDOT.

The purpose of the Quality Assurance Program is to monitor the quality of the inspection program and to evaluate the need for possible adjustments to individual Quality Control Programs.

1.2.6.2 Components

The Quality Assurance Program within WisDOT's Bridge Inspection Program has numerous components, which cover the oversight responsibilities assigned to WisDOT's BOS and its regions. WisDOT's BOS Bridge Inspection Unit Supervisors will lead the Quality Assurance Program.

1. Quality Assurance Field Reviews
   WisDOT utilizes various types of field reviews to assess the quality of inspection personnel, processes, and reports. These reviews are comprised of a sampling of completed inspections and may vary in scope, scale, interval, and selection criteria. Where prudent, load rating reviews will be performed.

2. Quality Assurance DT Form
   Every Quality Assurance Field Review will be documented using a standardized quality assurance form (WisDOT DT2003). This form will include the type of field review, the adequacy of quality control documentation, a narrative on subject's performance, and detailed analysis of inspection report data.

3. Field Review Summary Report
   Quality Assurance Field Reviews within each year will be summarized in a concise report that documents the trends of current inspection practices while highlighting any potential opportunities for improvement.

4. Disqualification and Requalification Procedures

Inspectors that exhibit unethical behavior or fail to uphold the requirements of WisDOT's Bridge Inspection Program may be subject to a disqualification/requalification procedure.

5. WisDOT Quality Assurance Metrics
The Highway Structures Information System (HSIS) is WisDOT's online bridge information database and it has the capability to run queries of its data to aid in the creation of certain performance metrics. These metrics are aimed at guiding future WisDOT Policy.

6. Quarterly Inspection Performance Measures on Inspection Compliance
Specialized query reports within HSIS allow WisDOT to monitor compliance to inspection interval requirements on a quarterly basis.

Each component mentioned above will be discussed in further detail in the following sections.

1.2.6.3Quality Assurance Field Reviews

Quality Assurance Field Reviews occur separately from the normal inspection process and are carried out by an independent team (as defined below). These field reviews shall be performed on a cycle such that each program is reviewed every fourth year. Further details of field reviews are described below.

Scope
Field reviews are an evaluation of the structure and its condition as it relates to the most recent Routine inspection report. Field reviews are not meant to substitute or constitute an inspection of any type and should not be treated as a full-scale inspection. The main purpose of the Quality Assurance Field Reviews is to assure that correct inventory data was captured, appropriate elements and assessments were documented, and justification via pictures and/or diagrams was provided for NBI component values. While the review does not require defect quantities to be calculated, a check of such values shall be performed to determine applicability to the structure's condition. Another goal for the field reviews, in assessing quality, is to assure that documented NBI component values are within 1 of the review panel's rating and that documented overburden measurements are within 1" of what is measured during the review.
Each field review shall consist of an on-site inspection review of typically between 2 and 6 structures. Further items to review are as follows:

Fixed Bridges:
1. Review QC DT form for completeness and accuracy.
2. Check qualification of inspectors.
3. Review office bridge file documentation for organization, content and location, see form DT1729 Bridge File Documentation.
4. Check bridge overburden values are within 1" of actual measurement during the review.
5. Review recommended bridge maintenance items are appropriate and completed maintenance/repairs are documented.
6. Review UW probe form filled out completely.
7. Review structures requiring UW dive inspections.

8. Review UW profiles are completed per program requirements.
9. Review Scour Plan of Actions (POA's) for completeness and accuracy paying special attention to triggers required for monitoring and closing the bridge during flood events. Review a minimum of 1 structure per review.
10. Review Scour Appraisal documentation (scour calcs, evaluations/assessment forms, summary of scour evaluation, information on plans, etc.).

Movable bridges: On an annual basis perform a minimum of 2 QC office reviews and a minimum of 1 QA field reviews of moveable bridges statewide.

Items to review:

1. Check for accuracy and qualifications of inspectors
2. Review DT forms are completed properly filled out
3. QA review of inspection report similar to a fixed bridge QA review
4. Check files and look for supporting documents for conditions
5. Look for documentation process for severe finding, what triggers process and implementation
6. Safety, inspection and structure procedures defined including Tag Out procedure
7. Review list of contacts for emergency or maintenance issues
8. Review maintenance history, process and scheduling.

Types of Field Reviews

Field reviews are categorized into one of four types: County, Region, Supplemental, or Federal.

County Program Reviews are led by the WisDOT's Bridge Inspection Unit Supervisor. The Regional Program Manager is responsible to establish a 4-year cycle in which each county within their jurisdiction shall be reviewed. The review should be performed by a team of at least two participants, although it may be performed solely by the Unit Supervisor. These participants can be any combination of the following parties: Bridge Inspection Unit Supervisor, Regional Program Manager, Regional Inspection Staff, County Program Manager, other BOS Staff, or FHWA Division Bridge Engineer.

Region Program Reviews are led by WisDOT's Bridge Inspection Unit Supervisor and are scheduled such that each program will be reviewed on a 4-year cycle. The review shall be performed by a team of at least three participants. These participants can be any combination of the following parties : Bridge Inspection Unit Supervisor, other BOS Staff, FHWA Division Bridge Engineer, Regional Program Manager, or Regional Inspection Staff.

Supplemental Reviews are led by WisDOT BOS Staff and will be performed as needed. These reviews provide a more focused approach to assuring quality by targeting areas of concern discovered by WisDOT, regardless of structure ownership. Additionally, these reviews may involve office data reviews and/or field reviews.

Federal Reviews are led by the FHWA Division Bridge Engineer. These reviews are typically performed every year and generally involve the review of any publicly-owned bridges within select regions of the state. These reviews tend to rotate regions on a 4-year cycle, similar to Region Program Reviews. The main difference to highlight is that WisDOT acts as a separate and independent agent from FHWA's Metric Review Program. WisDOT's presence is solely in an auxiliary capacity to aid the FHWA Division Bridge Engineer on his/her review as an extra review panel member. This effort also affords WisDOT the opportunity to collect more quality assurance data to further assure Inspection Program quality.

Selection Criteria

Structures used for Field Reviews will be selected based on one or more of the following factors.

Condition

Structures in poorer condition afford the review team the opportunity to gauge an inspector's thoroughness and diligence during the last inspection. Age can also play a role within this criterion as it may be prudent to review a structure inspection of a new structure in which accelerated deterioration is documented. Similarly, it may be prudent to review a structure inspection of an old structure in which limited deterioration is documented.

Large Drops in NBI Component Values

Bridges generally age gradually over time and it is therefore concerning when a structure's component rating for Deck, Superstructure, Substructure, or Culvert drops rapidly between inspections. This criterion would allow WisDOT the opportunity to become informed of any accelerated deterioration.

Lack of Documentation/Justification

Inspection reports with lower NBI component ratings should have sufficient documentation to justify the ratings. Reports without sufficient documentation (as determined by a Program Manager) may be subject to review.

Inspectors

Inspectors that perform an excessive number of inspections in a single day may not have utilized proper inspection practices or performed their inspection duties to the appropriate extent and may therefore be subject to review. Likewise, any inspectors that perform very few inspections per cycle may not be as proficient with Wisconsin's inspection practices and could be subject to review.

Random

Bridges selected for review may be selected on a completely random or a selectively random basis. Selectively random could limit the randomization to a pool of structures that was filtered to a certain region of the state, certain superstructure types, particular inspectors or firms, or structures along an easily traversable path. Completely random would not be limited by any filters and would pull from any publicly-owned structure in the State of Wisconsin.

Input from Bridge Owner/Program Manager

WisDOT is willing to accommodate recommendations from Bridge Owners or Program Managers if they have concern about how future maintenance should be handled for the structure or if they have questions about how to document specific elements of the structure. Additionally, if Bridge Owners or Program Managers have concern about a specific inspector or program, WisDOT will give consideration to reviewing inspections performed by said inspector.

1.2.6.4 Quality Assurance DT Form

The Quality Assurance DT Form is a standardized form that serves as a location to document findings from both the review of the Quality Control DT Form and the Quality Assurance Field Reviews. It will also act as a To-Do list for any Follow-up Actions required from the field reviews. Sections of the Quality Assurance DT Form are detailed below.

Quality Control DT Form Check
In this short section, the completeness and adequacy of the program's QC DT Form will be documented. Commentary on any deficient areas is required. The reviewer shall then sign their name to confirm that the QC DT Form was reviewed in its entirety. Further details on what goes into a check of the QC DT Form will be given later, in section 1.2.6.5.

Program's Performance
Once the field reviews have been completed, this summary section shall be filled out to enumerate findings within the categories of Commendable Practices, Room for Improvement, and Follow-up Actions. Commendable Practices are those aspects of the reviewed inspection program that yield quality work commensurate with or in excess of WisDOT's expectations. Room for Improvement is any portion of the reviewed inspection program that needs to be enhanced to meet WisDOT's expectation for quality. Follow-up Actions are any items that need to be addressed within the near future regarding findings from the field reviews.

Field Reviews
Several blocks are set aside for a detailed analysis of each inspection report that was field reviewed. Each block contains entries for Bridge ID, miscellaneous notes, required follow-up actions, elements, and NBI component values, among other things.

This completed form provides the layout for any required Follow-up Actions. Typical Follow-up Actions include the submission of miscellaneous inspection documents, diagrams, or pictures, and a load rating check. WisDOT's Load Rating Staff will review the load rating files for all structures that are field reviewed.

Upon completion of all Follow-up Actions, but no later than December 31st of that year, the QA DT Form shall be distributed to WisDOT BOS and the agency that was subject to review.

1.2.6.5 Example Schedules for Various QA Reviews

Quality Assurance Reviews of County or Region Programs shall begin with an assessment of the program's Quality Control DT Form. Because this DT 2002 Form is required to be submitted by February 1st for all programs due for QA Review within the upcoming year, and because the review of this form can take place strictly in an office setting, WisDOT recommends that the QC DT Form be reviewed the last few months of the previous year for planning purposes. This practice would allow a portion of the QA Review to occur outside of the typically busy inspection months. During the review of the QC DT Form, the reviewer should check several pieces of information. First, the reviewer should be ensuring that all documented Team Leaders are indeed qualified through WisDOT to perform inspections. Next, a comprehensive cross-check of the given bridge lists with HSIS will be performed. If the lists provided differ from the lists in HSIS, then the reviewer shall follow up with the program to correct any inconsistencies. Once corrections are agreed upon, the updated lists, if applicable, shall be emailed to WisDOT BOS. Lastly, the form shall be checked for completeness. Once the review of the QC DT Form is finished, the reviewer shall fill out the appropriate section of the Quality Assurance DT Form and sign his/her name.

The examples highlighted below may be used as a template for how WisDOT anticipates Quality Assurance Field Reviews to be scheduled.

County Program Reviews can take place in a single day. The BOS Unit Supervisor and/or the Regional Bridge Program Manager shall coordinate with the County Program Manager, County Highway Commissioner, and BOS Staff (if applicable) in order to find a date that will work for all involved parties to spend the day visiting bridges in the field. Once a date is agreed upon, the Unit Supervisor should then select several structures to field review using the selection criteria laid out in 1.2.6.3.

Region Program Reviews can take place in a single day, but may require BOS Staff to travel the day prior to the review. The BOS Unit Supervisor shall coordinate with the Regional Program Manager and the FHWA Division Bridge Engineer (if applicable) in order to find a date that will work for all involved parties. Once a date is agreed upon, the Unit Supervisor should then select structures to field review using the selection criteria laid out in 1.2.6.3. Further thought should be given to planning progression of field reviews along a path as it relates to each Regional Program in order to limit the amount of travel time for all parties.

Supplemental Reviews are scheduled as necessary. They may involve office/data type reviews or field reviews by BOS Staff. BOS Staff may or may not coordinate with Program Managers dependent on the review content. Program Managers may participate in the review if deemed necessary to clarify review results.
Federal Reviews are scheduled entirely by the FHWA Division Bridge Engineer. WisDOT's goal is to make time to participate alongside FHWA for the entirety of these reviews. WisDOT participants may include the Statewide Inspection Program Manager, Assistant Inspection Program Manager, Unit Supervisor, Region Program Manager, Local Program Managers, or other Team Leaders of the inspections being reviewed.

1.2.6.6 Field Review Summary Report

Upon completion of the year's Quality Assurance Field Reviews (of all types), BOS Staff will summarize trends and findings in a concise report. This report will be used to highlight current inspection practices and, if applicable, opportunities for improvement to the Bridge Inspection Program. BOS will look provide FHWA with a copy of this report by January 31st of the following year.

1.2.6.7 Disqualification/Requalification Procedures

Inspectors that fail to uphold the requirements for a quality inspector as established by WisDOT may be subject to the disqualification/requalification process. There are three different arms of this process and each one is described below.

Failure to Maintain Qualification
Should an inspector fail to attend required refresher trainings or fail to maintain their registration as a Professional Engineer, as applicable, WisDOT's Statewide Program Manager may disqualify this individual from performing inspections within the State of Wisconsin. These individuals may re-qualify as Wisconsin inspectors by simply completing the necessary training or updating any professional certifications, where required for Team Lead status.

Inspection Quality
Any inspector that shows a significant lack in inspection quality may be subject to disqualification via a review committee. Examples of inadequate inspection quality include but are not limited to: fraud, misconduct, falsification, other unethical behavior, and repeated failure to correct previous mistakes. The review committee shall be comprised of the following 3 individuals, all with voting powers:

- WisDOT Statewide Program Manager
- WisDOT Assistant Statewide Program Manager
- and a Region Program Manager from the same region as the inspector being reviewed or BOS Maintenance Unit Supervisor

In addition to the 3-member review committee, non-voting members may be selected by the committee to participate in the review or may choose to participate voluntarily. This includes any of the following:

- FHWA Division Bridge Engineer
- Region Program Manager(s)
-
- County Program Manager(s)
- WCHA Representative

Determinations within the committee will be decided by majority vote. After WisDOT investigates and compiles concerning evidence, the first action of this process is for the Statewide PM to notify the subject inspector that he/she is currently under review and for what reason. The subject inspector will have the opportunity to provide his/her defense to the review committee. The review committee shall then be assembled to determine

whether or not the offense warrants disqualification or permanent removal. The review committee is permitted to permanently disallow an inspector from inspecting in the State of Wisconsin at any step in this process. If the inspector is disqualified, then the review committee notifies the subject inspector, and the requalification process begins. It will be the disqualified inspector's responsibility to create a plan enumerating corrective steps to be taken to re-establish and prove inspection competency. The review committee will meet for a second time to review this proposal, make any desired changes, and return to the inspector. The last action of the review committee will be one final meeting to determine the outcome of the inspector's plan for improvement. At this point the committee will either grant re-qualification, establish an auxiliary re-qualification procedure, or permanently disallow the inspector from inspecting in the State of Wisconsin. The timeline for the disqualification/requalification procedure involving the review committee will be established on a case-by-case basis, not to exceed one year from the date of notification by the Statewide PM.

Inspection Timeliness
Any inspector or program manager that fails to maintain timely inspection practices may be subject to disqualification via a review committee. Examples include but are not limited to: failure to quickly address critical findings, failure to properly inspect structures within prescribed intervals, and failure to enter inspection reports in a timely manner. The review committee and requalification process are the same for this offense as it is for the prior offense.

1.2.6.8 WisDOT Quality Assurance Metrics

WisDOT utilizes its in-house software, the Highway Structures Information System (HSIS), as its tool for data management. HSIS is capable of running complex queries of the data to aid in producing several performance metrics used by WisDOT. Some of the metrics that WisDOT looks at in regards to inspection quality are described below.

Excessive Inspection Counts
WisDOT becomes concerned with inspection quality levels when an excessive number of inspections are performed in a single day. At the end of an inspection year, BOS Staff creates a metric on the number of inspections performed as part of an excessive inspection count. Also, WisDOT will track which inspectors are performing these excessive inspections. These inspectors may be subject to a Supplemental Field Review.

Average Inspection Time
WisDOT will look at inspection times as a tool to assess inspection quality for a given structure.

Time to Create/Complete
One important, and often overlooked, aspect of quality is timeliness. HSIS has the ability to produce data regarding the amount of time it takes inspectors to perform the final documentation aspects of the inspection process. After performing an inspection, inspectors will start by creating the inspection in HSIS. This lets HSIS know that an inspection was indeed performed and will allow the inspector to then enter the inspection

information into the database. Once an inspector enters all the information and determines that it is complete to his/her best knowledge, he/she will then digitally sign and submit the inspection report. WisDOT is gathering information on how long it takes for inspectors to first create the inspection after performing it then to complete it after creating it. WisDOT may then take this information to evaluate which inspectors, if any, may be subject to disciplinary actions as laid out in 1.2.6.7.

1.2.6.9 Quarterly Performance Measures on Inspection Compliance

In order to monitor adherence to inspection interval requirements as laid out by the National Bridge Inspection Standards (NBIS), WisDOT BOS Staff utilizes reports from HSIS. These reports query inspection data pertaining to inspection date, type, interval, and late reason in order to establish whether or not a structure's inspection is in compliance. These reports are used on a quarterly basis as intermediate measures to gauge inspection interval compliance as it relates to State and Federal bridge inspection requirements. These quarterly reports allow WisDOT to see trends that arise during an inspection season and afford the Bridge Inspection Program the opportunity to preemptively correct deficiencies that will be captured on future FHWA Metric Reviews. This effort is an attempt to strengthen WisDOT's Inspection Program in advance of federal requirements for corrective action or improvement.

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1.3 TYPES OF BRIDGE INSPECTIONS AND ASSESSMENTS

1.3.1 Introduction

There are numerous types of bridge inspections. Each inspection type has been designed to obtain specific information from a structure. For instance, when a structure is built, an Initial Inspection is done to document the as-built condition of the structure and its structural elements. This is the baseline inspection, and all future inspection findings are compared to this information. Routine Inspections are performed at regular intervals to monitor the working condition of structure elements. This is the most common type of inspection. The results from a Routine Inspection are used to assess structure safety and structure maintenance needs. Special Inspections are used to monitor known defects in a structure and assess completed repairs.

All of the inspection types that are used by the Wisconsin Department of Transportation (WisDOT) help to create a complete picture of a structure's condition and are described in detail in this chapter.

1.3.2 Initial Inspection

1.3.2.1 Purpose

CFR 650.305 states: *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.*

The initial inspection is the first inspection of a new, replaced, or rehabilitated bridge. The initial inspection serves to record required inventory data and elements, establish baseline conditions, and establish the intervals for other inspection types and activities. The initial inspection is a fully documented inspection, using the structure plans, to determine inventory data about the structure for entry into HSIS. In addition, Nonredundant Steel Tension Member (NSTM) or Underwater Dive inspections that are needed must be performed as soon as practicable, but within 12 months of the bridge being open to traffic. Performing repairs or preservation activities does not prompt a need to perform an initial inspection but may require a Special inspection – see Section 1.3.10.3.

The initial inspection should be completed as soon as practical, preferably before the structure or portion of the structure opens to traffic, but no later than 3 months after opening to traffic. Completing prior to opening to traffic allows for an inspection under more convenient circumstances for both the inspector and the travelling public. This will also assist in completing a punch list for the construction project.

The following is provided to assist with determining if an initial inspection is to be completed.

1. An initial inspection is to be completed whenever any primary structure work concept is completed which includes:

    - Replace Structure
    - Replace Deck

- Replace Superstructure
- Bridge Widening
- Raise Structure
- Replace/Eliminate Joints
- Replace Bearings
- Repair Superstructure – Restore Condition & Capacity
- Repair Substructure - Restore Condition & Capacity
- Overlay Deck - Concrete
- Overlay Deck - PPC
- Structural Reinforced Concrete Overlay
- Culvert Extension

2. A completed new bridge that is not open to public traffic is not subject to the initial inspection requirement until opened to traffic since it is technically closed.
3. Bridges under construction, closed to traffic or under phased/staged construction, require an initial inspection to be completed within 3 months for any portion of a bridge that opens or reopens to traffic.

   a. Projects with many phases or rapid progression through phases (e.g., nightly or weekend closures), it may not be feasible to complete an initial inspection every time a portion of a bridge opens to traffic. Up to 3 months of construction work may occur and multiple phases might have elapsed before the initial inspection is required. Those portions of the new bridge open to traffic and the bridge members/elements that constitute or carry vehicular live load path are to be inspected.

   b. An existing bridge being rehabilitated must continue to receive regularly scheduled inspections for any portions open to traffic.

      If during construction, a required inspection cannot be conducted due to extenuating circumstances such as a hazardous project site, contact the Statewide Program Manager in advance of when the inspection is due. Preapproval by FHWA is required in advance of the inspection due date plus the tolerance*. Approvals, communications, and circumstances related to the delayed inspection must be documented in the bridge file.

      * Tolerance is 3 months for ≥ 24 month interval and 2 months for < 24 month interval.

   c. Consideration should be given to including provisions in construction contracting documents that require the contractor to perform any NBIS inspections due while the construction project is underway.

      Open to Traffic: A bridge open to traffic during rehabilitation must continue to have regular scheduled inspections on all portions open to traffic.

      Refer to Section 1.1.14 for more inspection information about closed bridges.

Temporary bridges for highway traffic must have an NBIS inspection completed prior to opening to traffic (WisDOT's Standard Specifications for Highway and Structure Construction Section 526). A temporary bridge open to traffic greater than 24 months must be inspected and inventoried at 24 months per the NBIS and data entered in HSIS. A bridge ID for a temporary bridge open greater than 24 months must be assigned per

Chapter 2.5 of the WisDOT Bridge Manual. Upon removal of the temporary bridge, the bridge ID should be marked as REPLACED in HSIS.

Private bridges connected by a public road on both ends must be inventoried and inspected within 3 months of being identified. A bridge ID must be assigned per Chapter 2.5 of the WisDOT Bridge Manual Chapter.

Newly found bridge must have an initial inspection completed within 3 months of the bridge number assigned by WisDOT. The region PM will assign a number once the structure has been confirmed to be a bridge by a certified bridge inspector.

1. An initial inspection include the following actions which are to be completed by the inspection team: Collect structure inventory data required by the SNBI and WisDOT policy.

2. Determine, quantify, and rate new elements and assessments.

3. Review and update existing elements, assessments, and SNBI data items; and code condition ratings when necessary.

4. Determine the appropriate inspection types, activities, and intervals required for the structure and update HSIS based on the guidance provided in the SIM.

5. Photographs and sketches, as required and necessary to detail the initial condition of the bridge.

6. Upon completion of the inspection, notify the responsible PM and construction project manager of the inspection findings including recommended corrections (punch list) and summary of missing file documents.

The initial inspection TL must verify the following information exists or is referenced in HSIS:

1. An existing set of plans, as-builts, shop drawings, and design computations for the bridge.

2. Hydraulic and scour computations and scour vulnerability.

3. An analytical determination of load capacity (performed by a qualified Wisconsin professional engineer).

4. All structure inventory data required by the SNBI and WisDOT policy.

5. Other relevant information required by the department to maintain an accurate bridge file.

6. Baseline structural conditions and element quantities.

7. Record description and location of any construction issues, nonconforming materials allowed to remain, or existing problems in the structure.

8. Location and condition of any NSTM, SRM, and fatigue details.

9. Structure specific inspection procedures, when necessary.

10. Underwater channel profiles at the upstream and downstream fascia (for structures over water).

11. Using the profile information, assessment of the need for future underwater dive inspections.

Figure 1.3.2.1-1: Inspector Performing an Initial Inspection.

1.3.2.2 Precision

The Initial Inspection is required to document "as-built" **conditions**, not whether the structure was constructed per Plans and Specifications. Since this is a baseline inspection, all deficiencies, cracks, construction errors, alignment problems, etc. should be quantified and documented. The compiled documentation will be used during future inspections to determine if defects discover in the future existed when the structure was constructed or if they have materialized from the loading applied to the structure.

Inherent Defects

The initial inspection may identify inherent defects which are a characteristic of the material or results from normal construction practices - it is not indicative of damage or deterioration. An inherent defect by itself would not result in a maintenance, preservation, or repair action. In general, describe the inherent defect including the location and extent on the inspection report in the narrative section under the element defect that closely represents the inherent defect and rate as CS1. Some examples of inherent defects are listed below. This is not an all-inclusive list.

1. Poorly consolidated concrete (honey combing) with penetration depth <=1" without exposed rebar.
2. Chert pop-outs, generally <=1" depth or <=3" width.
3. Snowplow/vehicle scrapes/gouges that does not expose coarse aggregate.
4. Grinding of the wearing surface which does not create a loss of friction.

5. Patched areas on the top flanges of prestressed girders from construction framing hangers.
6. Patched holes in the deck surface from parapet forming practices.
7. Patched area in deck or abutment from joint replacement.
8. Rust stains from rebar chairs or forming ties.

Defects that are not inherent to normal construction practices or material characteristics but can be attributed to poor construction practices or poor material qualities are those that would likely lead to future maintenance or preservation actions. Those defects are to be rated for the element in the appropriate CS. Some typical defects resulting from poor construction practices are listed below.

1. Poorly consolidate concrete with penetration depth >1" or with exposed rebar (Delamination/Spall = CS3).
2. Small spalls on the top flanges of prestressed girders from construction framing hangers that are not patched.
3. Damage to girders from handling or the chain tie-downs to the trailers.
4. Dents in the top flanges of girders from deck removals (distortion defect).
5. Chert pop-outs, generally >1” depth or >3” width (Delamination/Spall = CS2).
6. Tight cracks (state size) in prestressed members as delivered or as initially installed. IE end cracks or top flange cracks in wide flange girders.
7. Grinding of the wearing surface which results in loss of friction.

1.3.2.3 Inspection Interval

Each newly constructed or rehabilitated structure shall receive an Initial Inspection as soon as practical but within 3 months of opening the structure to traffic. An Initial Inspection must be completed before the construction contract is finalized.

Below is a list of required bridge inspections conducted after the initial inspection.

Inspection or Activity Interval		
Type	**Maximum Inspection Interval (Months)**	
	Low Risk	**High Risk**
Routine	**24**	**12 (2)**
	48 (1)	
In-Depth	**Varies (48 to 72) (3)**	
Floor Beam identified as SRM, In-Depth	**96**	**24**
NSTM	**24**	**12**

UW-Dive	60 72 (4)	24
UW-Profile	96	Varies (24 to 60) (5)
SI&A Review	48	
Scour POA	48	

1. Structure must qualify per section 1.3.3.3
2. Includes bridges that are closed; see section 1.3.12 for more information
3. Interval is dependent on structural detail per section 1.3.4.3
4. Structure must qualify per section 1.3.7.3
5. Dive inspected bridges are on a 60-month cycle per section 1.3.6.1

1.3.3 Routine Inspection

1.3.3.1 Purpose

CFR 650.305 defines a *Routine Inspection as: 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.*

1.3.3.2 Precision

Routine inspections are generally conducted from the deck, ground, or water level or from permanent work platforms and walkways. If any element of the bridge appears to be distressed, the inspector should take a closer look at that element. Critical load-carrying members (*e.g.,* steel and concrete girders, decks, slabs, concrete box girders, piers and bents, bearings, abutments) should be closely monitored. Fatigue prone details or elements should receive a detailed, close-up (arm's length) inspection. See Section 3.6, NSTM Inspection and Part 2 Appendix D, Fatigue Prone Details for typical critical details. Inspection of underwater portions of the substructure is limited to observations during low-flow periods, probing for signs of undermining, and streambed profile measurements where applicable. This is further discussed in Section 3.9 All inspection results should be fully documented on the report form stored in HSIS.

Any changed or deteriorated structural conditions that might affect previously recorded bridge load ratings should be noted on the inspection form. If the inspector determines that a new load rating is warranted, the "load rating" box should be marked with a "Y". The owner is responsible for rating the structure. That owner is also responsible for getting the updated structure data into the appropriate structure file(s) located in the HSI. The HSI electronic file is the Official Bridge File.

Public safety is the primary concern of the structure inspector. Therefore, the inspector should make sure Wisconsin's structures are physically safe as well as structurally stable. Special attention should be paid to the condition of parapets, railings, pedestrian fencing, guardrail,

sidewalks, spalling concrete surfaces, and other safety related members. The following are examples of conditions that may warrant documentation and notification of the Local, County or Region Manager:

1. trip hazards, severe approach settlement, or large spalls on sidewalks;
2. rebar protruding from decks, walks, or parapets;
3. loose, missing, or damaged railings or parapets;
4. missing or damaged guardrail;
5. loose concrete that could fall onto a traveled way (road, walk, bike path, waterway, or rail line); and
6. any other condition that the inspector perceives as a threat to public safety.

The inspector may have several documents available to assist him in organizing and performing the inspection. These documents include: as-built plans, original shop drawings, as-built repair plans and shop drawings, and previous inspection reports.

1.3.3.3 Routine Inspection Interval

Routine inspections are performed at regular intervals not to exceed (NTE) 24 months. **For high risk bridges the regular interval shall not exceed 12 months.**

High Risk Bridges meeting any of the following criteria must be inspected at intervals NTE 12 months:

- Any of the following condition ratings is coded 4 or less:
  - B.C.01 Deck Condition Rating
  - B.C.02 Superstructure Condition Rating
  - B.C.03 Substructure Condition Rating
  - B.C.04 Culvert Condition Rating
  - B.C.11 Scour Condition Rating
- Bridge is posted less than 40 ton based on load rating.

The routine inspection interval may be reduced by the county or region inspection program manager based on other factors such as element condition ratings, scour environment, history of vehicle impact damage, other known deficiencies, or inspector recommendation.

Bridges with Deck, Superstructure, Substructure, Culvert, or Scour Condition Rating coded four (4) or less due to localized deficiencies, a Special inspection – Isolated Activity may be completed in lieu of a Routine inspection as described in Section 1.3.10.2.

Low Risk Bridges meeting the following criteria are eligible for an extended regular interval of 48 months:

- NBI Deck, Super, Sub, Culvert ≥ 6 or N.

- Must have an initial or routine inspection and another routine inspection 24 months or more apart.
- Span Type – all slab types, all girder types (except thru girder, girder/floorbeam), all box girder - multiple, arch under fill without spandrel, 3 and 4 sided frames only, T-beam, and pipe.
- Span Material – Concrete or Steel
- Inventory Load Rating Factor ≥ 1.0
- Routine Permit Loads = A or N.
- Fatigue Details (E & E') = None
- Cannot have steel defect 1010 Cracking.
- Cannot be load posted.
- Routing Permit Load item = A (routine permit loads are not restricted) or N (no permit loads allowed)
- Cannot have Element 161 Steel Pin, Pin & Hanger Assemblies or Pin thru Web.
- Border bridges with adjoining states are eligible with adjoining state agreement.
- Not a NSTM (fracture critical) bridge
- Hwy Min Vertical Clearance ≥ 14.0' (on or under)
- Scour Vulnerability = A or B (stable for scour)
- Scour Condition Rating ≥ 6
- Channel Condition ≥ 6
- Channel Protection Condition ≥ 6
- Must be less than 50 years old.
- No bridges with complex features

The routine inspection interval may be kept at the standard 24-month interval by the county or region inspection program manager based on other factors such as element condition ratings, scour environment, history of vehicle impact damage, other known deficiencies, or inspector recommendation.

For Steel twin-tub girder bridges that were analyzed and found to be no longer NSTM but system redundant member (SRM) bridges shall have a maximum required inspection Interval of 48 months for the interior of the tub girders. The rest of the bridge is to follow the eligible routine inspection Interval. For more information on SRM's see the Redundant Members section in this chapter.

The 48-month extended Intervals is optional and is up to the structures owner to utilize. Local agencies are required to be rated at a level of Compliance or Substantial Compliance for each of the FHWA 23 metrics to be eligible to utilize the extended Interval provisions.

Failure to rate at these levels for any of the 23 metrics will require the local agency to write a Plan of Corrective Action (PCA) to correct the deficiencies. The plan must be approved by the Statewide Inspection Program Manager and Regional Program Manager and the deficiencies must be corrected within the timeline agreed to in the PCA to maintain eligibility.

The structures owner interested in participating in the 48-month extended intervals needs to have the County PM or Commissioner fill out the DT2002 Structure Inspection Quality Control Form and follow the instructions enclosed.

1.3.4 In-Depth Inspection

1.3.4.1 Purpose

An In-Depth Inspection is 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. This higher-level inspection can be performed on any structure type, though it is commonly performed on steel superstructure bridges with problematic details that need close up evaluation (as discussed below).

Each bridge requiring in-depth inspections must have documented structure specific inspection procedures in the bridge file developed in accordance with Section 4.2 of the MBE. The procedures, whether general or specific, will include the following:

- A qualified inspection team leader must be present during the entire in-depth inspection.
- Identify each bridge member requiring an in-depth inspection and document each location.
- Detail visual or advanced inspection methods or techniques to be used to fully ascertain the existence of or extent of a deficiency not readily detectable using routine inspection procedures. In many situations, this may be a visual hands-on inspection.
- Review and update the in-depth inspection interval.
- Specify any additional qualifications, or specialized training, required of the inspection team leader or specific bridge inspectors (divers, riggers, NDE certified, etc.).
- Specify needed special access equipment or traffic control.

An In-Depth Inspection can be scheduled independently of a Routine Inspection, though generally at a longer interval, or it may be a follow-up for a Damage Inspection. Generally, specialized equipment is required to obtain the necessary hands-on access to the element (snooper trucks, scissor lifts, ladders, etc.).

It is important to correctly document in HSIS the inspection types and activities performed. It may be practical to perform other inspection types, such as the Routine inspection, concurrently with the In-Depth inspection. An activity such as Non-Destructive Evaluation (NDE) may be completed as part of the inspection. Make sure to select (check) the appropriate boxes in HSIS for all inspection types and activities completed.

For large and complex structures, In-Depth Inspections may be scheduled separately for defined segments of the bridge or for designated groups of elements, connections, or details that can be efficiently addressed by the same or similar inspection techniques. If the latter option is chosen, each defined bridge segment and/or each designated group of elements, connections, or details should be clearly identified in the structure specific inspection procedures and notes as a matter of record.

Moveable bridges have complex features requiring In-Depth inspections. The electrical and mechanical functional systems are part of the Complex Feature inspection of a movable bridge. See Part 3 for inspection requirements of movable bridges and Section 1.3.13 for inspection guidance for Complex Feature bridges.

The activities, procedures, and findings of In-Depth Inspections should be thoroughly documented with appropriate photographs, test results, measurements, and a written report in

the HSI system. In addition, access methods must be clearly documented so that future scheduling needs can be determined.

WisDOT requires an In-Depth inspection at a defined interval for the structures mentioned in Section 1.3.4.3. However, there are other conditions and/or structural details that may prompt an unscheduled In-Depth Inspection.

Several common conditions or structural details that could prompt an In-Depth Inspection (and possibly NDE) include:

- Apparent cracks in steel members
- Apparent cracks, de-bonding or loss of tendon section in a prestressed or post-tensioned member
- Heavily corroded or failed hold down devices.
- Severe section loss in a steel member or primary gusset plate
- Buckled or bent steel girders or beams.
- Welded cover plate end terminations
- Live load bearing anchor pins, and link-bars
- Field welds on tension members
- Intersecting welds, or category D, E, or E' details
- Unique or Problematic Details

The decision to conduct an unscheduled, In-Depth Inspection, with or without the use of NDE, is the responsibility of the Regional Program Manager. Items to consider when making this decision include (but are not limited to) ADT, Condition, Age and Location.

1.3.4.2 Precision

As indicated previously, an In-Depth Inspection is a visual, hands-on inspection of one or more structural elements. Each element under investigation should be within arm's length of the inspector. The inspection may include a recommendation for a load rating to rate the residual capacity of damaged or deteriorated members, depending on the extent of the damage or deterioration. Nondestructive load tests may be conducted to assist in determining a safe bridge load-carrying capacity. The inspector should exercise sound judgment in recommending when a load capacity analysis is warranted.

1.3.4.3 In-Depth Inspection Interval

In-Depth Inspections are **required** for the following:

- Bridges with pin thru web or pin & hanger assemblies (excluding NSTM Structures)

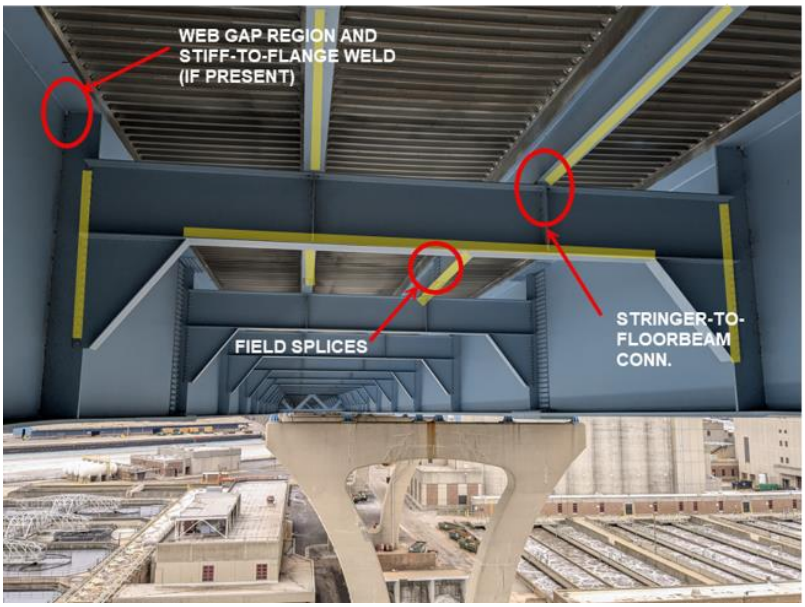
- Steel Bridges with floor systems (excluding NSTM Structures)
- 3 or 4 Chord Deck Trusses

Maximum <u>Required</u> In-Depth Inspection Interval:

Bridge Type (non-NSTM Bridges)	Requirement	Maximum Interval
Pin & Hanger Assemblies	Visual, Hands-on	72 Months
Steel Floor Systems (Floorbeam/Stringer)	Visual, Hands-on	96 Months
3 or 4 Chord Deck Trusses	Visual, Hands-on	48 Months

Members/Details of a floorbeam system bridge to be inspected hands-on during the required in-depth inspection:

1. Floorbeam bottom flange
2. Connections to girders especially web gap region and bolts
3. Stringer field splices and connections to floorbeams
4. Areas adjacent or below joints or floorbeams
5. Any other area that the inspector or PM deems necessary



Members/Details of a 3 or 4 chord truss bridge to be inspected hands-on during the required in-depth inspection:

1. Truss tension member, use design plan (rating staff can assist to identify if needed)

2. All vertical gusset plates
3. Any steel superstructure element in CS4
4. Any member/details found similar to the floorbeam system bridge
5. Any other area that the inspector or PM deems necessary

Other In-Depth inspections can be scheduled to supplement routine inspections, or as a follow-up to a special, initial, or damage inspections. But these will generally not have a recurrence interval.

For In-Depth inspections on structures with pins, a minimum of 20% of the pins shall be evaluated with NDE methods, including all components that have indications of cracking, distress, fretting rust, or seizing. The locations that have been evaluated shall be thoroughly documented, and efforts shall be made in subsequent in-depth inspections to vary the components being tested.

1.3.5 Nonredundant Steel Tension Member (Fracture Critical) Inspection

1.3.5.1 Purpose

CFR 650.305 defines a Nonredundant Steel Tension Member (NSTM) inspection as: *A hands-on inspection of a nonredundant steel tension member.* An NSTM inspection is a specific type of In-Depth inspection. However, since it is specifically called out in the Code of Federal Regulations, NSTM Inspections must be coded separately from In-Depth inspections on the National Bridge Inventory submittal and in HSI.

CFR 650.305 defines a nonredundant steel tension member (NSTM) as: 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.*

Common NSTM's include (but not limited to):

- Tie girders on tied arch bridges
- Tension chords or tension diagonals on trusses
- Tension flanges on non-redundant girders
- Tension flanges on non-redundant steel pier cap beams
- Pins on non-redundant girder/truss bridges
- Primary gusset plates connecting NSTM's
- Connecting points of NSTM's chords/diagonals

Primary gusset plates, regardless of connecting NSTM or compression elements, with continuing deterioration (active corrosion, distortion) are risk factors. The locations of these elements, as well as the amount of deterioration shall be measured and documented in the bridge inspection report. The Interval and methods of measurement shall be recorded in the inspection procedures.

NSTM Inspections are regularly scheduled inspections. NSTMs require more thorough and detailed inspections than the members of non-NSTM bridges. In recognition of this, Federal Regulation 23 CFR 650.313(g) requires bridge specific inspection procedures to be developed and documented for each NSTM bridge.

Figure 1.3.5.1-1: Inspectors Performing a NSTM Inspection

Bridges with NSTMs have written bridge specific inspection procedures and NSTM diagrams which clearly identify the location of all NSTMs, specify the Interval of inspection, describe specific risk factors unique to the bridge, and clearly detail inspection methods and equipment to be employed. Guidance for the specific inspection procedures is provided under Section 1.3.5.4.

An NSTM Inspection can be scheduled independently of a Routine Inspection, though it is common to schedule both during the same inspection. Generally, specialized equipment is required to obtain the necessary hands-on, arm's length access to the element (under bridge inspection vehicle, scissor lifts, ladders, etc.). The first NSTM inspection for a bridge or for a bridge with rehabilitated NSTMs must be completed as soon as possible but within 12 months of the bridge opening to traffic per CFR 650.313(f)(3).

On some bridges, it may be practical to include all elements of the structure during the inspection. In this case, both the Routine and NSTM (Fracture Critical (Arm's length)) boxes shall be checked in the Highway Structures Information (HSI) System when entering the inspection.

For some bridges, the NSTM Inspection may be scheduled and completed separately from the Routine Inspection. If this option is chosen, each defined NSTM shall be clearly identified in the inspection procedures and inspection specific notes as a matter of record.

The activities, procedures, and findings of NSTM Inspections must be thoroughly documented with appropriate photographs, test results, measurements, and a written report in the HSI system. In addition, access methods must be clearly documented so that future scheduling needs can be determined.

Floorbeams spaced greater than 14 feet apart shall have a hands-on inspection for the entire tension portion of the floorbeam. A hands-on inspection also applies to connections located in tension zones, such as the floorbeam connection(s) to the primary load carrying member and connections to secondary members such as stringer to floorbeam. These floorbeams will be inspected using the same techniques as NSTM. This shall be noted in the inspection procedures. The floorbeams shall be inspected at the NSTM inspection Interval for the bridge in question and be conducted by a certified NSTM inspector. When floorbeams spaced greater than 14 feet are shown to be non-critical through analysis (approved by Bureau of Structures), the hands-on inspection is not required and should be inspected with typical visual methods.

In rare cases where arms-length access cannot be safely accomplished by traditional methods (reach-all, ladder, etc.), alternative means of inspection are necessary. These methods require detailed inspection procedures that must be approved by BOS prior to use. Please coordinate with BOS Structures Maintenance prior to developing these procedures.

In addition, if the owner agency has demonstrated that floorbeam members spaced greater than 14 feet apart have system or internal redundancy based on an FHWA approved methodology per CFR 650.313(f)(1)(i), then NSTM procedures are not required for those members.

1.3.5.2 Precision

An NSTM Inspection is a hands-on inspection. CFR 650.305 defines a hands-on inspection as: *Inspection within arm's length of the member. Inspection uses visual techniques that may be supplemented by nondestructive evaluation techniques.* Every square foot of the member/member component is examined. The observations and/or measurements are used to determine the structural capacity of the member/member component, to identify any changes from previous NSTM Inspections, and to ensure that the structure continues to satisfy present safety and service requirements.

Under-bridge access equipment is typically required to move the inspector within arm's length of the members. There may be permanent work platforms and walkways available on some larger structures to aid in inspection work. The access methods used during the inspection must be documented in the inspection procedures.

1.3.5.3 Nonredundent Steel Tension Member Inspection Interval

Nonredundant steel tension members are inspected at regular intervals not to exceed 24-months except as required based on NSTM condition and inspection findings.

NSTM Bridges meeting the following criteria must in be inspected at intervals NTE 12 months:

- NSTM Inspection Condition State is coded ≤ 4

The inspection Interval must be identified in the written inspection procedures.

Maximum Required Inspection Interval:

Hands-on/visual ..... 24 months

Hands-on/visual when NSTM Bridge Inspection Condition Item is coded ≤ 4 ..... 12 months

Nondestructive Evaluation (NDE) ..... 72 months

Details to consider for NDE of NSTM Components include:

- Pin and hanger assemblies
- Live load bearing anchor pins and link bars
- Pin thru web assemblies
- Welded cover plate end terminations on NSTM's
- Field welds on NSTM's
- Intersecting welds, or category D, E, or E' details on NSTM's
- Unique or Problematic Details on NSTM's

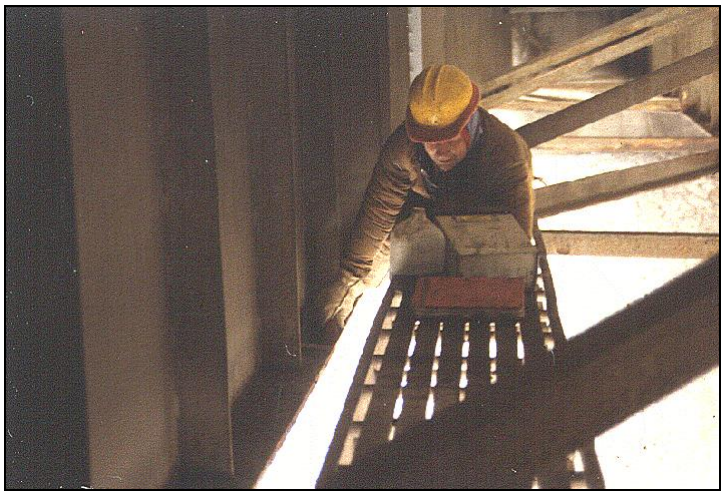


Figure 1.3.5.3-1: Inspector Performing Ultrasonic Testing for Crack Detection.

1.3.5.4 Specific Inspection Procedures

A bridge identified with a nonredundant steel tension member (NSTM) must have a detailed written inspection procedure specific to that bridge. These inspection procedures are to be kept in the bridge file, reviewed, and updated for **each** NSTM inspection. The inspection procedures should be written in a manner that is useful to the inspection team.

The inspection procedures must address any of the following areas that are relevant to the specific NSTM bridge.

- General Information –
  - Provide a general statement indicating the scope of the inspection
    - Include hands-on visual assessment of identified NSTMs
    - Identify problematic details.

  - Identify the method(s) to be used to complete the inspection.
  - Identify any other inspection types or activities that will be performed at the same time, for example, routine or NDT.
- Clearly specify the Interval of the NSTM hands-on visual inspection.
- NSTM Diagram which identifies the location of the nonredundant steel tension members, primary gusset plates, including any floorbeams needing hands-on inspection. The team lead or inspection program manager should verify the diagram is up to date. Form DT2011 may also be used to document the NSTM diagram.
- Workforce/Staffing
  - Staffing level – number of inspectors, team members
  - Staff qualifications needed
  - Define the duties to be performed by each team member or team of inspectors.
- Inspection Tools
  - Special tools not common to a routine inspection
  - Special lighting needs – if needed
  - Nondestructive testing (i.e. magnetic particle, a dye penetrant kit or ultrasonic testing device).
  - Method of access and equipment needed for hands-on inspection of each nonredundant steel tension member. The minimum size and location on the bridge the equipment will be needed.
    - Rope/rigging
    - Ladders
    - Scaffolding
    - Aerial work platforms
    - Under-bridge inspection truck
- Traffic control needs – on and under including any permits required.
    - Roadway
    - Pedestrian
    - Navigation
    - Railroad
- Scheduling – include any conflicts. Some common conflicts to address are as follows:
  - Daytime inspection times
  - Traffic congestion times
  - Known conflicts under the bridge
  - Railroad or navigation traffic
  - Availability of inspection staff
- Site conditions that impact the inspection
  - NSTM condition
  - Clean surface areas (as necessary) to allow for thorough visual inspection
  - Lighting required to improve visibility
  - Utility attachments
  - Environmental concerns
  - Railroads
  - Safety Concerns
    - Confined spaces
    - Traffic (on and under)
    - Homeless people
    - Night work
- Describe the inspection sequence to inspect the NSTM’s.
  - Planned sequence that prevents missing details.

  - Historic sequence.
- Contacts and/or situational awareness communication
  - State and local agencies (DNR, county hwy dept, local municipality)
  - Federal agencies (Coast Guard, FHWA, Army Corp, etc.)
  - Law enforcement
  - Emergency response
  - Adjacent property owners
  - Utility company
  - Media
  - WisDOT Region Communications Manager (WisDOT bridges)
- **Identify specific risk factors** that impact safety or serviceability and problematic details/locations affecting the NSTM. Some possible specific risk factors and problematic details are listed below:
  - Cover plates
  - Primary gusset plates with continuing deterioration (active corrosion or distortion). The locations of these elements, as well as the amount of deterioration shall be measured and documented in the inspection report.
  - Discontinuities resulting in stress risers
  - Bolted/riveted connections present
  - Fatigue and fracture prone details (category D, E and E' details)
  - Triaxial constraint
  - Problematic materials
  - Poor welding techniques
  - Intersecting welds
  - Tack/field/intermittent welds
  - Back-up bars
  - Out-of-plane distortion
  - Retrofits/repairs
  - Existing steel cracking
  - Steel section loss
  - Load posting
  - NSTM Condition item of 4 or less
  - Subject to oversized or overweight loads
  - Historic impact damage
  - Service life (>30 yrs)
  - High ADTT (>5,000)
  - NDT (MT, UT, PT, etc.) is required because of existing defect
  - Historically significant structure issue.
  - Pin/hanger or pin through web connections
  - Mechanical fasteners (bolts and rivets)
  - Other

NSTM Diagram included with the inspection report and procedures must clearly identify the location of all nonredundant steel tension members.

1.3.5.5 NSTM Supplemental Inspection Form

The nonredundant steel tension member (NSTM) inspection report must include a detailed supplemental inspection form which identifies and documents the condition of each NSTM

and AASHTO fatigue detail. The inspection report must provide qualitative and quantitative information concerning the NSTM and fatigue detail. This information is important for several reasons: it can offer insight about the condition of the NSTM, it can provide a history of the bridge, and it can be used to substantiate the thoroughness of the inspection effort. The supplement inspection form will be included with form DT2007 for the NSTM Inspection Report. Links to the supplemental inspection forms are below.

Links to supplemental NSTM inspection forms:

DT2010 (Word Document) & DT2011 (Word Document)

Supplemental NSTM Inspection Form A (Excel Spreadsheet)

Supplemental NSTM Inspection Form B (Excel Spreadsheet)

Supplemental Steel Girder-NSTM Inspection Form (Excel Spreadsheet)

1.3.5.6 Highway Structures Inventory System (HSIS)

The specific inspection procedure and the supplemental inspection form must be uploaded into HSIS on the *Documents/Images* tab. The procedures will be loaded under the *Category* of Inspection Procedures and the supplemental inspection form will be loaded under the Category of NSTM.

1.3.5.7 Redundant Members

Bridge members having documented engineering analysis and meeting the requirements provided herein may be classified as System Redundant Members (SRMs). Members must analytically demonstrate satisfactory redundancy through the application of one or more of the following:

1. AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members*. This includes the research and evaluation efforts by Purdue University applied to Wisconsin steel twin tub girders.

   * Prior to this publication, steel twin tub girders within the Marquette Interchange (Milwaukee, WI) were evaluated for system redundancy per NCHRP Report 406. These members received FHWA concurrence as what is now known as a SRM, and per the May 9th, 2022 FHWA memo, that analysis methodology is grandfathered in. However, the methodology for evaluating system redundancy per NCHRP Report 406 is no longer considered state-of-the-art, and future reevaluations of these structures, as required by subsection 1.3.5.7.1, shall adhere to the current AASHTO SRM Guide Specification.

2. Wisconsin Bridge Manual Chapter 24
   a. Section 14 Floor Systems (July 2025 release)
   b. Section 15 Box Girders

All bridge specific redundancy evaluation reports are stored in the electronic bridge files in HSIS. Reports identify the methodology used in redundancy

determination and identify the members that affect the system redundancy. WisDOT's implementation of inspection procedures for system redundant members (SRM) was approved by FHWA Wisconsin Division and the Office of Bridges and Structures on June 23, 2025. Below is a copy of the FHWA approval.

U.S. Department of Transportation
Federal Highway Administration

Memorandum

Subject: **ACTION:** Wisconsin – Procedures for System Redundant Members

Date: June 23, 2025

From: Shay K. Burrows, P.E.
Acting Director, Office of Bridges and Structures

SHAY KENNETH BURROWS
Digitally signed by SHAY KENNETH BURROWS
Date: 2025.06.23 11:05:38 -04'00'

In Reply Refer To: HIBS-1

To: Linda Swann
Acting Division Administrator
Madison, WI

We have reviewed your office's transmittal of the Wisconsin Department of Transportation's (WisDOT) request to implement inspection procedures for system redundant members (SRM), submitted in accordance with the May 9, 2022, FHWA memorandum titled Inspection of Nonredundant Steel Tension Members. Transmittal was in the form of the attached letter from Joshua Dietsche (WisDOT State Bridge Engineer) to Joe Balice (FHWA Wisconsin Division Bridge Engineer) on June 5, 2025.

Having reviewed and coordinated with the Wisconsin Division Office to resolve comments on the procedures attached to the WisDOT's request, the Office of Bridges and Structures concurs with its approval.

Please note that the Division Office is responsible for monitoring the implementation of the approved procedures as part of your annual National Bridge Inspection Standards (NBIS) review in accordance with the National Bridge Inspection Program Compliance Review Manual, including verification of the following WisDOT actions:

(1) Identification of bridges determined to have system redundancy that do not require Nonredundant Steel Tension Member inspection as outlined in the NBIS (23 CFR 650.313(f)(i)),
(2) Documentation of the finding of system redundancy in the bridge file, and associated inspection and condition criteria, and
(3) Application of the inspection and condition criteria as documented in the approved procedure.

If you or your staff have any questions, please contact Stephen Bartha, Acting Safety Inspection Team Leader at (857) 270-4250 or at stephen.bartha@dot.gov or Derek Soden, Structural Engineering Team Leader at (720) 413-7436 or at derek.soden@dot.gov.

Attachment

2

cc:
HIBS-40
Derek Soden, HIBS-10
Justin Ocel, HIBS-10
Joe Balice, Division Bridge Engineer, HDA-WI

Inspector Qualifications

SRM - Team Leader must have NSTM team leader qualification (Section 1.2.3.2) to inspect system redundant members.

Steel Tub Girders

Applicability

Steel twin tub girders that have demonstrated sufficient redundancy through analytical evaluation and meet the following baseline conditions may be classified as SRMs and are subject to the inspection requirements contained herein. All other steel twin tub girders are classified as NSTMs.

The initial baseline condition shall be established via a current NSTM inspection and shall verify the following:

a. Fatigue cracks have been effectively mitigated.

b. Primary load carrying members, which include exterior diaphragms, are in Condition State 3 or better.

c. Resolved any impact damage to primary load carrying members, such as stress risers (cracks, sharp reentrant corners) or large out-of-plane distortion.

Reevaluation Criteria

Subsequent inspections shall verify that the baseline conditions are still valid. If defects are found that violate these requirements, then a structural review is required to ensure the member still has system redundancy. An analysis based structural review per 1.3.21.2 must be completed under the direction of the WisDOT bridge load rating supervisor. Additional items that require a structural review are:

1. CS3 or CS4 distortion in the bottom flange or web of the tub girders in the negative moment region.

2. CS3 or CS4 distortion in the flanges or web of the exterior diaphragms.

3. Missing bolts in field splices or external diaphragm to girder connections.

4. SRM exposed to intense, prolonged, or high temperature fire, a review should be conducted to determine if the girder's structural integrity has been compromised. Consideration will include observed distortions, appearance of damaged paint, and fuel type. Consult with the Bureau of Structures – Structures Fabrication Unit for this determination.

5. The position or number of striped lanes has changed.

6. Change in deck overburden or other increase in dead load to the superstructure.

7. Removal of external diaphragms (not allowed).

The structural review may conclude the defect found does not affect the SRM determination without going through a full reevaluation of the structure. This shall be documented in the structural review and noted in the element defect per Structural Review process.

If a full reevaluation fails to confirm SRM status, the appropriate spans of the bridge will revert to a NSTM and require an NSTM inspection at the maximum interval per Section 1.3.5.3 starting with the next scheduled routine inspection. Those procedures are to be followed until a time at which the girder or external diaphragm are restored to its original baseline condition or strengthened through retrofitting and successfully reevaluated for SRM status.

For each initial, routine, in-depth, damage, and special inspection, the inspection team leader will complete the SRM Baseline Condition Report form, provided in Part 1 Appendix C for steel twin tub girders and uploaded into HSIS with the inspection. Primary structural members in CS4 will automatically trigger a structural review. If any of the items on the form are selected “Yes”, the inspector will request a structural review through HSIS when entering the inspection data. To request a structural review in HSIS, select the Action box on the Element tab for the SRM element, then select a Structural Review activity on the Edit tab.

Design and Construction Details

Modern steel tub girder bridges, specifically those in Wisconsin’s inventory that have an SRM status, have been designed and constructed with fatigue and fracture resistant details. Most bridges have been fabricated with high performance steels (HPS) which give the members improved impact energy. Those bridges with non-HPS were fabricated with A709 steel per a fracture control plan with zone 2 Charpy V-notch impact energy requirements. Newly constructed bridges need to follow the AASHTO guidance to be eligible.

Routine Inspection Requirements

Steel twin tub girders analyzed and found to be SRMs (no longer NSTM) receive a routine inspection per section 1.3.3.3. Document the access methods, focus areas and conditions to monitor in the bridge specific inspection procedures.

In-Depth Inspection Requirements

The in-depth inspection of steel twin tub girders classified as SRMs consists of a close-up detailed inspection of the interior and exterior of the tub girders, including the full depth exterior diaphragms. The in-depth inspection will serve to collect and document the existing condition of the interior and exterior of the steel tub girders in greater detail than a typical routine inspection. This may include a hands-on inspection at some locations which will be outlined in the bridge specific procedures for in-depth inspections. The inspection will include a close-up visual inspection by walking the full-length interior of the steel tub girder and by viewing or accessing the exterior of the steel tub girders, including the exterior full height diaphragms, using binoculars, drones, under-bridge inspection unit, or other access methods

sufficiently so the inspector is confident the bridge and element condition can be assessed. The in-depth inspection may be completed in conjunction with the routine inspection.

The maximum in-depth inspection interval of steel twin tub girders classified as SRMs is 48 months. However, in-depth inspection interval is the lesser of 24 months or the eligible routine interval when any of the following occurs on the interior of the tub girder:

- Steel cracking or distortion defect exists in CS2, 3, and 4.
- Corrosion or Connection defects exist in CS3 over 5% of total Steel Closed Web/Box Girder element length in any one span. *For example, a 150' twin-tub girder span could have a maximum of 5% x 300' =15 LF total combined CS3 corrosion or connection defect in that span before being subject to a reduced inspection interval.*

Special Inspection Requirements

Refer to the special inspection section 1.3.10.

Specific Inspection Procedures

The in-depth inspection for SRM bridges requires detailed written inspection procedures specific to the bridge. These inspection procedures are to be kept in the bridge file, reviewed, and updated for each in-depth inspection. Refer to Section 1.3.5.4 Specific Inspection Procedures for applicable information required in the redundant member specific inspection procedures. Additionally, address the following by identifying the areas of concern and risk factors within the inspection procedures:

- Full-depth diaphragms and their connections to the tub girders
- Specific inspection procedures for *sub*structure elements classified as SRMs. Note: Several tub girder structures classified as SRMs have either NSTM or SRM substructure elements.
- Regions, elements, components, and/or members that are critical to the alternative load path developed in an SRM analysis in the faulted condition that should be carefully inspected to ensure they can continue to function in that unintended role.
- Review section 1.3.5.4 for additional applicable risk factors

Steel Floorbeams

Applicability

Floorbeams spaced 14 feet or less shall be considered redundant members, subject to routine inspection requirements.

Floorbeams spaced greater than 14 feet, that have demonstrated sufficient redundancy through analytical evaluation and meet the following baseline conditions may be classified as SRM and are subject to the inspection requirements contained herein. All other floorbeams* not meeting these requirements are otherwise subject to an NSTM-level inspection.

The initial baseline condition shall be established via a current NSTM-level inspection and shall verify the following:

a. Out-of-plane distortion induced cracking in the web gap region (small area of web between the flange and vertical connection plate) has been effectively mitigated. If not mitigated, then the floorbeam itself may be considered an SRM based on analysis, but the connection to the main girder would require NSTM level inspections until a time in which the crack can be mitigated.

b. Fatigue cracks have been effectively mitigated.

c. Floorbeam in Condition State 3 or better.

d. Stringer(s) within the adjacent longitudinal bays in Condition State 3 or better.

e. Deck within the adjacent longitudinal bays is in Condition State 3 or better.

*The disqualification for SRM status also applies to those floorbeams longitudinally adjacent to the floorbeam in question. This is because the performance of those floorbeams will depend on the condition of the floorbeam in question.

Reevaluation Criteria

Subsequent inspections shall verify the baseline conditions are still valid. If defects are found that violate these requirements, then a structural review is required to ensure the member still has system redundancy. An analysis based structural review per 1.3.21.2 must be completed under the direction of the WisDOT bridge load rating supervisor.

Additional items that require a structural review are:

1. Stringer or floorbeam distortion is CS3 or CS3,

2. Missing bolts/rivets in stringer or floorbeam field splices,

3. The position or number of striped lanes has changed, or

4. Change in deck overburden or other increase in dead load to the superstructure.

The structural review may conclude the defect found does not affect the SRM determination without going through a full evaluation of the structure. This shall be documented in the structural review and noted in the element defect per Structural Review process.

If a full reevaluation fails to confirm SRM status, the affected floorbeam(s) will require NSTM inspection at the maximum interval per Section 1.3.5.3 starting with the next scheduled routine inspection. Those procedures are to be followed until a time at which the floorbeam is restored to its original baseline condition or strengthened through retrofitting and successfully reevaluated for SRM status. Note that the methodology to establish redundancy within the WisDOT Supplement to AASHTO SRM Guide Specification, section 8.2, considers an automatic 10% section loss in critical steel members to allow for minimal section loss up to a condition state 4.

For each routine, in-depth and special inspection, the inspection team leader will complete the SRM Baseline Condition Report for Floorbeams form, provided in Part 1, Appendix D, for the floor beams and uploaded into HSIS with the inspection. Primary structural members in CS4 will automatically trigger a structural review. If any of the times on the form are selected “Yes’, the inspector will request a structural review through HSIS when entering the inspection data. To request a structural review in HSIS, select the Action box on the Element tab for the SRM element, then select a Structural Review activity on the Edit tab.

Design and Construction Details

Details to be considered prior to evaluation as an SRM can be found in Section 2.2 of the SRM Guide Spec*. However, the details listed in the SRM Guide Spec are not required in a typical floorbeam.

*In conjunction with the SRM Guide Spec, floorbeam systems with the following details can be evaluated using the simplified methodology described in section 24.14 of the Wisconsin Bridge Manual:

1. Stringers are continuous over the floorbeams, and

2. Floorbeams are noncomposite with the deck.

Routine Inspection Requirements

Details to be considered Steel floorbeams analyzed and found to be a SRM (no longer NSTM) receive a routine inspection per Section 1.3.3.3. Document the access methods, focus areas and conditions to monitor in the bridge specific inspection procedures.

Special Inspection Requirements

The special inspection will serve to provide a better visual inspection on the SRM floorbeams. Refer to the Special Inspection Section 1.3.10. The use of drones may be used for a Special Inspection per the bridge specific inspection procedures at a recommended intermediate interval of 48 months.

In-Depth Inspection Requirements

The in-depth inspection of SRM floorbeams will serve to collect and document the existing condition of the floorbeam in greater detail than a routine inspection. The intervals will be per Section 1.3.4.3. The in-depth inspection may be completed in conjunction with the routine inspection.

Specific Inspection Procedures

The in-depth inspection for SRM floorbeams require detailed written inspection procedures specific to the bridge. These inspection procedures are to be kept in the bridge file, reviewed, and updated for each In-depth inspection. These procedures shall be in accordance with section 1.3.5.4, NSTM specific inspection procedures.

- Identify specific areas of concern/risk factors:
    - Regions, elements, components, and/or members that are critical to the alternative load path developed in an SRM analysis in the faulted condition that should be carefully inspected to ensure they can continue to function in that unintended role.

1.3.6 Underwater Profile Activity

1.3.6.1 Purpose

Underwater profile activity involves gathering streambed elevations and alignment information used to assess streambed conditions and monitor channel movement at bridges and bridge like structures over waterways. The data gathered is compared to as-built information and past profiles. The survey is in the form of an underwater profile or a hydrographic survey. The underwater profile consists of cross-sections of the streambed taken parallel to the bridge. At a minimum, profiles must be obtained at the upstream and downstream fascia. It is not required to profile around substructure units (as these are covered by the underwater probe and/or Dive Inspections), but this may be done during the profile activity as directed by the inspection team leader or the inspection program manager.

Adequate information on the stability of the waterway and an assessment of the risk to a bridge from scour can be achieved by gathering streambed elevations and alignment information through an underwater profile activity or hydrographic survey. Records must be maintained for a historical comparison to determine the extent of any scour, channel shifting, degradation, or aggradation of the channel.

Scour is the movement of channel bed material by the action of the moving water. This movement may result in degradation, or erosion, of material as well as aggradation, or accumulation of material. Degradation of the channel bed may lead to structure instability, posing an often-unseen threat to safety.

There are three forms of scour that can affect the safety of bridges and waterfront structures.

1. General scour is the general degradation or loss of the bed material along a considerable length of a waterway. It can be the result of natural erosion, mining activities, construction, or other events.

2. Contraction scour involves the removal of material from the bed and banks across all or the majority of the width of a channel. Contraction scour is caused by a reduction in the upstream channel cross-section, which results in increased flow velocities, increased bed shear stresses, and subsequent loss of material.

3. Local scour is the removal of material from a smaller area and is restricted to a minor portion of the width of the channel. The main mechanism of local scour is the formation of vortices at the base of piers, piles or other substructure elements as a result of currents, propeller wash, discharge/intake pipes, or other factors.

As discussed in Section 1.3.8, divers should note any signs of scour during underwater inspections. An important assessment during any inspection is how much of the substructure foundation is exposed when compared to design plans.

The inspector should check for scour at every structure over a waterway. The inspector should be aware that scour is generally most severe during periods of high flow[1] and when flows recede to normal levels; the presence of scour is often covered up with silt or timber debris, making detection difficult. Comparison of previous profiles is typically needed to detect and assess general and contraction scour.

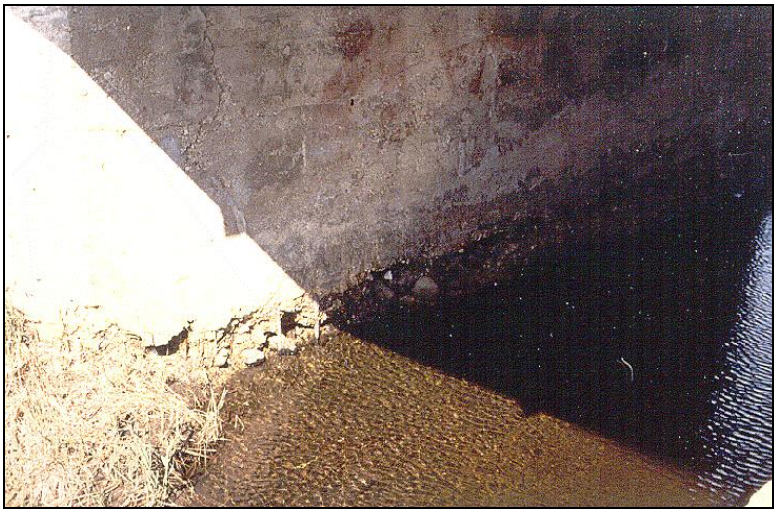


Figure 1.3.6.1-1: Local Scour at the Base of an Abutment.

The ability to capture the true picture of the underwater landscape with an underwater profile is limited. A hydrographic survey is performed in conjunction with the underwater dive inspection using sonar equipment on select bridges. Generally, a hydrographic survey is

[1] High flows following a major rainfall event can generally be expected to occur about 12 hours after precipitation ceases as a rough rule of thumb; however, every waterway is different, based on a variety of factors.

completed on bridges over larger bodies of water where the water stretches across the full span or multiple spans and boat access is necessary and possible.

The profile may be completed by a TM trained by a TL or PM. A review of the TM's work must be completed by a TL or PM. The results of the profile must be compared with historical data to ascertain potential movement of the channel and risk of substructure undermining. If potential movement of the channel and risk of substructure undermining exists, this may be considered a critical finding and require notification to the Statewide Program Manager. See Section 1.7 for the Critical Finding Procedure and Documentation.

The results must be entered into HSIS with documentation that show or explain the profile. The HSI system will allow the uploading of numerous file formats including Excel spreadsheets. At the inspector's discretion, a spreadsheet may be created for a structure and uploaded to the HSI system for future reference and modification, if necessary. Example template spreadsheets can be utilized on our website under Inspection Sketches and Templates.

Closed cell structures, such as box culverts and pipes do not require underwater profiles. However, channel probing for scour at the end of a floor or apron is required during the routine inspection. Document findings from probing and channel movement concerns or conditions on the Underwater Probe Form in the inspection report.

Additional requirements and recommendations for when the underwater profile activities are to be performed can be found in Section 1.3.7.

Underwater Profiles in Conjunction with Underwater Dive Inspection

Structures requiring underwater dive inspections will have a "global area" underwater profile activity performed at the bridge during the dive inspection including the following information:

1. Underwater profile activity and global area photographs up and downstream
    a. Underwater profile at up and downstream fascia
        a. Bottom elevations at sufficient intermediate points between substructure units to adequately determine the top and bottom banks of the waterway, the thalweg of the waterway, and any significant waterway elevation changes.
        b. Maximum water depth measurements at each substructure unit in the water.
    b. The lateral movement of the channel up and downstream will be monitored by the following method:
        a. Review existing arial photographs to compare to the field conditions.
        b. Take up/downstream photographs of the channel and conditions from the bridge deck or channel banks.
            1. Capture enough photos to capture an upstream and downstream view of the entire flood plain; Keeping the bridge rail in the picture

will allow for consistent photos and determining angle of attack of waterway; or

2. Overhead photographs up and downstream of the entire flood plain using a UAV (drone).

c. Document signs of lateral movement or changes in the channel

2. Hydrographic surveys can be performed in lieu of an underwater profile activity based on the determination of the PM. If a hydrographic survey is performed, then generate fascia cross sections to include in the inspection report. The following guidance can help to determine when to choose a hydrographic survey:

a. Generally, performed on bridges over larger bodies of water where the water stretches across the full span or multiple spans.

b. Locations of bodies of water that are best surveyed using a boat for access.

c. Locations that typical methods of channel survey would be impractical.

d. Locations where monitoring lateral stream migration by review of new and historical photographs is not sufficient to ascertain channel condition/movement. Include the global area photographs when the hydrographic survey is completed.

1.3.6.2 Precision

Hydrographic survey data is used to evaluate trends in channel bottom movement and to compare channel bottom elevations to footing elevations. Water depth measurements should typically be recorded to the nearest tenth of a foot. However, scour evaluations are typically based on changes in elevations greater than 0.5 foot since most channel bottoms are irregular surfaces with random cobbles, debris, and sand ripples.

It is generally an acceptable practice for scour inspectors to measure the water depth relative to the water surface, in waterways without steep profiles or obvious hydraulic drops, assuming the waterline elevation in most waterways is constant over the surveyed area adjacent to the bridge. In actuality, since water always flows toward a lower topographic elevation, it is common for there to be at least 0.1 foot decrease in water surface elevation over a length of 500 feet in most waterways in Wisconsin. For waterways with steep profiles or obvious hydraulic changes in the water surface elevation, all water surface elevations must be recorded if direct water depth measurements are taken. Rather than documenting several water surface elevations, the inspector may choose to record the channel bottom elevation to a constant elevation using a surveyor's level or total station equipment.

During all underwater surveys, the water surface elevation shall always be referenced to a known elevation on or near the bridge.

In most instances, a bridge profile is located on a tangent or vertical curve. In order to expedite the streambed profile, the Wisconsin DOT recommends determining the elevation of an accessible bridge component. For example, the top of a bridge rail, or edge of deck are

common landmarks an inspector can utilize when taking streambed elevations. The purpose of the profile is to observe changes to the streambed from inspection to inspection. So long as every inspector subsequently uses the same landmark and elevation for data recording, an accurate account is developed. Therefore, it is imperative that the inspector clearly identify the elevation of the landmark (these can be derived off existing plans or arbitrarily chosen) and the location the measurements are taken from (e.g. North Abutment – Upstream Fascia at Centerline of Bearing). When using an arbitrary elevation, the inspector may assume a constant elevation along the length of the structure, even if located on a vertical profile. Again, this is acceptable because the profile is a comparative tool. So long as the subsequent inspections replicated this arbitrary elevation and the locations at which measurements are taken, the data shall indicate whether there have been any changes to channel alignment or elevation.

While the true streambed elevation determined may be skewed due to the bridge profile following a tangent or vertical curve, the data will be able to be compared to later inspections.

1.3.6.3 Inspection Interval

1. All structures over water except 4-sided structures (i.e. box culverts and round/elliptical pipes) are required to have an initial underwater profile activity completed during the Initial Inspection with subsequent underwater profiles completed at a maximum interval of 96 months.

2. Bridges with a concrete floor for the streambed, other than culvert type bridges, require only an initial profile activity completed. The profile elevations should be obtained along the end of the concrete floor. If the concrete floor runs beyond the right-of-way, the initial profile can be taken along the bridge soffit.

3. Higher Risk Bridges are those meeting the criteria below. These bridges have the underwater profile activity completed at a maximum 24-month interval.

    a. B.C.09 Channel Condition Rating ≤ 4 (poor)
    b. B.C.10 Channel Protection Condition Rating ≤ 4 (poor)
    c. B.C.11 Scour Condition Rating ≤ 6
    d. B.AP.03 Scour Vulnerability = C, D, E, or U (bridge is scour critical)
    e. B.AP.04 Scour Plan of Action = A scour POA is required but not implemented (N) or scour POA is required and implemented (N).

Structures that require underwater dive inspections will have Global area profiles at the required dive interval and can forgo the requirements of Section 1.3.6.3(3). Evidence of channel movement, degradation, or aggradation may necessitate more frequent Global area profiles based on the recommendation of the dive inspection TL. However, dive inspection TL should recommend to the PM more frequent Global area profiles be scheduled if there is evidence of channel movement, degradation, or aggradation.

1.3.6.4 Equipment

Several water depth measurement methods, with a variety of equipment, can be used during a scour inspection with a hydrographic survey. These water depth measurements, often called

soundings, can be obtained by manual means (lead line or sounding pole) or technological equipment (fathometer, sonar, radar, etc.). Refer to Chapter 19 in Part 5 for information on underwater profile equipment.

1.3.7 Scour Monitoring Activity (Scour Monitoring Inspection per SNBI)

Nearly all bridges which cross waterways have some vulnerability to scour damage caused by a flood event. A scour monitoring activity is intended to ensure monitoring for structure stability and stream instability during and after a flood event.

A scour critical bridge requires an implemented scour plan of action (POA) (see section 1.3.18) which details the monitoring when triggered by a flood event. A scour monitoring activity is required when flood conditions are reached as described in an implemented POA. At a minimum, the scour monitoring activity must include a post-flood activity of cross sections (profile) and probing after a significant flood event. Document and record the activities in HSIS as a Scour Monitoring Activity and a UW-Profile Activity.

A scour monitoring activity is also required at any bridge where there are signs of channel movement, degradation, or aggradation during or after a flood event.

A scour monitoring activity is recommended for any bridge over a waterway when the bridge experiences any of the following:

- A flood event when the bridge or the roadway approach is overtopped. *
- A river system reaches flood state at a bridge. *
- The presence of highwater flow velocities which could cause scour.
- If determined by the bridge owner or inspection program manager.

The National Weather Services' Advanced Hydrologic Prediction Service provides flood predictions and flood stages for river systems.

A scour monitoring activity typically involves documentation and actions during and after a flood event by an individual assigned by the PM, TL, or bridge owner (or owner's representative). The inspection activity must be completed by a competent TM under the guidance and coordination of the PM or TL. A review of the TM's work must be completed by the PM or TL. The findings must be documented in HSIS as a scour monitoring activity. Multiple visits to the same bridge during a flood event as part of the scour monitoring can be recorded as a single scour monitoring activity. The start date is the first day the bridge is observed for an event and the end date is the day of the final post-flood survey or activity.

A scour monitoring activity includes the appropriate activities and documentation listed below to ascertain the extent of sour and channel movement:

- Underwater profile at the up and downstream fascia (see Section 1.3.6 for information about underwater profile activity).
  - When required as part of an implemented scour POA for a triggering flood event.
  - Recommended at a bridge over water when the bridge experiences a flood event.
- Underwater probing around the substructure units

  - When required as part of an implemented scour POA after a triggering flood event.
  - Recommended post-flood for bridge units in 4’ or less water when the bridge experiences a flood event, to determine the extent of any scour.
- Record of observations including photographs
  - Flood monitoring record or log entered into HSIS – a spreadsheet log is recommended for ongoing or prolonged monitoring. Document in the Inspection Specific Notes section of the inspection report for simple monitoring.
  - Photographs to record condition, changes in condition, and maintenance actions.
- Recommended maintenance actions.

Special/Damage Inspection – a flood event may require review of condition assessments or ratings for elements or components affected. These inspections must be completed by a TL.

1.3.8 Underwater Dive Inspection

1.3.8.1 Purpose

CFR 650.305 defines an underwater inspection as: *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.*

Underwater Dive Inspections are a necessary part of an effective structure management program and are mandated by the Federal Highway Administration (FHWA) on routine intervals. Underwater dive inspections should be completed in accordance with OSHA 29 CFR 1910 SUBPARTS T AND Y, and the requirements described in this section. An underwater dive inspection is required if water conditions exist at the structure that prohibit access to all portions of an element by visual or tactical means ensuring a level of certainty during Routine Inspections. If probing or observations of the substructure elements for scour or deterioration cannot be completed during low-flows (water depths 4’ or less), then an underwater dive inspection is required.

These specialized inspections serve an important part in protecting the public, providing reliable service, and reducing maintenance and construction costs. Structural conditions above water that could lead to failure, loss of life or property damage are often observed well in advance by inspectors, maintenance workers, and sometimes even passing motorists. Conversely, significant underwater structural conditions cannot be observed by these individuals until the defect has progressed to the point where distress is evident above water. Unfortunately, structures exhibiting significant underwater defects often collapse before the distress is evident above water.

Although each type of material has predominant mechanisms of deterioration, the environment (moderate temperatures, moisture, oxygen, and chlorides or other chemicals) at the waterline is most conducive to all forms of deterioration. Furthermore, unique mechanisms, such as bacterial corrosion, are also common near the waterline on structures. This deterioration and distress may not be recognizable from above water, nor can the extent and severity be determined in most cases without inspecting the underwater elements.

Underwater profile requirements in conjunction with underwater dive inspections can be found in Section 1.3.6.

1.3.8.2 Precision

Due to limited underwater visibility, the inherent access restrictions of the underwater environment, and the presence of marine growth, the required underwater diving inspection precision depends on the level of effort. Three underwater diving inspection intensity levels are defined by the FHWA. The expected underwater diving inspection precision is based on the individual coverage percentage of these three levels of effort. A summary is provided in Figure 1.3.8.2-1 with narrative descriptions of each level following the figure.

Level	Purpose	Typical Detectable Defects/Expected Findings			
		Steel	Concrete	Timber	Composite
I	General visual/tactile inspection to confirm as-built condition and detect severe damage	Extensive corrosion and holes Severe structural damage	Major spalling and cracking Severe reinforcement corrosion Broken piles	Major loss of section Broken piles and bracings Severe abrasion or marine borer attack	Permanent deformation Broken piles Major cracking or structural damage
II	To detect surface defects normally obscured by marine growth	Moderate structural damage Corrosion pitting and loss of section	Surface cracking, spalling, erosion Rust staining Exposed reinforcing steel and/or prestressing strands	External pile damage due to marine borers Splintered piles Loss of bolts and fasteners Rot or insect infestation	Cracking Delamination Material degradation
III	To detect hidden or interior damage, evaluate loss of cross-sectional area, or evaluate material homogeneity	Remaining thickness of material Electrical potentials for cathodic protection Change in material properties	Onset of reinforcing steel corrosion Internal voids Change in material properties	Internal damage due to marine borers (internal voids) Decrease in material strength Change in material properties	Change in material properties

Figure 1.3.8.2-1: Summary of Intensity Levels.

Level I Effort

An inspection involving a visual examination or a tactile examination using large sweeping motions of the hands where visibility is limited. Although the Level I effort is often referred to as a “Swim By” inspection, it must be detailed enough to detect obvious major damage or deterioration due to overstress or other severe deterioration. It should confirm the full-length continuity of all members and detect undermining or exposure of normally buried elements. A Level I effort may also include limited probing of the substructure and adjacent channel bottom. Refer to Figure 1.3.8.2-2 for a view of a structure during a Level I effort.

Figure 1.3.8.2-2: Inspector Conducting a Level I Inspection Effort.

Level II Effort

A Level II effort is a detailed inspection that requires marine growth to be removed from portions of the structure. Cleaning is time-consuming so there is a need to limit the detailed inspection to a representative sampling of components. For piles, a 12-inch high band should be cleaned at designated locations, generally near the waterline, at the mudline, and midway between the waterline and the mudline. On an H-pile, marine growth should be removed from both flanges and the web. On a rectangular pile, the marine growth removal should include at least three sides; on an octagonal pile, at least six sides; on a round pile, at least three-fourths of the perimeter. On large diameter piles, three feet or greater, one-foot squares should be cleaned at four locations approximately equally spaced around the perimeter, at each elevation. On large solid faced elements such as pier shafts, one-foot squares should be cleaned at four random locations, at each elevation. The Level II effort should also focus on typical areas of weakness such as attachment points and welds. The Level II effort is intended to detect and identify damaged and deteriorated areas that may be hidden by surface biofouling. The thoroughness of cleaning should be governed by what is necessary to discern the condition of the underlying material. Removal of all biofouling staining is generally not required. Refer to Figure 1.3.8.2-3 for a view of a structure during a typical Level II effort.

Figure 1.3.8.2-3: Inspector Conducting a Level II Inspection Effort.

Level III Effort

A detailed inspection typically involving nondestructive evaluation (NDE) or partially-destructive evaluation (PDE), conducted to detect hidden or interior damage, or to evaluate material homogeneity. Typical inspection and testing techniques include the use of ultrasonic, coring or boring, physical material sampling, and in-situ hardness testing. Level III testing is generally limited to key structural areas, areas which are suspect or areas which may be representative of the underwater structure. Refer to Part 5 of this Manual for additional information on nondestructive and partially destructive evaluation. Also, refer to Figures 1.3.8.2-4 and 1.3.8.2-5 for views of inspectors conducting Level III efforts.

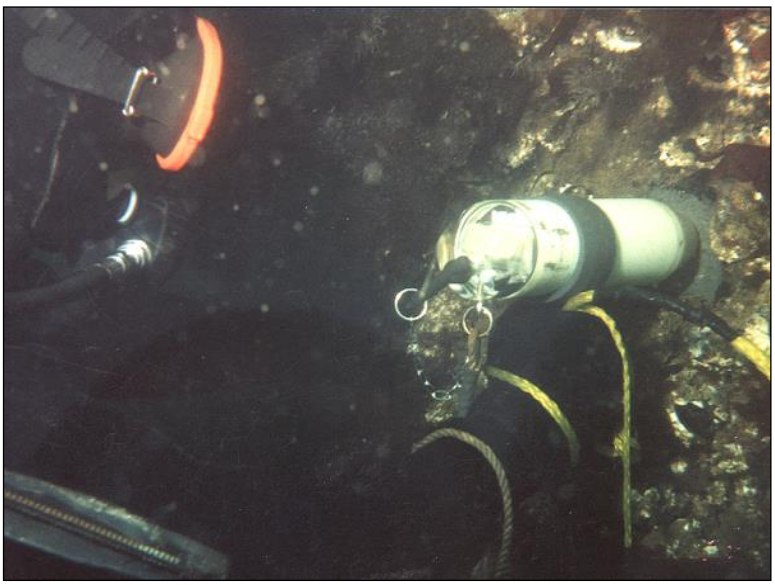


Figure 1.3.8.2-4: Inspector Using a D-Meter to Conduct a Level III NDE Inspection Effort.

Figure 1.3.8.2-5: Inspector Using a Drill to Conduct a Level III PDE Inspection Effort.

1.3.8.3 Underwater Dive Inspection Interval

CFR 650.311(b) establishes the minimum requirements for underwater inspection interval. These requirements are described and elaborated on in the NBIS *Metrics for Overview of the National Bridge Inspection Program*.

Underwater Dive Inspection Interval:

Wisconsin requires adherence to the following criteria :

1. An underwater profile is required as part of the Initial inspection.

2. An underwater dive inspection must include at least a Level I effort on 100 percent of all underwater elements, a Level II effort on 10 percent of all underwater elements, and a Level III effort as determined by the TL or PM.

Lower risk bridges -

UW inspections are performed at regular intervals not to exceed (NTE) 60-months, or NTE 72-months when adhering to FHWA approved UW criteria. The criteria a structure must meet to be considered for the 72-month Routine UW-inspection Interval are as follows:

- B.C.09 Channel Condition ≥ 6(satisfactory)
- B.C.10 Channel Protection Condition ≥ 6 (satisfactory)
- B.C.11 Scour Condition Rating ≥6 (satisfactory)
- B.C.15 Underwater Inspection Condition ≥ 6 (satisfactory)
- B.AP.03 Scour Vulnerability item = A or B (stable for scour)
- Structure must have at least two (2) Underwater Dive inspections on file to be eligible.

- Border bridges with adjoining states are eligible with adjoining state agreement.
- No bridges with complex features
- No substructure Elements with the Microbial Induced Corrosion Defect (8901).
- No substructures with timber substructure elements are eligible.
- Must be less than 50 years old.

The 72-month extended UW-Inspection Intervals is optional and is up to the structures owner to utilize. The eligibility requirements are the same as previously describe for the 48-month routine inspections. The structures owner interested in participating in the 72-month extended UW-inspection Intervals needs to have the County PM or Commissioner fill out the DT2002 Structure Inspection Quality Control Form and follow the instructions enclosed.

High risk bridges –

Bridges meeting the following criteria must be inspected at a reduced interval not to exceed 24 months:

- B.C.09 Channel Condition ≤ 3 (serious)
- B.C.10 Channel Protection Condition ≤ 3 (serious)
- B.C.11 Scour Condition Rating ≤ 3 (serious)
- B.C.15 Underwater Inspection Condition ≤ 3 (serious)
- B.AP.03 Scour Vulnerability = 0, C, D, E, or U
    - C or D = bridge is scour critical
    - 0 or E = scour appraisal has not been completed
    - U = bridge has unknown foundation

Partial underwater inspections may be completed on bridges meeting the reduced interval criteria for the underwater inspection due to localized deficiencies. The inspection program manager, with concurrence from the bridge owner, may request from the SPM, approval to complete a partial underwater inspection on the element(s) contributing to the condition rating of 3 or less. The request, including the reason for the exemption and the inspection procedure for the partial inspection must be provided to the SPM at least 2 months prior to the month the inspection is due. Bridges granted the partial underwater dive inspection by the SPM will have the inspection results entered in HSIS as an Underwater inspection with only the inspected elements checked, with the inspection procedures for the partial inspection, and with the documentation showing SPM approval. A full underwater dive inspection must occur on a 24-month interval.

1.3.8.4 Methods of Underwater Dive Inspection

After identifying that a structure requires an underwater diving inspection, it must be decided which underwater diving inspection method should be used. Underwater diving inspection methods are categorized as “manned” or “unmanned”. The following factors influence the determination of which method of underwater diving inspection is best suited for a structure:

1. Water depth (depth greater than 4 feet should be performed by diving)

2. Water visibility
3. Water velocity (if greater than 2 feet per second, should be performed by diving)
4. Streambed conditions (if soft or irregular, should be performed by diving)
5. Presence of debris or other obstructions/obstacles
6. Substructure configuration

The qualified “manned” methods consist of an inspection-diver using commercial Self-Contained Underwater Breathing Apparatus (SCUBA) equipment or surface supplied air (SSA) equipment. For qualified “manned” methods, the Team Leader is required to be a certified diver and be able to perform the underwater inspection.

The “unmanned” methods typically use a real-time submersible videography lens or electronic imaging devices to transmit observation data to a qualified Team Leader. Although electronic imaging devices are not often used in Wisconsin, submersible videography lenses have been used on telescopic poles and in remote operated vehicles (ROVs). When the diver is not a qualified Team Leader and the Team Leader is not a qualified diver, the lenses can be attached to a diver’s helmet. While these “unmanned” methods are acceptable if they are conducted in a way that ensures a sufficient level of certainty, they should be considered only as a secondary alternative if the more preferable qualified “manned” method is not feasible. The Team Leader’s assessment capabilities are adversely affected when unable to perform the actual physical inspection.

Underwater diving inspection in Wisconsin is most frequently conducted by a dive team using SCUBA equipment. This method consists of using a standard exposure suit and a portable air tank. The inspector(s) will make a visual and tactile evaluation of the substructure units by swimming around the individual units. SCUBA equipment allows the inspector greater freedom of movement, the ability to visually inspect the substructure units both above and below the waterline, even in poor water visibility, and to reach all areas even in deep water. Limitations of the SCUBA method are: the duration of the inspection due to a limited air supply (dive should typically be finished prior to a pressure gauge reading of 750 psi); a permissible depth range for safe operation (120 feet); additional tethering in swift currents; and specialized training and equipment for the inspectors. Refer back to Figures 1.3.8.2-2 to 1.3.8.2-5 for views of inspectors using scuba equipment.

Although more common in underwater diving construction, an underwater diving inspection can also be conducted by a dive team using a surface supplied air system. The equipment consists of a standard exposure suit, a full-face mask/helmet, and umbilical cords connecting the diver to the surface. The inspector(s) will make a visual and tactile evaluation of the substructure units by swimming around the individual units. This method of inspection provides many of the same benefits as a SCUBA inspection along with being well-suited for adverse diving conditions, such as swift velocities (typically up to 14 fps), polluted water, and long diving durations. Limitations of the surface supplied air method is that the equipment: limits free movement; a permissible depth range for safe operation (220 feet), and specialized training and equipment for the inspectors. Refer to Figure 1.3.8.4-1 for a view of an inspector using surface-supplied-air equipment.

Figure 1.3.8.4-1: Inspector Using Surface-Supplied Air Equipment.

1.3.8.5 Inspection Equipment and Tools

The underwater diving inspectors will require a larger than normal amount of equipment to complete the various tasks associated with the structure investigation regardless of the method used. These items are a mix of common tools and specialized equipment that will provide a breathing medium, means of movement, and aid the inspector in collecting data at the structure.

Personal Equipment

For an underwater diving inspection, a brief equipment list is as follows:

1. Exposure suit (wet or dry)
2. Dive mask or helmet
3. Breathing apparatus
4. Air supply (portable tank or surface compressor unit)
5. Weight belt
6. Dive fins
7. Buoyancy compensator

8. Depth gauge / pressure gauge (All dives should be terminated at 750 psi.)
9. Wristwatch
10. Light source

Furthermore, an inspector conducting a diving inspection should carry additional equipment such as a knife and reserve air tank or J-valve on the tank.

Access Equipment

While access is often gained from the shoreline, some structures are best accessed by use of a boat. Typically, an 18-foot or larger vessel can safely carry the equipment and crew. On some occasions, access may be gained from the structure itself.

Communication Equipment

While it is not mandatory to be in voice communication during shallow water dive inspections, two-way voice communication greatly aids in the efficiency of the inspection data collection and recording, and it provides an added level of safety. For deep-water inspections, the use of two-way voice or hand-signal communication is recommended. The advantages of direct voice communication are:

1. The diver can communicate directly with the note-taker to describe the location, type, and size of any observed defects.
2. The diver can discuss any observations with surface personnel.
3. When using video equipment, the surface personnel can direct the diver to specific areas that appear suspect or where closer investigation needs to be conducted.
4. The diver can immediately report the extent of any problems.

Tools

The inspection team should have access to the appropriate tools and equipment as warranted by the type of inspection being conducted. A number of tools should be available to the inspector and can be categorized as hand-tools or power-tools. Since power-tools are not frequently used, a brief list of typical hand-tools is as follows:

1. Ruler
2. Calipers
3. Probe (ice picks, awls, screwdrivers, etc.)
4. Geologist hammer
5. Scraper
6. Wire brush

7. Pry bar

Testing Equipment

Often an inspection requires some level of material testing to ascertain the condition of the substructure unit that may not visually show any significant signs of deterioration. Testing is also the main component of a Level III inspection. Testing may be either nondestructive or partially destructive and is described in detail in Part 5 of this Manual. Nearly all the methods in Part 5, which are applicable for use on substructures, can also be performed underwater.

Photography & Videography Equipment

A still or video camera can provide a visual record of defects or deterioration that is observed by the inspector. This information can be reviewed with others to better define and evaluate the significance of the defect.

A still camera can be fitted with a variety of lenses and flash units that are suited for different conditions. In low visibility, the camera will need to be placed close to the object and will require a wide-angle lens. Particles that are suspended in the water, which make it cloudy, reduce ambient light and can reflect light from the flash unit into the lens. When visibility is very low, clear water boxes can be used. A clear water box is constructed of clear plastic and is filled with clean water. By placing the box against the object to be photographed, the box of clean water will displace the murky water allowing for a clear photograph. Refer to Figure 1.3.8.5-1 for a view of a typical clear water box.

Figure 1.3.8.5-1: Inspectors Attaching a Clear Water Box to an Underwater Camera.

Video equipment is generally available as self-contained submersible units, or as a submersible camera lens attached to the diver with a cable connection extending to a surface monitor and controls. The latter allows a surface operator to direct the shooting, control the lighting and focusing, and communicate with the diver to obtain the optimum image. A

soundtrack could also be dubbed with the video image by the diver or topside personnel to provide a running commentary pertaining to the observations.

1.3.8.6 Underwater Dive Inspection Procedures

A bridge that receives an underwater inspection must have a detailed written inspection procedure specific to that bridge. These inspection procedures are to be kept in the bridge file, reviewed, and updated for **each** underwater inspection.

The inspection procedures must address any of the following areas that are relevant to the specific bridge receiving the underwater inspection.

- Clearly Identify the location of all underwater elements and assess all units during the dive inspection.
    - Dry units can be documented as “dry” without additional detail.
    - Units that can be probed shall be and notes shall be taken on the condition of the streambed and substructure unit.
    - Units require diving shall be inspected according to guidelines in the SIM.

- Specify the interval of the underwater inspection.

- Describe any specific risk factors.

    - The specific risk factors shall be separated into Diver and Structure risk factors.

    - **Diver Risk Factors**

        - For each bridge, include diver risk factors (diver safety) in a separated paragraph under the Inspector Site-Specific Safety Considerations section of the report. Start the paragraph “Diver risk factors:”. If there are no diver risk factors, please note “Diver risk factors: None”.

        - Some diver risk factors could be:

            - Debris accumulation
            - Limited visibility in the water
            - Rapid stream or current
            - Soft or unstable streambed or stream banks for walk in entry.
            - Pollutants in water

Note this is not an exhaustive list so others may qualify as diver risk factors.

    - **Structure Risk Factors**

        - Include all structure risk factors (related to scour, environment, or structure) at a bridge in the specific procedures section of the dive inspection report.

Provide a separated section of the procedure to address structure risk factors. This section should be started with “Structure risk factors:”. List or describe the structure risk factors. If there are no structure risk factors, please note “Structure risk factors: None”. Remove any structure risk factors listed elsewhere outside of the UW-Dive specific procedures section or separate documents since they should only be listed in UW-Dive specific procedures section of the report.

- Some structure risk factors could be:
    - Debris accumulation
    - Rapid stream or current
    - Pollutants in water
    - Marine environment
    - Meandering channel
    - Unknown foundation
    - Scour critical bridge
    - Observed scour
    - Environmental conditions (i.e. MIC for steel, timber piling – limnoria) which may accelerate deterioration.

Note this is not an exhaustive list so others may qualify as structure risk factors.

- Clearly detail inspection methods
- Equipment and tools to be employed.

1.3.9 Underwater Probing

This method of underwater inspection involves either wading or probing with a rod or the feet. Underwater probing is the most basic type of underwater inspection activity and can often be performed by an inspector wading in the water with no additional training. Underwater probing activity is required for all structures over water that are not dove at regular intervals and should be done in conjunction with the Routine inspection.

Underwater probing activity is conducted by evaluating the substructure units and the waterway by using a probe rod or sounding pole. The inspector wearing waders (or a dry/wet suit) walks around the substructure, probing the units and channel bottom with the rod and with his/her feet, while visually inspecting the areas above and directly below the waterline where visibility permits. Limitations of the wading inspection are deep water, poor water visibility, excessively soft or irregular streambed conditions, and swift currents that make

movement difficult or dangerous. Refer to Figure 1.3.9-1 for a view of an inspector conducting a wading inspection.

The results of the probe are entered into the HSI system when an inspection type is created under the Tab titled Underwater. This tab lists all substructure units on the bridge. If the substructure unit is dry at the time of the probing, the inspector shall note that on the form for the unit in question.

Figure 1.3.9-1: Inspector Conducting a Wading Inspection.

1.3.9.1 Inspection Equipment and Tools

Personal equipment typically includes hip waders, a hard-hat, reflective vest, and tool belt for an inspector conducting an underwater probing/visual (wading) inspection.

1.3.9.2 Underwater Probing Activity Interval

Underwater probing is required to be performed as part of the routine inspection, initial inspection, as well as during an underwater dive inspection. If underwater probing activity cannot be performed during the inspection, the inspector has 3 months to complete the underwater probing while the routine inspection type is open in HSI.

1.3.10 Special Inspection

1.3.10.1 Purpose

A special inspection is scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency, to monitor special details or unusual characteristics of a bridge that does not necessarily have defects, or update the inventory and inspection data after a completed repair or maintenance action. A TL is required on site for any special

inspection whenever a bridge component condition rating(s) (B.C.XX items) is assessed. The following is a list of possible special inspection activities and required inspector qualifications:

Special Inspection Activities and Qualifications

Special Inspection Activity	Minimum Qualifications	Interval
NDE for Timber	Trained in the type of NDE being performed under direction/oversight of a TL (see Specialist under SIM 1.2.3.2).	As needed and determined by the PM
NDE for Steel	Certified or trained in the type of NDE being performed under direction/oversight of a TL (see Specialist under SIM 1.2.3.2).	As needed and determined by the PM
NDE for Concrete	Trained in the type of NDE being performed under direction/oversight of a TL (see Specialist under SIM 1.2.3.2).	As needed and determined by the PM
Isolated Activity - reduced interval in lieu of Routine Inspection	TL	Per SIM 1.3.10.2
Completed Repair or Change in Overburden	TL	Per SIM 1.3.10.3
Special Inspection without Activity	TL	As needed and determined by the PM

*Under direction and oversight means TL may not be on site but provides training and review of TM work.

For special inspections where a TM completes the inspection, the TL must review the report and sign as reviewer in HSIS. If the TL identifies an issue the PM should be notified.

1.3.10.2 Special Inspection – Isolated Activity

Where Deck, Superstructure, Substructure, Culvert, or Scour Condition Ratings are coded four (4) or less due to localized deficiencies, a Special inspection with Isolated activity which is limited to the localized deficiencies can be used in lieu of a Routine inspection (except for

buried and single span bridges, see next paragraph). A Routine inspection must be conducted at the appropriate interval.

In general, single span and buried bridges have similar Routine or Special-Isolated inspection resource needs. It may be advantages to complete the Routine inspection instead of a Special-Isolated inspection. This decision is left to the specific bridge's inspection program manager. The intent of the Special inspection - Isolated activity is to focus limited inspection resources on critical needs by not requiring a routine inspection for an isolated/localized deficiency; however, in general, single span and buried bridges have similar inspection resource needs for a Routine or a Special - Isolated inspection.

The following must be updated in HSIS by the PM for a Special Inspection – Isolated activity:

1. The region PM will set the minimum interval to a minimum of 1/2 of the Routine inspection interval.

2. For local bridges, the local PM must request the region PM set the Special inspection - Isolated activity interval to 1/2 or less of the Routine inspection interval.

3. The responsible PM will review and set the Routine inspection interval not greater than 24 months.

4. The responsible PM will include the reason for the inspection, the elements requiring inspection, and any special inspection procedures under the Special Inspection Specific Procedures box of the Notes tab.

1.3.10.3 Special Inspection for Completed Repairs or Overburden Change

Since both the Specification for the National Bridge Inventory (SNBI) and Element Level rating scales are based upon existing structural conditions, repairs may improve the condition ratings assigned to a structure. Repairs resulting from damage (likely from corresponding damage inspection) or any permanent repair that affects the SNBI rating requires a Special inspection. Maintenance, temporary repairs, or preservation activities would not require a Special inspection. However, when the overburden on a bridge is changed due to a maintenance or preservation activity, a Special inspection must be completed to update the overburden quantity, the element(s) condition states and Condition ratings, when applicable. A Special inspection for a repair or overburden change should be completed within 6 months of the completed action.

1.3.10.4 Special Inspection Procedures

Documented inspection procedures are to be included with the inspection report for special inspections used for monitoring conditions. 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.

1.3.10.5 Special Inspection Interval

In general, Special Inspections are scheduled at the discretion of the individual responsible for structure inspections for the unit of government that owns the structure.

1.3.11 Damage Inspection

1.3.11.1 Purpose

A Damage Inspection is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions. Flood damage, barge impact, and vehicle impact are common examples of events that may call for a Damage Inspection. A team lead qualified inspector will perform a damage inspection. Additional inspector qualifications for damage that may affect bridges with NSTM, SRM, or underwater elements are as follows:

- For NSTM, SRM, bridges, if the damage may affect an NSTM, SRM, member/element, the inspection must be performed by an NSTM TL qualified inspector following NSTM inspection procedures.
- For damage suspected to have occurred to an underwater portion of a bridge in water greater than 4 feet deep, the underwater portion must be completed by an UW Dive TL qualified inspector.

The TL is required to notify the PM with direct jurisdiction over the bridge as soon as possible. The PM is required to sign off as reviewer in HSIS if they were not the inspector no later than 30 days after the inspection is completed. The PM, with consultation with the TL, when necessary, must determine an appropriate inspection interval based on the condition of the bridge.

Figure 1.3.12.1-1: Impact Damage to a Steel Girder Bridge.

Figure 1.3.11.1-2: Failed Wingwall.

1.3.11.2 Precision

The scope of a Damage Inspection should be sufficient to determine whether there is a need for emergency load restrictions or closure of part or all of the structure to traffic. The inspector should also assess the level of effort necessary to repair the damage. The amount of effort expended on this type of inspection may vary significantly and depends on the extent of the damage. If major damage has occurred, inspectors should evaluate fractured and cracked members, determine the extent of section loss, make measurements for misalignment of members, and check for any loss of foundation support. A structure inspection form should be filled out and submitted for entry into the HSIS database with addenda and pictures, if necessary, with all the information mentioned above. This inspection may be supplemented by a timely In-Depth Inspection to document more fully the extent of damage and the urgency and scope of repairs. Proper documentation, verification of field measurements and calculations, and perhaps a more refined analysis to establish or adjust interim load restrictions are required follow-up procedures. The ability to make on-site calculations to establish emergency load restrictions may be desirable. A particular awareness of the potential for litigation must be exercised in the documentation of Damage Inspections. Therefore, all documentation should be legible and thorough.

1.3.11.3 Damage Inspection Interval

A Damage Inspection is an unscheduled inspection that is performed to determine if significant damage has been done to the bridge. Generally, a law enforcement officer on the site of an accident involving a structure will notify the appropriate individuals and request a Damage Inspection be performed to determine if the bridge should be closed. A Damage Inspection may be followed up by an In-Depth Inspection to document the full extent of the damage.

1.3.12 Service Inspection

1.3.12.1 Purpose

A service inspection is not a required inspection but is scheduled at the discretion of the bridge owner to identify major deficiencies, safety issues, and maintenance actions. It is performed by bridge maintenance or inspection staff under direction and oversight of a TL. The TL is not required to be on site but provides training and review of TM work. The inspections are meant to be performed from the ground and are not intended to be as rigorous as a routine inspection but requires documentation of any follow-up actions in the HSI. It's recommended to enter a service inspection in HSI if the inspector doesn't find anything during a bridge visit. The service inspection can be selected in HSI within the Create tab under Activity type. Document the purpose of your visit under the inspection notes in the service inspection.

1.3.12.2 Service Inspection Interval

It is considered good practice to schedule a service inspection around the half inspection interval on bridges that have a history of safety or maintenance issues and on bridges with a 48-month inspection interval.

1.3.13 Complex Feature Inspection

1.3.13.1 Purpose

A complex feature is the bridge component or member with advanced or unique structure members, operational characteristics, construction methods, and/or requiring specific inspection procedures. The focus is on the bridge component or member that warrants additional attention due to their inherent complexity. Bridges with complex features are identified in the SNBI Item B.IR.04 Complex Feature.

WisDOT has identified complex features as the mechanical and electrical elements of movable spans and the primary cables and anchorages of cable stayed bridges. Inspections results and procedures of bridges with complex features will be recorded and reported as follows:

Complex Feature	HSIS Inspection or Activity Type
Movable bridge mechanical and electrical elements	Movable Inspection
Cable stay – primary cables and anchorages	Complex Feature Inspection

Complex features are subject to specialized inspection procedures. Additional inspector training and experience is required to inspect structures with complex features. The inspection of complex features require greater engineering knowledge and/or expertise to accurately and fully determine the condition of the various bridge elements. They may also require specialized equipment or climbing to access these features. The qualification to inspect primary cables of a cable stayed bridge is to be a TL and possess the FC or NSTM NHI course certificate. Moveable bridge qualifications are described in Part 3 of this manual. The complex feature qualified inspection team leader must be present during the complex feature inspection.

1.3.13.2 Bridge Inspection Procedures

CFR 650.313(g) establishes the requirement for all complex structures to have a bridge specific inspection procedure in place. Complex feature inspection procedures are to address the following topics applicable to those features:

- Equipment needs
- Personnel needs and qualifications
- Access requirements
- Scheduling considerations
- Coordination with agencies and/or partners
- Risk factors
- Identify/describe the specific complex feature of the bridge to be inspected
- Explain the inspection methods and techniques to be utilized
- Description of the inspection interval
- Documentation requirements
- Reporting and follow-up processes

Movable structures should normally receive the same types of inspections as fixed structures, as described in the foregoing. Furthermore, most movable bridges will require additional specialized inspections, such as NSTM, in-depth (movable), underwater diving, and underwater survey inspections. In addition to the structural systems, the operating systems need to be inspected on a routine basis. An annual in-depth inspection which WisDOT calls a Movable Inspection involves the inspection of the complex features of the mechanical and electrical systems of the movable bridge. It is typically most advantageous to perform the annual movable systems inspection in early spring. Specific inspection tasks relative to movable structures are described in detail in Part 3 of this Manual.

1.3.13.3 Complex Feature Inspection Interval

Complex Feature	Inspection Interval
Movable bridge mechanical and electrical elements	24-month Movable Inspection
Cable stay – primary cables and anchorages	96-month Complex Feature Inspection

The inspection TL will consult with the bridge specific PM to determine if the complex feature inspection interval must be decreased based on condition or other factors.

1.3.14 Closed Bridge - Routine Inspection

1.3.14.1 Purpose

Bridges closed to highway traffic that remain in the HSI system as highway bridges fall under two closure situations: (1) closed because of an active rehabilitation or repair project and (2) closed because the condition is unsafe for public use. The status in HSI system must be marked Closed.

Closed Due to Rehabilitation or Repair

Bridges closed because of an active rehabilitation or repair project have the following inspection related requirements.

1. A routine, NSTM, in-depth, and dive inspections (and all activities) are not required to be completed during a bridge closure for rehab.

2. FHWA does not require notification in advance of an inspection that will not be completed because of the bridge closure.

3. The bridge status in HSIS should be changed to CLOSED.
4. Grace dates should be considered and applied when necessary for all inspections due during the bridge closure period.
5. Safety of users on open facilities under a closed bridge and bridge closure system/traffic control for the closed bridge roadway approaches are the responsibility of the contractor and project management as part of the rehabilitation/construction contract.
6. The initial; and when necessary, missed routine, NSTM, in-depth, dive, inspections are **due within 3 months of the bridge opening to traffic**.
    a. The initial inspection only requires the inspection of the elements and condition ratings for what was rehabbed.
    b. A Routine, NSTM, In-depth, and Dive may be required if they were **not** completed because the bridge was closed – all should have been grace dated in HSIS.
7. If work is performed on NSTM or Dive elements during the rehab, the NSTM and/or Dive inspection are due within 12 months of the bridge opening to traffic. As noted in 5a above, the initial is due within 3 months of opening, so for optimization, it is best to perform NSTM, In-depth, Routine, and Initial at the same time.
8. When bridge is reopened to traffic, bridge status in HSIS should be changed to BUILT.

Closed Due to Safety Reasons

Bridges closed for safety reasons must continue to receive Routine inspections entered into HSIS. . Routine inspections will include, at a minimum, the following:

1. Evaluate the traffic control, signage, and closure system(s) recorded under Assessment 9036 Bridge Closure System

2. Evaluate primary structural elements for structural stability.

3. Evaluate safety for users accessing facilities/features on and under the bridge.

4. Notify the bridge owner of any safety concerns that required attention.

Inspections such as NSTM, Complex, Underwater, etc. are no longer required for the closed structure unless those inspections are crucial to ascertaining the stability of the structure in the field. The Regional PM shall be consulted to determine if these inspection types are required for individual bridges.

1.3.14.2 Interval

A routine inspection on closed bridge shall be conducted on a 12-month interval and shall be entered in the HSI system.

1.3.15 Closed Bridge Criteria

1.3.15.1 Introduction

A structural condition or deficiency that poses a threat to public safety requires an immediate full or partial closure of the bridge. A full or partial closure of the bridge is part of the critical finding procedure. Notifications and actions related to the critical finding procedure are described in more detail in Section 1.7. This section establishes the criteria based on bridge inspection findings and the primary reasons for prompt actions to close a bridge. This section also identifies actions to take should a closure be required. An inspection team leader, inspection program manager, or bridge owner should be involved in the decision to close a bridge. If an immediate threat to safety has been identified, the bridge closure or restriction must be completed by any safe means necessary. By adhering to these criteria and conducting regular safety inspections, bridge inspectors and owners can make informed decisions regarding the closure of a bridge to prioritize public safety and infrastructure integrity.

1.3.15.2 Closure Criteria

The following criteria may prompt the full or partial closure of a bridge:

1. **Structural Integrity –**
   - Structural damage or deterioration that compromises the safety of the bridge or the public.
   - Safety inspections that reveal significant deficiencies or safety concerns, may prompt temporary closure until evaluated or necessary repairs are completed.
   - Unarrested active crack in NSTM which has potential to cause member failure.
   - SNBI Component Condition Rating (Item B.C.01-07, 11-15) ≤ 1.
   - Results of a Structural Review recommending closure.
2. **Load Capacity** (see Bridge Manual Section 45.10):
   - Bridges not capable of carrying a minimum gross live load weight of three (3) tons at the Operating level must be closed.
   - When deciding whether to close or post a bridge, the owner should consider the character of traffic, the volume of traffic, the likelihood of overweight vehicles, and the enforceability of a weight posting.

3. **Flood Conditions –** Closure may be necessary and justified during flood events to ensure the safety of the public. Bridges should be evaluated for safety before reopening to traffic after being closed due to flooding.
    - Flood waters over bridge deck or roadway approaches.
    - Flood waters into the superstructure pose a significant risk to bridge stability. Closure may be necessary during flood events to ensure safety. Bridges classified as scour critical have closure triggers identified in a bridge specific Scour Plan of Action.
    - SNBI Component Condition Rating Item B.C.09 Channel Condition Rating ≤ 2.
4. **Emergency Situations:**
    - In the event of emergencies such as accidents, fires, terrorist threats, or natural disasters, closing the bridge may be necessary to ensure public safety or facilitate emergency response efforts.

1.3.15.3 Bridge Closure Actions

All factors requiring bridge closure cannot be anticipated; therefore, the actions listed below are general in nature and may not be applicable to all bridges. The following actions should be implemented when necessary.

1. Call 911 for assistance and notification (when needed). Contact local law enforcement as necessary to assist in setting up closure and detour.
2. Close the roadway over and under the structural elements in question. This can be achieved with vehicles, traffic control devices, and other methods for initial closure. More permanent closure systems and traffic control must be installed in a fastidious, and expedited manner, and in accordance with the MUTCD, supplemented with the WisDOT MUTCD.
3. Contact appropriate owner agency to provide and establish traffic control. Follow the Critical Finding Notification process in Section 1.7.3.2.
4. Bridges over navigable waters -restrict or close the waterway by methods necessary. Notify the US Coast Guard.
5. Bridges over a railroad(s) – contact railroad owner.
6. For state owned bridges, contact the Traffic Management Center (414-227-2166), the region incident management coordinator (RIMC), and region operations/traffic section to coordinate appropriate traffic control and detour.
7. Restrict pedestrian access to the structural elements in question.

Notifications to relevant authorities should be provided with the necessary details of the closure, including the reason for the closure, initial steps taken, and the anticipated duration.

8. Plan for long term actions.

1.3.16 Load Posted Verification Activity

1.3.16.1 Purpose

A Load Posted Verification Activity is a review of the signage associated with a load posted bridge. Bridges not able to carry the State legal loads, as determined by the Statewide Program Manager (SPM), are load posted for reduced live load capacity. These bridges may have been designed for a truck load that is lighter than what is allowed by law today. Otherwise, these

bridges may have suffered some sort of deterioration or damage that has reduced the load capacity below the legal statutory allowable load. In either regard, load posted bridges should be monitored regularly to ensure their serviceability and safety.

Figure 1.3.16.1-2: Load Posted Truss Bridge.

1.3.16.2 Precision

The inspector shall review the signage for the load posted bridge on-site to determine:

- Are all of the required load-posting signs (including advanced warning signs) in the proper locations?
- Are the signs legible, and do they have the correct load posting displayed?
- Does the HSI system have the correct load posting recorded in the data?
- Is Assessment 9034 (Weight Limit Posting) included in the inspection data?

The results of the inspection should be recorded as a Load Posted Verification activity in HSI (an inspection type is not required to be selected), including photos of ALL of the Signs. A new form DT2122 is required when a form is not on file or the load posting has changed. This type of activity does not need to be performed by certified bridge inspectors. A Team Member can be sent out to perform specific inspection or measurement tasks under the direction of an Inspection TL.

The load posting can be found in the HSI system under the Capacity tab and is also included on page 2 of the inspection report under the Capacity Section. Inspectors should review the capacity information with in-service signs and do the following:

1. If the Load Posting (Capacity Section) is blank and no signs are present in the field, Assessment 9034 should not be used in the inspection, and should be removed if it has carried forward from a previous inspection where signs were present.

2. If the Load Posting (Capacity Section) does not match posting signs present in the field, assessment 9034 shall remain in the inspection report, activity "Load Posted Verification" shall be added on the Create/Edit tab in HSI, and the Load Posting Verification Form (DT2122) shall be submitted with the inspection.

3. If the Load Posting (Capacity Section) has a weight limit value but no signs are present, assessment 9034 shall be used with the appropriate missing sign quantities in CS4.

Load posting signs and installation shall comply with Section 2B.59 of the FHWA Manual on Uniform Traffic Control Devices (MUTCD) and the Wisconsin MUTCD. Bridges requiring load posting may also require advance posting signs at the nearest intersecting roads or other points where a driver can detour or turn around.

A bridge located close enough to an intersecting road that the load posting sign can clearly be read from that intersection should have the advanced sign placed at a further approach intersection. Additional advanced warning signs can be placed at other approach road intersections that may generate prohibited vehicles.

1.3.16.3 Interval

This activity only needs to be performed when load postings change on the structure. Load posting signs must follow the Wisconsin Manual on Uniform Traffic Control Devices and shall be installed as soon as possible but no later than 30-days after the owner is notified of the need for the posting change. 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. For structures that have a load posting at any level, form DT2122 – Bridge Load Posting Field Verification must be on record in HSI and must display the current signage. A new form is required when a form is not on file or the load posting has changed. This form shall be submitted immediately after sign installation.

1.3.17 Deck Evaluation Activity

1.3.17.1 Purpose

Over time, bridge decks deteriorate. Routine inspectors use various methods of inspection (both visual and audible) to detect defects in bridges decks.

Chain dragging is a common audible method to help detect delaminations and shall be recorded in HSI under the Deck Evaluation activity. Only when one hundred percent of the wearing surface is chain dragged shall the Deck Evaluation activity be selected in HSI.

A sketch of the delaminated areas shall be uploaded into HSI for the activity. Often times, more in-depth methods are required to ascertain condition. Common methods employed by WisDOT include:

- Ground Penetrating Radar (Part 5, Chapter 10)

- Infrared Thermography (Part 5, Chapter 11)
- Chloride Ion testing (Part 5, Chapter 16)

When these testing methods are used, they shall be recorded in HSI under the Deck Evaluation activity and all reports from the testing shall be uploaded into the activity.

The deck evaluation activity is performed by a team member under the direction and oversight of an inspection team leader at intervals based on information provided in SIM Part 1 Appendix A.

1.3.17.2 Precision

Refer to Part 5 of this manual for anticipated precision of each method.

1.3.17.3 Interval

For state-owned structures, deck evaluation interval is based on the Deck Scanning Policy in Part 1 Appendix A. Local bridge owners may choose to follow a similar interval to aid in the selection of appropriate bridge treatments.

1.3.18 Scour Plan of Action Activity

1.3.18.1 Purpose

The National Bridge Inspection Standards (NBIS) regulation, CFR 650.313(o), requires *bridges determined to be scour critical or have unknown foundations, prepare and document a scour plan of action (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.*

In HSIS, a POA activity needs to be selected and the scour POA document uploaded under the documents/images tab.

1.3.18.2 Precision

In general, a plan of action should focus on providing information that the inspector needs when out in the field. The plan should refer to plans, previous inspector reports, UW-Profile data, and other items pertinent to the bridge.

The POA should detail the foundation type, previous scour history, and monitoring benchmarks for the inspector to assess while in the field. It should also include a bridge closure plan that details when the bridge should be closed, who's responsible for the closure, when the bridge can be re-opened and what detour route should be used during the closure.

1.3.18.3 Interval

The POA document shall be updated every four calendar years or after significant flooding events. This requires a new POA activity entry into HSIS.

A POA is required for bridges with a scour vulnerability (B.AP.03) code of C, D, E, U or 0.

Bridges with a Scour Plan of Action (item B.AP.04) = N or Y (required scour POA) or scour vulnerability (Item B.AP.03) = C, D, E, U or 0 must have a post-flood activity of cross sections (profile) and probing after a significant flooding event or at times as described in an implemented POA. Document and record the cross sections (profile) and probing in HSIS.

1.3.19 Vertical Clearance Measured Activity

1.3.19.1 Purpose

The clearances on or under a bridge shall be measured periodically to determine changes in clearance that affect the mobility of Oversize/Overweight vehicle traffic.

Figure 1.3.19.1-1: Vertical Clearance Photo

1.3.19.2 Precision

Clearances should be taken in every lane, at edge of lanes, edge of paved shoulders, and at barrier edges to determine the low point for vehicular travel. Create the vertical clearance activity in HSI and upload the document(s) under the documents/images tab showing an elevation view photo which includes the locations and dimensions of the vertical clearances.

1.3.19.3 Interval

This activity shall be completed every time there is a construction project on or under a bridge. In addition, after a bridge is hit the inspector conducting the damage inspection should determine if they need to re-measure clearances and if so, enter a vertical clearance verification along with the damage report.

1.3.20 Critical Findings Activity

1.3.20.1 Purpose

A critical finding (CF) is a structural or safety related deficiency that requires immediate action to ensure public safety. Furthermore, a CF is a deficiency found on a bridge which critically threatens the structural stability of the bridge and/or the public safety, and is of such severity
that requires immediate follow-up action. See Part 1, Chapter 7 for more details.

Figure 1.3.20.1-1: Critical Finding Photo

1.3.20.2 Precision

When criterion is met to designate a critical finding, as defined by Part 1, Chapter 7, a critical findings activity shall be entered into HSIS with a completed DT2026 form attached.

1.3.20.3 Interval

This activity is entered into the HSI System on an as-needed basis.

1.3.21 Structural Review Activity

1.3.21.1 Purpose

A structural review is completed by a licensed Wisconsin Professional Engineer to evaluate the observed field conditions and determine the impacts on the load rating and safety of the structure. Structural reviews may include a review of the field inspection notes and photographs, review of as-built plans or analysis as deemed appropriate by the engineer.

Both the owner and designated program manager of the bridge are responsible for ensuring that a qualified individual completes and documents the review in the Highway Structures Information System (HSIS). For locally owned structures, the local bridge owner is required to have a staff engineer, or consultant engineer perform the review (PE required).

1.3.21.2 Precision

Triggering events:

- When a primary structural element is newly observed to be in a severe condition (CS4).
- When the quantity of a pre-existing CS4 primary structural element has increased since the last inspection.
- When the quantity of a pre-existing CS4 primary structural element has not increased, but the severity of the defect has worsened (i.e. section loss from physical measurements increased from 15% to 25% since last review).

Timeline:

- The structural review shall be completed no later than 60-days after the inspection.
- The review must be documented in HSIS; the inspection that documented the defect cannot be signed without the review documentation entered.
- If during the review the defect is determined to be a Critical Finding, the owner agency shall follow the timeline(s) and steps set forth by that policy found in Part 1 Chapter 7.
- Repairs can be performed in lieu of the Structural Review, provided the repair is either a standard repair detail from WisDOT or the repair has been designed by a Wisconsin PE.
- The repair must be completed within 60 days of the finding. The inspection cannot be closed out until the repair is completed, and the plans and calculations are uploaded into the HSI system, along with photographs of the completed repair.

Requirements:

- If the result of the structural review indicates the need for a long-term bridge or lane closure, this shall happen immediately, and the Critical Findings policy and procedures shall be followed. Contact the Statewide Inspection Program Manager.
- If the result of the structural review indicates that the bridge be load posted, or the existing load postings be lowered, the agency shall follow load posting requirements set forth by the Department.

- If the review indicates that the load capacity of the structure is not controlled by the defect, then no action is required.
- In all cases (except error) the defect quantity shall remain in CS4 regardless of the outcome of the structural review until the element is repaired or replaced.

Documentation:
For all triggering events, the results of the review shall be documented in HSIS as follows:

- Under the Inspection tab, when a trigger event is entered as part of an inspection into HSIS for the element and defect, a Structural Review tab will appear. On that tab, the Engineer will enter the following information:
    1. Reviewer name and PE number.
    2. Method of review (engineering judgement, analysis, etc.)
    3. Overall notes pertaining to the review, as well as specific notes for each primary structural element that has a CS4 quantity.
    4. Final recommended actions (load postings, closures, repair, monitoring, etc.)

- If calculations were performed, they must be uploaded into the HSI system.

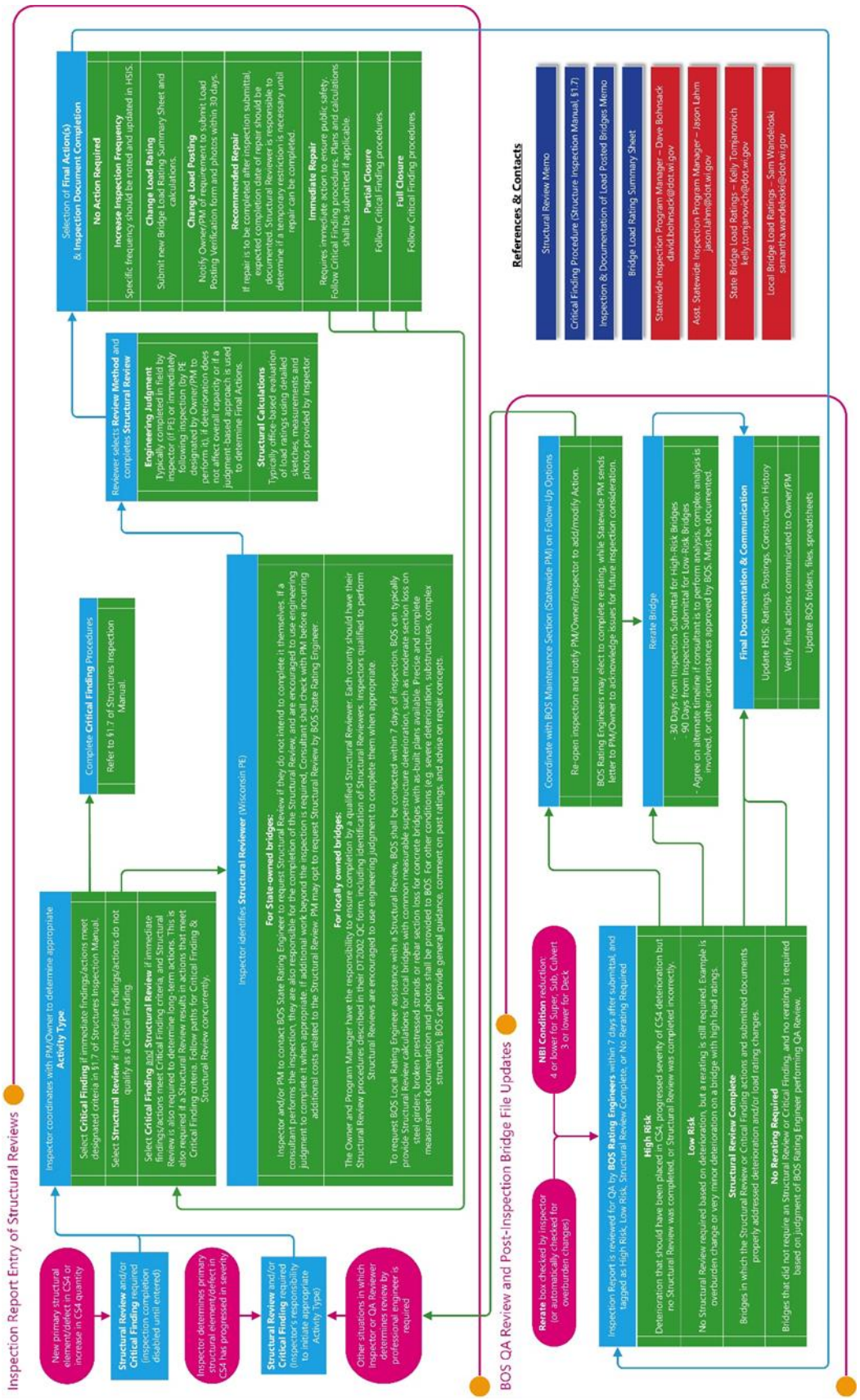
A structural review is **required** for the following element/defect combinations (and will be automatically selected by the HSIS):

- Any steel superstructure, substructure, or culvert element(s) with these Severe (CS4) material defects:
    - (1000) Corrosion
    - (1010) Cracking
    - (1020) Connection
    - (1900) Distortion
- Any concrete (prestressed or reinforced) superstructure or substructure element(s) with these Severe (CS4) material defects:
    - (1080) Delaminations/Spalls/Patch Areas/Exposed Prestressing
    - (1110) Cracking PSC
    - (1130) Cracking RC
    - (1190) Abrasion/Wear
    - (8906) Precast Concrete Connections
- Any timber superstructure or substructure element(s) with these Severe (CS4) material defects:
    - (1020) Connection
    - (1140) Decay/Section Loss/Abrasion/Wear
    - (1150) Checks/Shakes/Cracks/Splits/Delamination
    - (1900) Distortion
- Any masonry superstructure or substructure element(s) with these Severe (CS4) material defects:
    - (1610) Mortar Breakdown
    - (1620) Splits/Spalls/Patched Areas

  - (1640) Masonry Displacement
- Any substructure element with these Severe (CS4) structural defects:
  - (4000) Settlement
  - (6000) Scour
- Any bearing element with this Severe (CS4) material defect:
  - (2240) Loss of Bearing

In all other situations, the inspector may request a structural review for an element/defect combination if he/she feels that the condition warrants a review by a professional engineer.

The following is the structural review process guide to assist inspectors in HSI entry:


Inspection Report Entry of Structural Reviews
New primary structural element/defect in CS4 or increase in CS4 quantity
Structural Review and/or Critical Finding required (inspector completion disabled until entered)
Inspector determines primary structural element/defect in CS4 has progressed in severity
Structural Review and/or Critical Finding required (inspector's responsibility to initiate appropriate Activity Type)
Other situations in which inspector or QA Reviewer determines review by professional engineer is required
Inspector coordinates with PM/Owner to determine appropriate Activity Type
Select Critical Finding if immediate findings/actions meet designated criteria in §1.7 of Structures Inspection Manual.
Select Structural Review if immediate findings/actions do not qualify as a Critical Finding.
Select Critical Finding and Structural Review if immediate findings/actions meet Critical Finding criteria, and Structural Review is also required to determine long-term actions. This is also required if a Structural Review results in actions that meet Critical Finding criteria. Follow paths for Critical Finding & Structural Review concurrently.
Complete Critical Finding Procedures
Refer to §1.7 of Structures Inspection Manual.
Inspector identifies Structural Reviewer (Wisconsin PE)
For State-owned bridges:
Inspector and/or PM to contact BOS State Rating Engineer to request Structural Review if they do not intend to complete it themselves. If a consultant performs the inspection, they are also responsible for the completion of the Structural Review, and are encouraged to use engineering judgment to complete it when appropriate. If additional work beyond the inspection is required, Consultant shall check with PM before incurring additional costs related to the Structural Review. PM may opt to request Structural Review by BOS State Rating Engineer.
For locally owned bridges:
The Owner and Program Manager have the responsibility to ensure completion by a qualified Structural Reviewer. Each county should have their Structural Review procedures described in their DT2002 QC form, including identification of Structural Reviewers. Inspectors qualified to perform Structural Reviews are encouraged to use engineering judgment to complete them when appropriate.
To request BOS Local Rating Engineer assistance with a Structural Review, BOS shall be contacted within 7 days of inspection. BOS can typically provide Structural Review calculations for local bridges with common measurable superstructure deterioration, such as moderate section loss on steel girders, broken prestressed strands or rebar section loss for concrete bridges with as-built plans available. Precise and complete measurement documentation and photos shall be provided to BOS. For other conditions (e.g. severe deterioration, substructures, complex structures), BOS can provide general guidance, comment on past ratings, and advise on repair concepts.
Reviewer selects Review Method and completes Structural Review
Engineering Judgment
Typically completed in field by inspector (if PE) or immediately following inspection (by PE designated by Owner/PM to perform it). If deterioration does not affect overall capacity or if a judgment-based approach is used to determine Final Actions.
Structural Calculations
Typically office-based evaluation of load ratings using detailed sketches, measurements and photos provided by inspector
Selection of Final Action(s) & Inspection Document Completion
No Action Required
Increase Inspection Frequency
Specific frequency should be noted and updated in HSIS.
Change Load Rating
Submit new Bridge Load Rating Summary Sheet and calculations.
Change Load Posting
Notify Owner/PM, if required, to submit Load Posting Verification form and photos within 30 days.
Recommended Repair
If repair is to be completed after inspection submittal, expected completion date of repair should be documented. Structural Reviewer is responsible to determine if a temporary restriction is necessary until repair can be completed.
Immediate Repair
Requires immediate action to ensure public safety. Follow Critical Finding procedures. Plans and calculations shall be submitted if applicable.
Partial Closure
Follow Critical Finding procedures.
Full Closure
Follow Critical Finding procedures.
References & Contacts
Structural Review Memo
Critical Finding Procedure (Structure Inspection Manual, §1.7)
Inspection & Documentation of Load Posted Bridges Memo
Bridge Load Rating Summary Sheet
Statewide Inspection Program Manager – Dave Bohnsack david.bohnsack@dot.wi.gov
Asst. Statewide Inspection Program Manager – Jason Lahm jason.lahm@dot.wi.gov
State Bridge Load Ratings – Kelly Tomjanovich kelly.tomjanovich@dot.wi.gov
Local Bridge Load Ratings – Sam Wandeloski samantha.wandeloski@dot.wi.gov
BOS QA Review and Post-Inspection Bridge File Updates
Rerate box checked by inspector (or automatically checked for overburden changes)
NBI Condition reduction: 4 or lower for Super, Sub, Culvert 3 or lower for Deck
Inspection Report is reviewed for QA by BOS Rating Engineers within 7 days after submittal, and tagged as High Risk, Low Risk, Structural Review Complete, or No Rerating Required
High Risk
Deterioration that should have been placed in CS4, progressed severity of CS4 deterioration but no Structural Review was completed, or Structural Review was completed incorrectly.
Low Risk
No Structural Review required based on deterioration, but a rerating is still required. Example is overburden change or very minor deterioration on a bridge with high load ratings.
Structural Review Complete
Bridges in which the Structural Review or Critical Finding actions and submitted documents properly addressed deterioration and/or load rating changes.
No Rerating Required
Bridges that did not require an Structural Review or Critical Finding, and no rerating is required based on judgment of BOS Rating Engineer performing QA Review.
Coordinate with BOS Maintenance Section (Statewide PM) on Follow-Up Options
Re-open inspection and notify PM/Owner/inspector to add/modify Action.
BOS Rating Engineers may elect to complete rerating, while Statewide PM sends letter to PM/Owner to acknowledge issues for future inspection consideration.
Rerate Bridge
- 30 Days from Inspection Submittal for High-Risk Bridges
- 90 Days from Inspection Submittal for Low-Risk Bridges
- Agree on alternate timeline if consultant is to perform analysis, complex analysis is involved, or other circumstances approved by BOS. Must be documented.
Final Documentation & Communication
Update HSIS: Ratings, Postings, Construction History
Verify final actions communicated to Owner/PM
Update BOS folders, files, spreadsheets

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FUNDAMENTALS OF STRUCTURE INSPECTION

1.4.1 Introduction

The practice of inspecting Wisconsin's structures is complex but well defined. An inspector needs to perform many duties before, during, and after the physical inspection of a structure. To start, inspectors need to meet strict qualifications set by the State, as guided by the federal government. These inspector qualifications set minimum limits on the inspector's knowledge and experience. In addition to the mental requirements, structure inspection can be physically demanding work. Therefore, an inspector needs to be physically fit. Next, inspectors should understand the duties they are expected to perform. These duties range from protecting the public's safety to performing the inspection, preparing the reports, and more.

Inspectors need to understand the various inspection requirements. There are several different types of inspection that can be required for a structure. Some inspections are visual and conducted from a distance, while others are primarily tactile (hands-on) and conducted within the arm's reach of each structure member. Some inspections are conducted every few years, while others may be required every few months. Inspectors need to know which type of inspection is appropriate or required.

Before going out to the field, inspectors should properly plan an inspection. This includes reviewing available data, arranging for qualified personnel and appropriate equipment, scheduling traffic control, and so on.

In the field, good inspectors are meticulous and pay great attention to details. Inspectors need the experience to know what to look for, the knowledge to understand what they see, and the skills to accurately document what is significant. After the inspection is complete, inspectors need to know how to prepare and submit the inspection results to the proper authority.

A structural inspection involves a series of tasks that require dedicated, well-trained inspectors to fulfill them. The purpose of this chapter is to explain the fundamental background that an inspector needs to do the job well.

1.4.2 Duties of Inspectors

1.4.2.1 General Responsibilities

The structure inspector's job is one of great importance. In performing their daily duties, structure inspectors ensure the safety of the traveling public in a tangible way. Few other transportation jobs offer this opportunity. This opportunity comes with several key responsibilities.

The five general responsibilities of a structure inspector are as follows:

1. Maintain public safety and confidence.

2. Protect the public's investment.

3. Provide bridge inspection program support.

4. Maintain accurate bridge records.

5. Fulfill legal responsibilities.

Maintain Public Safety and Confidence

The primary responsibility of the structure inspector is to ensure public safety and confidence. The people and businesses of Wisconsin depend on safe transportation facilities to carry the commercial traffic that fuels our economy as well as the public traffic of daily life. The traveling public has zero tolerance for structure failures. Failures result in property damage, traffic congestion, lost investment in infrastructure, and the potential loss of life. When a structure fails, the public loses confidence in the safety of the transportation network. To alleviate the potential for a structure failure, it is the responsibility of the structure inspector to provide the following:

1. Thorough inspections identifying structure conditions and defects.

2. Condition reports documenting the deficiencies and providing recommendations for remediation.

Structure inspectors often work within the sight of the public and may be under close scrutiny. Inspectors should always exercise responsible behavior. It reflects well on the inspector and Wisconsin Department of Transportation (WisDOT) when inspectors behave professionally. If approached by a curious bystander, the inspector should always be courteous and professional. This helps to reinforce the public's confidence in the work that inspectors do. However, the inspector should not discuss inspection findings unless authorized to release such information by the owner of the structure.

Protect the Public's Investment

Structure failures can and have happened in Wisconsin. In 1989, the Tayco Street drawbridge in Menasha collapsed blocking roadway traffic and navigation between Lake Winnebago and the Fox River's downstream locks. Nobody was hurt, but a large pleasure boat was only fifty feet away from the bridge and moving toward it when the south bascule span came crashing down into the water. The span collapsed because a pin that connected the end of the span to a massive counter weight system failed. It cost several million dollars to replace the bridge. The channel that served as a busy weekend artery for pleasure boaters was closed for several days. Businesses near both ends of the bridge were negatively affected because employees and patrons had to take a longer, alternate route across the river to reach them. The bottom line is that, when a bridge fails, the economic impact is very real and often immediate. The inspector has a duty to detect and report serious defects before they cause failures. Early detection of serious defects can help to ensure public safety and helps to prevent the economic hardships associated with catastrophic structure failures. Had the pin problem on the Tayco Street drawbridge been detected earlier, maintenance and repair action could have been taken. The cost of the maintenance required would have been significantly less than the resulting costs of the failure. Since funding for bridgework comes from tax dollars, it is important that inspectors do their part to minimize the cost to the public. Thorough and regular inspection practices, coupled with timely and consistent maintenance, will save the public money and help to

prevent any loss of property or life. It is also the responsibility of the inspector to detect minor problems in order to correct them early on and prevent more costly repairs in the future.

Figure 1.4.2.1-1: Failed Tayco Street Bascule Bridge, Menasha.

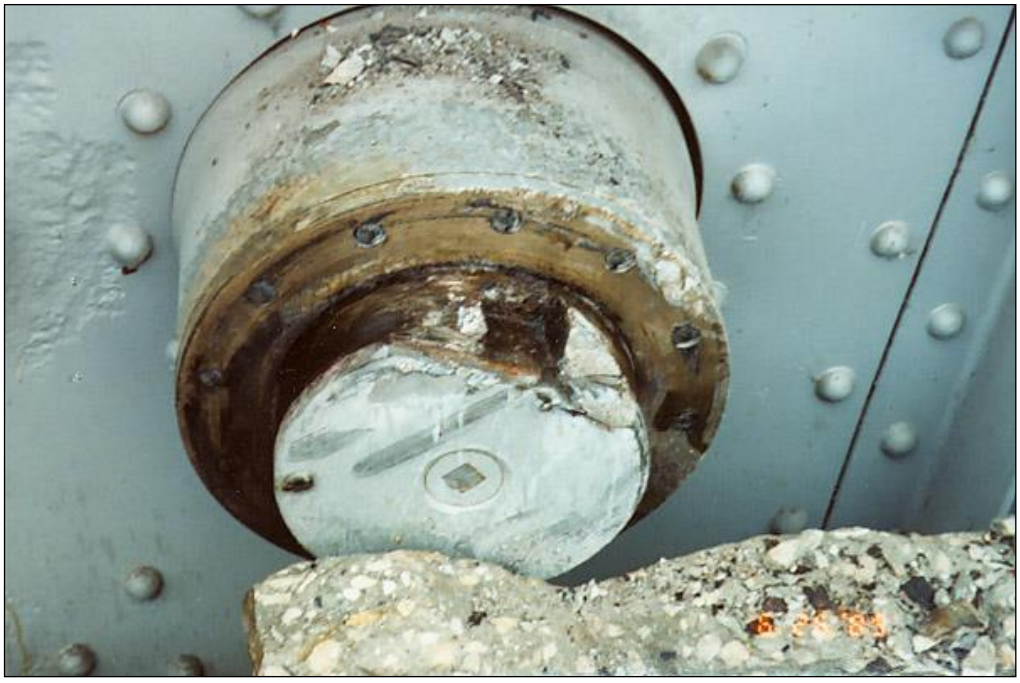

Figure 1.4.2.1-2: Failed Pin, South Bascule Span of Tayco Street Bridge.

Provide Bridge Inspection Program Support

Experienced inspectors should be able to look at a structure and identify elements that are critical and elements that are failure prone. Once the suspect elements are identified, the inspector needs to recognize which of those elements are in need of maintenance, repair or replacement. The knowledge required to make such judgments comes from experience and training. Therefore, it is important that all inspectors meet or exceed the minimum qualification requirements set forth in this manual. All elements in need of maintenance should be identified, located, described, and recorded in sufficient detail to create a reasonable estimate for repair materials and cost. Inspectors need to evaluate the general condition, safety level, and serviceability of each bridge and structure with certainty. No element of the structure should be overlooked during an inspection. At the time of the inspection, the inspector should make recommendations for the maintenance or repair of any problems and deficiencies. The Inspection Team Leader and the Region inspection Program Manager or county inspection Program Manager should review the inspector's recommendations and consider what the most efficient course of action is before implementing the final recommendations. Efficient maintenance and repairs will help to maintain the safety of the structure in the most cost effective manner.

The bridge inspection program is funded through public dollars, therefore the inspector is financially responsible to the public.

The National Bridge Inspection Standards (NBIS) requires all state agencies to create and maintain a bridge inspection program. An inspector is a valued asset within that program that allows it to work efficiently to appropriate funds accordingly through the inspector's diligence in the field.

The inspector is obligated to maintain this work ethic for the public good.

Maintain Accurate Structure Records

The inspector is responsible for providing accurate bridge dimensions and element quantities on the inspection form. The inspector is also responsible for documenting structural deficiencies and maintenance or repair items that are identified in the field. The inspector is also responsible for noting any functional deficiencies. Documentation should be thorough enough that a program manager, load rating engineer or other qualified individual who has not seen the structure can get a good understanding of the condition of the structure. The inspector needs to prepare neat and legible notes and sign the inspection forms. When necessary, the inspection form should be supplemented with additional notes and photographs.

The inspector shall:

1. Verify that all data is correct on the inspection form.

2. Verify that elements are correctly identified and quantified on the form.

3. Spot-check structure dimensions in the field to verify that the dimensions are correct.

Fulfill Legal Responsibilities

The inspection report is a legal document, and proper, standard language needs to be used. The inspector should not use vague adjectives. Concise descriptions that "paint" a picture of the structure should be used. The description should also express the specific location and quantity of the condition. All information will be entered into the WisDOT Highway Structure Information System (HSIS).

A completed inspection report becomes a legal document/record when it is signed by a qualified Inspection Team Leader. By signing an inspection report, the Team Leader is attesting to having been at the structure site and having taken an active part in the inspection of the structure. The Team Leader's signature also certifies that the information in the inspection report is accurate. Federal and state laws, as well as organizational policies, define the qualifications and duties of the Inspection Team Leader. Anyone who signs an inspection report without meeting those qualifications, or signs a report without being at the structure site and participating in the inspection as defined in the American Association of State Transportation and Highway Officials (AASHTO) *The Manual for Bridge Evaluation* may be subject to prosecution for forgery or fraud under Wisconsin Statutes §§943.38 and 943.39 or other applicable state or federal laws.

Disciplinary Actions

Structure inspection ensures the safety of the public and helps to maintain the public's investments. It is a privilege for an individual to carry out this work, and it is expected that all conduct their work in a professional manner.

Unfortunately, there may be some occurrences where an ulterior motive takes precedence over professional judgment. These occurrences may be prohibited to protect the interests of the Program. Occurrences may be any event that conflicts with state and/or federal requirements, or one where reasonable professional expectations of the individual (such as using a reasonable level of professional judgment when inspecting or providing recommendations) are questioned. Occurrences may range from acts of fraud (*e.g.,* recommendations solely based on personal interests) to failure to maintain required training requirements. Other examples of fraud may include untrained inspectors, ultra-conservative ratings to promote future work, and ratings based on the directions of owners to reduce sufficiency rating.

Definition of Status

Active: Individual meets all requirements and is in good standing.

Inactive: Individual is in good standing, but does not meet all requirements.

Disqualified: Individual is not in good standing.

Disciplinary Process

Specifics are located in the Quality Assurance (QA) section of Part 1 Chapter 2 of this manual.

Findings may be forwarded to the Wisconsin Department of Regulation and Licensing, the Wisconsin Department of Justice or the U.S. Department of Justice, if deemed appropriate.

1.4.2.2 Basic Duties

The structure inspector has the following basic duties:

1. Planning the inspection
2. Preparing for the inspection
3. Performing the inspection
4. Preparing the inspection report
5. Identifying safety concerns and items in need of repair or maintenance
6. Recommending re-rating, load posting, or structure closure (follow-up for critical findings)

Planning the Inspection

The goal of the inspector is to be safe, efficient and systematic. One way to accomplish this goal is through proper planning. Planning includes developing an element inspection sequence; establishing personnel requirements and an estimated time frame for the inspection; formatting of field notes; anticipating traffic control, material testing, and bridge access equipment needs; and determining other measures to ensure productive use of time in the field. The previously documented inspection procedures are an excellent place to start with the planning of an inspection.

Preparing for the Inspection

Preparations for a field inspection should include reviewing the most recent inspection report for the structure, reviewing the structure plans, and mapping a route to the work site. All of the necessary inspection equipment and tools should be inventoried and prepared for the field. Good preparation saves the inspector time and saves the owner time and money. A well-prepared inspector can focus on the task of inspecting without worrying about organizational issues.

Performing the Inspection

The inspector should perform such examination, testing, and observation of the structure as required to evaluate its condition, safety, and serviceability. In order to accomplish this goal, the inspector should strictly adhere to the proper element level inspection procedures and NBIS Condition Rating procedures laid out in this Manual.

The inspector should maintain the proper structure orientation and element numbering system and follow the inspection sequence plan. Inspectors should maintain proper traffic control, when required, and adhere to the appropriate safety and fall protection guidelines. The inspector should record enough information to ensure proper recommendations can be made in a comprehensive report.

Preparing the Inspection Report

The report form should be completed in its entirety at the bridge site. It is important that the inspection report be prepared in a style that is easy to read and understand. All writing needs to be legible and appropriate pictures should be labeled, dated, and inserted to clarify the documentation. All photographs not used in the report should be labeled, dated, and kept in the project folder. It is a good quality control practice to have a third party review the form for errors and omissions before submittal. Any mistakes should be crossed out, initialed, and dated. The inspector should never erase any written information on a written inspection form.

Identifying Safety Concerns and Items In Need of Repair or Maintenance

The inspector has a duty to identify safety issues and structure elements in need of repair or maintenance. When a defective element is identified, the inspector should document the defect, locate it on the structure, quantify it, and list the recommended remedial actions on the inspection form. Items requiring immediate attention should be reported to the Region Structure Maintenance Engineer. This helps to ensure public safety and maximize the life of the structure. See Part 1, Chapter 7 (Emergency Notification and Follow-Up Documentation) for structure defects that require prompt action to maintain public safety.

Recommending Re-rating, Load Posting or Structure Closure

The final duty of the inspector may be to recommend a re-rating, load posting, or closure of a structure. This recommendation is based on the findings of the inspection and the experience of the inspector. For a re-rating or load posting, the inspector may need to get the professional opinion of a structural engineer, materials engineer, etc. or request an engineering analysis of the structure be performed. In Wisconsin, structure inspectors (*i.e.,* Program Managers and Team Leaders) have the authority to close a structure that poses an immediate danger to the public or shows signs of imminent failure. When a structure inspector finds a defect or damage to a structure that warrants a structure closure, the inspector should close the structure first and secure the location to ensure that nobody gets injured. The inspector may want to enlist the help of the state highway patrol, county sheriff, or local law enforcement to assist in restricting public access to the structure. Once the structure location is secure, the inspector should follow the appropriate emergency actions as described in Part 1, Chapter 7 of this manual.

1.4.3 Inspection Preparation

The inspection team should develop an inspection plan. Good preparation is an important part of any inspection. Important steps that will help in accomplishing a successful inspection are: planning, scheduling, equipment, personnel requirements, and field inspection procedures.

1.4.3.1 Historical Plan and Structure Review

It is critical to gather all of the historical information that is available. Information that may be available includes:

1. Original design plans

2. "As-built" plans
3. Original shop drawings
4. Construction history
5. Maintenance history
6. Rehabilitation history
7. Bridge inspection reports and photographs
8. Log of openings (for movable bridges)

This information should be reviewed by the Inspection Team Leader (ITL) prior to performing the inspection in order to determine the fatigue prone, failure prone, and nonredundant steel tension members. It is strongly recommended that original plans and documents remain in the office of the owner and that only copies are taken to the field.

1.4.3.2 Inspecting at Railroad Crossings

Region Railroad coordinators and Rails & Harbors:

- WisDOT staff work with the region railroad coordinator to get RR contacts or any RR issue.
- Additional resource for assistance dealing with RR companies is DTIM Railroads & Harbors (Greg Baer or Kris Sommers).

Railroad Company safety Training:

WisDOT staff must not take any railroad company offered safety course or training. State employees cannot take these courses because they have been found to require the "attendee" to indemnify the railroad. Per the Wisconsin State Constitution, WisDOT is not allowed to indemnify any private entity and therefore WisDOT legal counsel has said no state employees can take these railroad courses.

Entering RR Property on Foot:

- State statute 84.01 (10) allows WisDOT employees to enter private lands for inspections. Consultants and county staff will need to have RR permission to enter RR property.
- Stay at least 4' from the tracks when working. Inspectors that need to cross the tracks, cross perpendicular and move at least 4' away from the track to continue work.
- Provide advanced notice to the RR company of future inspections. See WisDOT BOS website forms section for Railroad Notification Letter sample - send on WisDOT letterhead or in an email to the RR contact, which notifies the RR of our inspections.

- When communicating, provide the RR crossing number or RR milepost, both can be obtained through this US DOT crossing inventory listing.

Entering RR Property with Equipment:

- Must be 25' away from the tracks with any motorized or mechanical equipment unless a RR flagger is present.
- If a flagger is needed for the inspection, maintenance, or repair, work with the regions RR coordinator to get a contact for the RR to obtain a flagger.
- Flagging bills go to the county highway department. Notify the county patrol sup that a flagging bill will be coming. This is charged to the RMA.
- Some county highway personal may know the local RR roadmaster so sometimes you can get a RR flagger working through them.

Safety First:

Make sure you follow WisDOT safety protocols for PPE – safety vests and pants, steel toed shoes, when required - hardhats and safety glasses.

Additional Advice:

1. When on RR property, get in, and get out. The statute allows us to enter to complete our work. Do not loiter or have do paperwork or have long discussions that could be completed off RR property.
2. Do not get into an argument with RR company employees. You can provide them with the letter notifying the RR company of your work and the state statute that allows you to be there. If they continue to insist you are trespassing, leave and contact your supervisor.
3. Be alert when on the RR property
    a. At some locations, trains are operated remotely, there may not be an engineer in the engine.
    b. Look both ways before crossing - trains come from either direction.
    c. Train locomotives may be quieter than you realize.
4. Never step on the rail – The RR staff gets very upset when they see someone step directly on the rail. Just don't do it, no need to stir up more problems.

Many of the permit and safety requirements that apply to inspections over railroads also apply when inspecting bridges over navigable waterways. The U.S. Coast Guard (USCG) must be notified if an inspection may reduce navigable clearances (*i.e.,* snooper truck platform or arm under bridge). The USCG oversees Wisconsin from two district offices as illustrated on their website (http://www.navcen.uscg.gov/?pageName=lnmMain). The inspection team leader should coordinate with the appropriate local USCG Marine Safety Office (MSO)/Dispatch (MSO Milwaukee, WI (414) 747-7161, MSO Sturgeon Bay, WI (920) 743-9448, MSO Duluth, MN (218) 720-5286, MSD St. Paul, MN (651) 290-3991) as well as notify the USCG District Office (Western Rivers: USCG District 8, d8marineinfo@d8.uscg.mil Phone: (504) 589-6277; Great Lakes: USCG District 9, thomas.c.perez@d9.uscg.mil Phone: (216) 902-6073) so that a "notice to mariners" can be published.

Figure 1.4.3.2-1: Inspectors Working Next to a Live Railway.

Inspectors should be aware of the rules and policies pertaining to confined space entry. The Occupational Safety and Health Administration (OSHA) definition of a confined space is as follows:

> "Confined space" means a space that:
>
> (1) Is large enough and so configured that an employee can bodily enter and perform assigned work; and
>
> (2) Has limited or restricted means for entry or exit (for example, tanks, vessels, silos, storage bins, hoppers, vaults, and pits are spaces that may have limited means of entry); and
>
> (3) Is not designed for continuous employee occupancy.

Inspectors should be aware of confined spaces. Inspection of the following structures can involve confined space entry:

1. Steel or concrete box girders
2. Steel arch rings
3. Arch tie girders

4. Cellular concrete structures

5. Long culverts

Three main issues must be addressed when inspecting inside of a confined space.

1. **Lack of oxygen** (human beings require an oxygen content above 19% to remain conscious)

2. **Toxic gasses** (paint fumes, exhaust, pollutants, smoke, carbon monoxide)

3. **Explosive gasses** (natural gas, methane, petroleum fumes)

When a confined space must be entered, the following safety precautions should be heeded:

1. Test the air in the space for oxygen content and toxic gasses before entering. Continue to test the air at fifteen-minute intervals while the space is being occupied.

2. If necessary, properly ventilate the space before and during occupancy

3. Adequate lighting and lifeline equipment should be used.

4. One inspector should remain outside of the space and remain in contact with the inspector inside. The inspector outside the space should never attempt to rescue the inspector inside. The outside attendant should call the fire department or 911 in the case of an emergency.

5. Keep running vehicles and exhaust creating machinery downwind of the space to avoid filling the space with carbon monoxide or other harmful exhaust fumes.

Inspectors should determine whether the structures they need to inspect have confined spaces. If yes, the inspector needs to contact the owner of the structure or space and determine what requirements and policies exist regarding entry of confined spaces. The inspector should also meet with their employer to discuss what requirements and policies the employer has regarding confined spaces. OSHA has strict requirements regarding confined space entry and these requirements are the minimum acceptable standard for confined space entry. The OSHA standard for “Permit-required Confined Spaces” can be found in Chapter 29 of the Code of Federal Regulations, Part 1910.146. It is in the inspector’s best interest to follow all of the pertinent rules and policies regarding entry of confined spaces. These spaces can be deadly if an inspector does not follow the proper procedures.

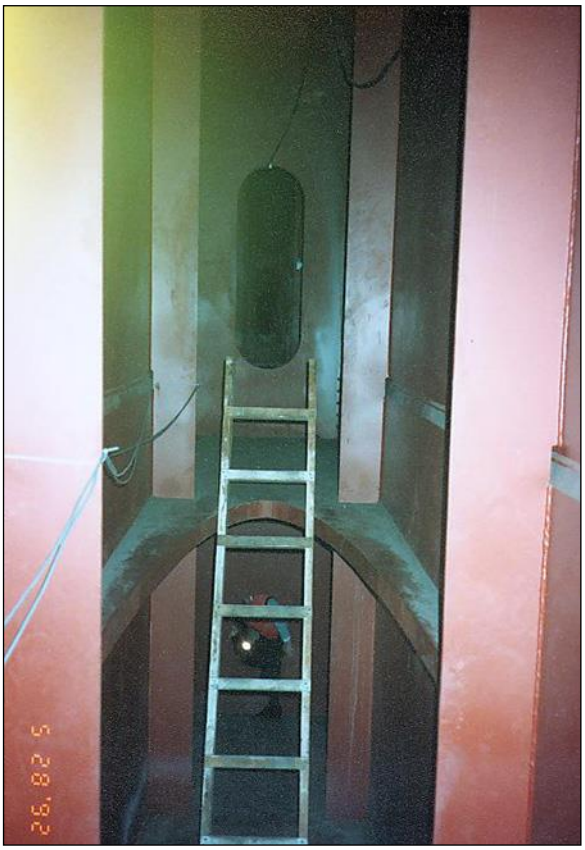

Figure 1.4.3.2-2: A View Inside of a Steel Box Girder.

1.4.3.3 Identification of Critical Details

Identification of Fatigue Prone Details

Fatigue failure of a material is due to the formation and propagation of cracks resulting from the cyclic application of loads. Fatigue cracks initiate from points of stress concentrations in structural members or details. Stress concentrations can result from:

1. Material flaws
2. Type of connection detail
3. Out-of-plane distortions

It is absolutely necessary for inspectors to understand the causes and effects of fatigue and be knowledgeable on all fatigue prone details. If left unchecked, a fatigue crack can propagate to a size where it may trigger a fracture in a structural member.

Material Flaws

Metal structures, particularly those that are welded, can contain flaws. Flaws can vary in size from undetectable non-metallic inclusions to large inherent cracks. Material flaws may exist

as external flaws such as pits or internal flaws such as nonmetallic inclusions or lamellar tears.

Fabrication Flaws

Fabrication flaws can also be introduced. Fabrication flaws can include:

1. Damage at the edges of drilled or punched holes
2. Gouges/notches from flame cutting
3. Gouges/notches from grinding operations
4. Sharp corners at coped or blocked details

Fabrication flaws due to welding are also common and include such phenomenon as:

1. Incomplete fusion
2. Slag inclusions
3. Porosities
4. Blow holes
5. Undercuts
6. Intersecting welds
7. Craters
8. Hot and cold cracks
9. Back-up bars or tack welds left in place
10. Plug welds in misaligned bolt holes
11. Arc strikes

Careless material handling during transportation or erection can create flaws such as:

1. Nicks
2. Notches
3. Indentations
4. Chain marks

Service Flaws

Flaws can be created while a structure is in service, as well. Vehicle impacts to the tension flange of a steel girder can induce tears or grooves into the steel. Corrosion can cause severe pitting.

The structure inspector should be aware of these types of flaws and note the existence of any such flaws found in the field. The inspector should also pay close attention to such flaws in tension zones of members and note any cracks that form at these locations.

Type of Connection Detail

"Type of detail" refers to the stress condition in a member or connection. Various details have different fatigue strengths associated with them. Typical details have been grouped into a number of categories (A through E') based on their respective fatigue strengths. See Part 2 Appendix D, Fatigue Prone Details for more detailed information. Members or details should be inspected in order of fatigue susceptibility (*i.e.,* E' details first and A details last). Type D, E, and E' details are the most susceptible to fatigue cracks and should be examined carefully. The location and condition of these members and details should be thoroughly documented on the inspection report form. The following are the most common fatigue prone details and should be closely inspected during In-Depth and nonredundant steel tension member Inspections:

- Groove Welds
    1. Older structures with web or flange groove welds made prior to nondestructive evaluation during fabrication.
    2. Longitudinal stiffeners on girder webs.
    3. Groove welds between intersecting longitudinal stiffeners and members.
- Ends of welded cover plates on tension flanges
    1. Cover plates with end welds: Cracking can develop at the toe or throat of the weld and most likely at mid width of the flange
    2. Cover plates without end welds: Cracking can develop at the end of the longitudinal weld and the flange tip.
- Ends of attachment plates welded to girder flanges/webs or truss members
    1. Welded splices or lateral gusset plates (details similar to cover plates)
    2. Perpendicular attachments welded to the member: These are more fatigue resistant than cover plate-like details.
- Diaphragm Connections

1. Ends of diaphragm connections that are not welded to the flange: Cracking may develop at the cope in the web-to-flange weld or near the top of the connection plate.

2. Ends of riveted angles attached to girder webs: Cracks may develop horizontal in the web near the flanges or vertically in the angle.

- End Connections of Floor Beams

    1. Copes and blocked ends of floor beams: Cracks may occur at the re-entrant angle of the cope or blocked flange, especially when the re-entrant angle is flame cut, with re-entrant notches.

- Floor Beam Brackets

    1. Bracket connections to girder webs: These have similar details to diaphragm connections

- Stringer to floor beam connection

    1. Cracking may occur in the coping

    2. Cracking may occur in the connections.

- Top and Bottom Lateral Bracing Connections

    1. Gusset plates welded to girder web or flange: Cracking may occur in the web gap at the toe of the weld.

    2. Gusset plate to diaphragm connection plate welds: Areas of intersecting welds create high restraint which increases probability of fatigue cracking.

- Transverse stiffeners

    1. Not diaphragm or floor beam connections. These details are fairly resistant to fatigue. Cracking may occur at ends of cut short stiffeners. Excessive web vibrations may result in cracking. For fitted stiffeners, movement can be depicted from observing the paint film.

- Truss bridge floor beams

    1. Connections to verticals: Cracks may develop in connection angles or at rivet/bolt heads.

    2. Connection to lateral bracing: Cracks may occur in horizontal gusset plates connecting the laterals to the floor beam flanges, often near the bearings

- Truss bridge verticals and diagonals

1. Verticals near bridge ends: Cracking may occur at first vertical member at the top near the gusset plate (typically starting at a rivet hole). Cracks may also occur at the floor beam connection.

2. Eye-bar members: Cracks may develop at pin holes where a lap forge exists. Cracking may also occur at the shank to head transition.

- Pin-connected links or hangers

1. Eye-bar links: Cracking may occur at the ends of pin holes, at the width transition or the end of the bar.

2. Hanger plates: Cracking may occur at pin holes or at edge of plate. Special attention is needed when corrosion and fixity are apparent due to in-plane bending

- Tack Welds

1. Welds between main structural components or gusset plates are common sources of fatigue.

- Plug welds
- Burn Holes
- Attachments (with welded connections)

Out-of-Plane Distortions

The majority of fatigue cracks result from out-of-plane distortion across a small gap, usually a segment of a girder web. The problem of distortion-induced fatigue cracking has been observed in the following types of bridges:

1. Trusses
2. Suspension bridges (floor system)
3. Two-girder bridges
4. Multi-beam and multi-girder bridges
5. Tied arch bridges
6. Box girder bridges

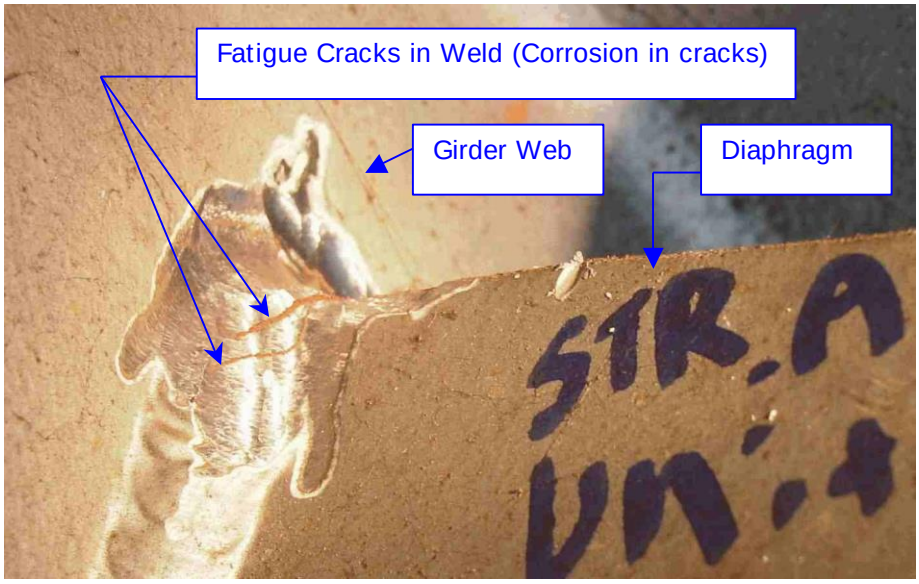

Figure 1.4.3.3-1: Fatigue Cracks in Weld Due to Out-of-Plane Distortion.

Identification of Failure Prone Members and Details

Certain members are more prone to failure than others are. The following are examples of members that are failure prone or members where failure would lead to accelerated deterioration of the overall structure:

Expansion Joints

There are two key failure modes for expansion joints: leaking and seizing. Leaking occurs when joints allow water to pass through the deck and spill onto the superstructure and substructure units. This can result in accelerated deterioration of the superstructure and substructure units near joints. Seizing occurs when joints fill up with non-compressible debris and can no longer facilitate expansion. The debris resists expansion, which results in the development of high stresses in the superstructure and at bearings. The resulting damage may include joints breaking out of back walls or decks, bearing failures or misalignments, or pier or abutment seat cracks.

Bearings

Bearing failure can have several consequences. A failed high-rocker bearing can kick out and result in the bridge superstructure dropping several inches. Frozen bearings can result in restricted thermal expansion forces, which are transformed into stresses. These stresses are typically very large and could result in overstress of members and accelerated deterioration of superstructure and substructure elements.

Figure 1.4.3.3-2: Failed Bearing at Pier.

Figure 1.4.3.3-3: Damage Resulting From Failed Joint.

Identification of Non-Redundant Steel Tension Members

Nonredundant Steel Tension Members (NSTMs) and member components should be identified and shown on the structure plans or a sketch of the structure to be submitted with the NSTM Critical Inspection forms. See Section 2.4.2.4 for a more complete description of NSTMs.

NSTMs have all or part of their cross-sections in tension. Frequently, cracking is a result of fatigue occurring near a weld, a material flaw, and/or changes in member cross-section. See Part 2 Appendix B for a review of typical fatigue prone details.

After the crack occurs, failure of the member could be sudden and lead to collapse of the bridge. For this reason, steel bridges with the following structural characteristics or components should be reviewed for a NSTM Inspection.

1. Overhead two-truss bridges
2. Low two-truss bridges (pony trusses)
3. Deck two-truss bridges
4. Thru-girder bridges
5. Two-girder bridges
6. Tied arch bridges
7. Movable bridges
8. Steel pier caps and cross-girders
9. Pin and hanger system on two-girder systems

See Part 2 Appendix D for examples of NSTM bridges, components, bending definitions, typical crack locations, and typical pin and hanger parts.

1.4.3.4 Inspection Plan

The inspection plan is the final step in preparation for the field inspection. From the information gathered, a plan needs to be organized for the field inspection. A pre-inspection visit to the site may be required to develop or finalize the inspection plan.

Visual inspection is intended to be the primary examination method. For NSTM, In-Depth, and Damage Inspections, it is required that each critical member or member component needing inspection be inspected hands-on, from a maximum distance of one arm's length for the entire length of the member and/or member component. Non-destructive evaluation techniques should be used to identify internal defects or hard to see external defects in critical members.

An inspection plan may include some or all of the following and should be prepared prior to the field inspection:

1. A brief historical fact statement.
2. Essential plans that would help with field inspection.
3. Identification of fatigue prone, failure prone and/or NSTMs on inspection form or attached plan.
4. Access equipment and personnel needed to perform the field inspection.
5. Inspection tools and safety equipment needed to perform the field inspection.

6. Summary of required non-destructive evaluation equipment.

7. Traffic control requirements.

8. Estimate of inspection time.

9. Coordination with and notification of owner and other agencies.

To meet the minimum requirements, all of the required information should be noted in appropriate locations of the applicable bridge inspection report forms. On larger or more complex structures, it may be necessary to create individual sections for each of the required areas of the inspection plan, which can be attached to the bridge inspection report form.

1.4.3.5 Special Considerations

The Inspection Team Leader has many things to consider when planning an inspection. Several key considerations are listed below:

Peak travel times

In urban areas, an inspection that requires traffic lane closures may be restricted to certain hours of low traffic volume. Some days, such as holiday weekends, may be banned from any traffic restrictions. In either case, it may not be possible to accomplish a forty-hour inspection workweek. The Inspection Team Leader should be aware of this and plan the inspection work accordingly.

Setup time

Setup time can be significant and needs to be considered when planning an inspection. For example, actions such as rigging can take several days to complete before the inspectors arrive on site. Traffic control setup may take an hour or so each day before inspection work can start. Equipment, such as manlifts or compressors, may need to be setup and torn down or stowed daily. Time should be allotted for these things in the inspection plan.

Access

Access requirements must be considered when planning an inspection project. If access equipment is needed, the Inspection Team Leader will need to decide which kind of equipment would be most effective and determine whether that equipment is available. The Inspection Team Leader should estimate the number of days or weeks the equipment will be needed. Keep in mind that although structure elements may appear similar, it may require more effort and time to access some versus others. Simple obstacles such as utility lines, fences, adjacent buildings, and cross bracing can make access more difficult.

Overall Condition

It takes longer to document and inspect an element that is deteriorated. Making the necessary sketches, taking photographs and measurements, etc. is time consuming. The inspector should use old inspection reports to determine the overall condition of the structure and estimate the time required for inspection and reporting accordingly.

Weather

Unexpected weather conditions can adversely affect structure inspection work. Rain and wind can make structures slippery or make manlift operation unsafe. Snow and fog can make traffic restrictions impractical and unsafe. Commonly, structures in low lying, wet areas will have fog cover until mid-morning. Any host of adverse weather conditions can potentially slow or halt a structure inspection. Depending upon the structure location, time of year, and local weather patterns; the inspector should try to anticipate what types of weather could potentially slow inspection work and provide a contingency plan to compensate.

Permits

Inspection of structures that pass over or through railroad right-of-way, structures that span navigable waterways, and structures with confined spaces may require permits. The Inspection Team Leader is responsible for determining whether or not a permit is required, obtaining the necessary permits, and following all rules or regulations stipulated by the permitting agency.

1.4.4 Field Inspection Procedures

Field inspection procedures are the implementation of the inspection plan and are documented on the inspection report. Good preparation will increase the quality of the field inspection and ensure that all needed tools, safety devices, and operational procedures are available to effectively and efficiently complete the task. It is critical that the Inspection Team Leader (ITL) guide the field inspection process to assure that each inspection is done safely and to the desired level of quality.

A certain degree of uniformity in inspection techniques and recording is required for efficiency and completeness. No element of the structure should be overlooked and the time spent on each element should be in direct proportion to the importance of the element. The inspector should systematically record all observations that may affect the structure “today” or cause concern in the future.

1.4.4.1 Historical Review

The construction history, along with any rehabilitation and maintenance history, should be reviewed at the bridge site prior to performing an inspection. This will be helpful in possibly defining deficiencies that may be found during the inspection. It also is advantageous to know the age of possible deficiencies to determine how critical they are when making final recommendations.

1.4.4.2 Critical Member Plan Review

The critical members that have been identified on the inspection plan should be reviewed prior to performing the inspection. Critical members are fatigue prone, failure prone or nonredundant steel tension membershe inspector should look at the existing bridge member and member connection details to determine whether any of those details are known to result in specific problems or deterioration. In addition, the inspector should determine if any repairs to or rehabilitation of the structure might have influenced the deterioration of other parts of

the structure. Locations where the potential for member cracking exists should be identified and highlighted on the inspection plan.

Bridge orientation should be evaluated to determine the location of critical members. The location of panel points in relation to bridge orientation should be determined.

1.4.4.3 Element Inspection Sequence

It is paramount that an inspector be thorough. When elements go unseen for extended periods, the structure is subject to localized failures. An easy way to ensure that no element is overlooked is to develop an element inspection sequence map. Structure plans are a great place to start. Identify all of the elements on the structure and formulate a logical sequence for inspection. Check members off on the plan as they are inspected.

The cardinal directions are north and east. Abutments and piers should be numbered from lowest to highest going from the south increasing to the north or from the west increasing to the east. Pier columns and longitudinal superstructure elements such as girders or stringers should be numbered from left to right on a bridge cross section when looking north or east.

If the structure elements are already identified and numbered on a set of structure plans, repair plans, or in a previous report, that same identification and numbering system should be used for continuity. The inspector should document an abnormal numbering system.

1.4.4.4 Traffic Control

Traffic control requirements should be reviewed prior to performing an inspection to assure safety of the inspection team and the traveling public (see Section 1.4.7). The Inspection Team should review the traffic control requirements with the people setting up the traffic control, if they are not part of the Inspection Team. The traffic control requirements should be documented on the inspection report form. Any unsafe traffic control conditions should be corrected before performing the inspection.

1.4.4.5 Access Equipment and Procedures

The inspector should anticipate the need for access equipment. For typical Routine Inspections, access equipment may not be required. If access equipment is needed, it needs to be evaluated to determine if it can provide the required visual “hands-on” inspection and non-destructive testing of all critical members and/or components.

Typical methods of access include but are not limited to:

1. Deck-parked, under-bridge inspection vehicles
2. Ground-parked aerial lifts (manlift, etc.)
3. Scaffolding and staging
4. Boats
5. Ladders

6. Climbing

Figure 1.4.4.5-1: Access Equipment.

Access equipment, safety features, and procedures need to be evaluated prior to inspection. The safety sheet provided with the equipment, including emergency evacuation procedures, should be reviewed and understood.

The Wisconsin Department of Transportation (WisDOT) owns and operates three under-bridge inspection units. Only trained WisDOT personnel are authorized to operate the units. These units may be used upon mutual agreement between the Region Program Manager (RPM) and the Statewide Program Manager (SPM). See Section 1.4.7 for a more complete description of equipment available from WisDOT.

1.4.4.6 Personal Safety

All personal safety equipment needed for an inspection should be identified and checked for condition. Such equipment may include, but is not limited to, high visibility clothing, body harnesses, hard hats, safety shoes, eye protection, ear protection, respiratory protection, and protection from hazardous paint or other materials.

Safety of the inspector is essential in providing a quality inspection. Providing for all safety requirements is paramount to maintain the confidence of the inspector and to ensure a quality inspection.

Figure 1.4.4.6-1: Body Harness/Tie-Off.

1.4.4.7 Inspection Tools

A review of the structure and its condition should be performed to determine what tools may be required to perform a thorough visual, “hands on” inspection and any needed nondestructive evaluation (NDE).

For a more detailed list of typical equipment for all types of inspection, see Section 1.4.5.

1.4.4.8 Field Inspection

For a Routine Inspection, all members can be inspected from the ground or a suitable work platform as long as the inspector has a good enough view to determine with certainty that all structure members are functioning properly. Members should not be cracked or sufficiently deteriorated to adversely affect the serviceability of the structure. For inspections other than Routine Visual, all critical pertinent members and components which require hands-on inspections need to be identified. For In-Depth, Damage, and Special Inspections, members and components may need to be inspected hands-on. For NSTM Inspections, the inspector shall inspect the critical pertinent members and components hands-on. All members and/or components should be cleaned, as necessary, so that all extraneous material is removed to provide a clear view of the underlying member.

The condition of each member and/or component needs to be determined, including any deficiencies such as section loss, cracks, impact damage, unspecified welds, field welds, tack welds, sharp bends or kinks, and other unspecified damage.

Non-destructive evaluation (NDE) may be required to further analyze defects. An inspector needs to meet the qualifications established in Part 1, Chapter 2 in order to perform an official NDE investigation. The following nondestructive evaluation procedures may be required as directed in Section 1.3.5 and Section 1.3.6:

1. Ultrasound for analysis of pins, welds, and cracks
2. Dye penetrant for surface flaw analysis
3. Magnetic particle for surface flaw analysis
4. Ultrasound thickness gauge for section loss analysis
5. Resistograph for timber elements with internal defects
6. Thermal Imaging Camera for shallow delamination in concrete elements

Deficiencies that require emergency repairs or action should be reported immediately by using the emergency notification procedures provided in Part 1, Chapter 7. If the ITL has doubts about the load-carrying capacity of a bridge when such deficiencies are found, they should take action to close the bridge and use the emergency notification procedures detailed in Part 1, Chapter 7 of this manual. ITLs do have the authority to close a bridge when, in their judgment, it is unsafe for continued usage. Local law enforcement or the State Patrol may be called to assist in bridge closure. The SPM or RPM have the authority to re-open a bridge. All bridge inspection reports should be signed and dated by the ITL.

1.4.5 Photograph Procedure

Photographs are an extremely useful tool for the inspector. Photographs are foremost a window to view a bridge component or a portion thereof as it was during a previous inspection. This is a powerful comparative tool that can indicate to the inspector whether the deterioration is worsening and requires a more thorough inspection, or little to nothing has changed and the rating of the element can remain as is.

1.4.5.1 Photograph Policy

All bridge files shall have a minimum of two photographs depicting the structure

- Roadway or approach view
- Elevation or side view

The inspector is responsible to ensure these photographs are within the HSI system. The direction the photo is looking is at the inspector's discretion but must be clearly noted in the photo description when uploading to HSI. These two required photos are for comparative purposes. The photos allow the inspector to observe any changes that may have occurred on or around the structure since the previous inspection. Moreover, one of these two photos shall be chosen by the inspector as the cover photo for the inspection report.

The inspector is not required to keep up to date cover photos (roadway or elevation photos) with every inspection, however, it is highly recommended. Photographs are a useful visual tool in allowing the inspector to compare the state of the current structure and its surroundings to the past inspections.

1.4.5.2 Element Level Photograph Policy

Elements with defects in Condition State 3 or 4 shall have an associated photo taken and included within the inspection report. Each defect in Condition State 3 only requires a general photo. Note this is for *each* defect in Condition State 3. All areas exhibiting Condition State 4 defects shall have an associated photo. The photo should be taken as close to the defect as possible. Larger areas exhibiting Condition State 4 defects should be broken up and appropriately labeled for reference.

Photographs of Condition State 3 and 4 defects shall be taken every inspection and uploaded to the HSI system.

Inspectors are encouraged to use WisDOT's Inspection Photo Best Practice Guidelines.

1.4.6 Inspection Equipment

An inspector needs the correct tools to do their job effectively and efficiently. The information provided in this section should help guide inspectors to select the correct equipment for their work in the field. Most of the tools that the inspector will use can be grouped into the following eight basic groups:

1. Personal Safety Gear
2. Cleaning Tools
3. Measuring Tools
4. Visual Aid Tools
5. Inspection Tools
6. Documentation Materials
7. Access Equipment and Methods
8. Specialty Inspection Equipment

1.4.6.1 Personal Safety Gear

1. Hard Hat
2. Steel-toed/steel shank shoes or boots
3. Work gloves

4. Reflective safety vest-Be sure the vest conforms to current OSHA and MUTCD standards.
5. Safety glasses – when working near flying particles (*i.e.,* using a wire brush, grinding).
6. Safety harness and up to two lanyards – when working over water or traffic, or when working at heights over 6 feet, a harness and lanyard should be worn and the lanyard must be tied to a stationary object such as a safety line or a solid structural member.
7. Life jacket – when working near water.
8. Safety boat – when working over water for long periods.
9. Hip boots or waders – when working in relatively shallow, slow moving waterways
10. First aid kit
11. 2-way radio or cell phone – for emergency communication
12. Flashing light / beacon – on the inspection vehicle to warn the public of inspector's presence
13. Dust mask
14. Respirator – when required
15. Sun screen
16. Insect repellent

1.4.6.2 Cleaning Tools

1. Whisk broom
2. Shovel – for removing large amounts of debris
3. 2" Scraper – for removing corrosion or plant/fungus growth
4. Steel wire brush – to remove light corrosion, for use on steel only
5. Brass wire brush – for use on non-steel
6. Spray primer – the inspector should prime steel that he scraped or brushed the paint off of
7. Flathead screwdriver – for scraping or probing

1.4.6.3 Measuring Tools

1. 100-foot tape

2. 6-foot rule
3. 4-foot level – for measuring cross slopes and approach settlement
4. 2-foot level – to check for plumbness
5. Calipers or D-meter
6. Micrometer
7. Feeler gauges
8. Optical crack gauge
9. Plumb bob – use with a rule to measure vertical alignment
10. Telescoping vertical clearance rod or electronic distance measuring device
11. Thermometer

1.4.6.4 Visual Aid Tools

1. Binoculars
2. 5x or 10x magnifying glass
3. Flashlight
4. Telescoping, hinged inspection mirror – to see inside box members or to see around corners

1.4.6.5 Inspection Tools

1. Geologist's hammer
2. Sounding chains – to check a horizontal reinforced concrete surface for delaminations (or thermal imaging camera)
3. Pocketknife
4. Ice pick – to probe wood for rot
5. Incremental borer – used to take core samples of wood elements
6. Tool belt
7. Probing rod –to check streambed for scour holes
8. Protractor – use with plumb bob to determine tilt angles; use to determine skew angles

1.4.6.6 Documentation Materials

1. Inspection forms
2. Extra paper
3. Clip board
4. Straight edge – to keep notes and drawings neat
5. Laptop computer or Tablet computer– for field data entry or report writing
6. Pencil/pen
7. Permanent marker
8. Paint stick/marker – for marking on the structure
9. Lumber crayon – for marking on the structure
10. Camera (Required with elements/defects in Condition State 3 or worse)

1.4.6.7 Access Equipment and Methods

1. Extension ladder
2. Man lift
3. Bucket truck
4. Truck mounted, moveable, under-bridge work platform
5. Rigging –structure mounted cables and platforms; used where lifts are impractical
6. Scaffolding
7. Boats, rafts or barges
8. Climbers or spiders – mobile inspection platforms that “climb” steel cables; well suited for tall vertical surfaces
9. Boatswain chair – one-man chair suspended by a rope and raised/lowered from above with a block and tackle system
10. Free climbing – some structures can be free-climbed by inspectors who have experience with the appropriate climbing procedures, ropes, and equipment. This method should be used only upon approval of the structure owner.

1.4.6.8 Specialty Inspection Equipment

1. **Non-Destructive Evaluation Equipment**: Refer to Part 5.

2. **Underwater Inspection Equipment**: Refer to Sections 1.3.7 and 1.3.8.

3. **Paint Inspection Equipment**

   a. Cutting tool (for X or cross cuts; e.g., pocketknife)

   b. Magnetic paint thickness gauge

   c. Cross cutting template & pressure sensitive tape

   d. Tooke Gauge

4. **Access or Lift Equipment**

5. **Confined Space Entry Equipment**: Consult Safety Directives and consult Occupational Safety and Health Administration (OSHA) policy.

6. **Fall Protection Equipment**: Consult Safety Directives and consult OSHA policy

7. **Survey Equipment**: Survey equipment may be required in special cases when movement in a structure needs to be measured in relation to a reference point. Use of transits, levels, etc. may be required.

8. **Mechanical/Electrical Inspection Equipment**: Refer to Part 3.

1.4.7 Methods of Access

Gaining access to structures can be challenging. The proximity of a structure to live traffic, a navigable waterway, an active rail line, overhead utilities, etc. can make inspection access difficult. The inspector should always use good judgment and be safe. All non-participating inspection vehicles and equipment should be parked off the roadway. Ideally, a driveway, parking lot or even a large gore area can be safer than parking along the roadway. If no such suitable area exists near the structure, then park on a nearby roadway with the least traffic or the widest shoulders. Be certain not to park in front of any signage that may reduce or block visibility for the travelling public. If other means are impractical and appropriate traffic control exists, inspection vehicles or equipment can be parked inside the safe work zone.

Access fence along freeways may be gated somewhere near a bridge. Most access gates are locked with Wisconsin Department of Transportation (WisDOT) locks. Inspectors may get keys to these locks upon request from the Region Structure Maintenance Engineer.

Crossing a live roadway is dangerous, especially on well-traveled roads. Exercise common sense and be safe. When working in or around active railroad corridors or navigable waterways, make sure to contact the railroad company or coast guard and follow all required safety and access requirements. This may include obtaining special permits. Information that should be documented in the structure inspection procedures.

The WisDOT currently has three under-bridge access vehicles available for use out of the Bureau of Structures (BOS) Maintenance Section. This equipment is scheduled for use by regional offices, counties, and locals for use in structure inspection work. Typically, counties and local government units will make scheduling requests to their regional office. The region

will compile all of the requests and forward them to the WisDOT BOS Maintenance Section. The objective of the WisDOT BOS is to keep this equipment scheduled and utilized for as much of each calendar year as possible, depending on the weather. The annual operations schedule are developed by the WisDOT BOS Maintenance Section personnel from requests received by February 1 of that year. Changes to the schedule can be and are made as needed throughout the year. Each inspection vehicle needs to be properly staffed with an experienced crew.

Each WisDOT under-bridge access unit shall be operated by a minimum of two trained people. For safety reasons, one person should be in the under-bridge basket and one person should be on the bridge deck or in the cab of the truck at all times. The person on the bridge or in the truck cab and the person in the bucket shall be trained to operate the vehicle. The under-bridge access unit shall observe all posted weigh limits on bridges. The WisDOT BOS Maintenance Section has staff trained to operate these vehicles. Regions and local government units should arrange to have their own inspector on the vehicle crew to work as a Team Member/Leader with WisDOT BOS Maintenance Section personnel. The region will arrange through the county or others to provide personnel, proper lane closure, traffic control equipment, a blocking truck, and communication for bridge vehicle operations. The region will also arrange press releases when appropriate, to alert local motorists to the presence of on-going bridge inspection work that may impede traffic.

WisDOT region offices may also schedule and provide traffic control through the counties for consultant structure inspectors.

1.4.8 Safety Practices

The safety of the public and of structure inspection personnel is paramount. Accidents can cause injury, suffering, family hardship, and even death. Accidents also cost money in terms of lost equipment, lost production, medical expenses, and possible litigation costs. Therefore, inspectors should always strive to create and maintain a safe working environment.

1.4.8.1 Inspection Team Safety

In order to minimize the chance of an inspector becoming injured, all inspectors in the field should adopt the following work habits:

1. Never work alone, when possible.
2. Be well-rested and alert.
3. Stay in good health and maintain a level of good physical conditioning.
4. Be familiar with and use the proper tools.
5. Keep work areas neat and uncluttered.

Inspectors ideally should operate in groups of two or more when performing any inspection fieldwork. It is good practice for inspectors to carry a mobile phone or a radio for use in case of an emergency. Inspection personnel should also have basic first aid training. It is recommended inspectors be familiar with cardiopulmonary resuscitation (CPR). The

American Red Cross and American Heart Association (AHA) offer training for CPR. To minimize the risk of a traffic accident, proper procedures for traffic control should be followed at all times. See Section 1.4.8.

Inspectors should be familiar with federal (Occupational Safety and Health Administration (OSHA)) and local safety standards and requirements. OSHA requirements are minimum standards and should be followed. Requirements that are more stringent may exist, depending on the activity, structure owner, etc. The inspector is responsible for knowing this information and adhering to the applicable requirements and standards. OSHA requirements and standards can be found at:

https://www.osha.gov/

Appropriate personal protective clothing should be worn at all times including vests, safety glasses, hardhats (where needed), and appropriate footwear. Proper hand, hearing, sight, breathing, and face protection should be used whenever appropriate. For instance, many of these protection devices are needed when scraping paint and corrosion and brushing bird droppings from any inspection area or during the use of any manual or power tools. All equipment, safety devices, and machinery should be kept in good operating condition.

Inspection vehicles, under-bridge access equipment, trucks, and vans should be operated in accordance with the operating manuals provided by the manufacturer. Personnel should be trained in the safe use of the vehicles and emergency procedures in the event of an equipment failure. These inspection vehicles need to be inspected and maintained in accordance with the manufacturer's recommendations.

Lanyards, safety harnesses, life jackets, and other personal safety equipment should be used in accordance with applicable standards. All safety equipment should be inspected regularly and in good working order. Worn or damaged equipment should be discarded. In addition, inspection personnel should be cautioned to keep safety equipment clean and away from potentially harmful chemicals such as gasoline, dye penetrant, and/or oil.

Proper safety precautions and confined space entry procedures should be employed when entering the interior of a box girder, vaulted abutment, long culvert, etc. Air quality monitoring, air exchanges, and/or the use of portable air packs may be required. The OSHA standard for "Permit-required Confined Spaces" can be found in Chapter 29 of the Code of Federal Regulations, Part 1910.146. It is in the inspector's best interest to follow all of the pertinent rules and policies regarding entry of confined spaces. These spaces can be deadly if an inspector doesn't follow the proper procedures.

There is no safety guideline that can replace common sense and good judgment. Each structure site is unique. If unusual working conditions exist, specialized safety precautions may be required and shall be documented in the structure inspection procedures.

1.4.8.2 Public Safety During an Inspection

To minimize the chance of the public becoming injured, all inspectors in the field should do the following:

1. Use appropriate traffic control for vehicles and pedestrians when required.

2. Maintain the traffic control (watch for tipped or moved cones, signs, etc. and replace/realign them as needed).

3. When working at heights, make sure all tools and equipment are secure and cannot fall onto people or traffic below. A tool pouch, belt, or bucket is a good way to secure tools and items. Hardhat chinstraps are also recommended to prevent hardhats from falling.

4. When inspecting members, make certain not to push or drop debris (loose concrete, paint chips, pack rust, etc.) onto people or traffic below.

Inspectors should be aware of the traffic and pedestrian traveled ways when performing any inspection fieldwork. If one inspector is performing aerial work (in a manlift, free climbing, etc.), it is good practice to have another inspector on the ground. The ground inspector should observe vehicle and pedestrian traffic and relay pertinent information, such as stray vehicles or pedestrians in the work zone, to the aerial inspector. The ground inspector can also take notes for the aerial inspector and guide him around the structure. By following this procedure, the aerial inspector can concentrate on inspecting, and the ground inspector can protect the public near the work zone. The aerial and ground inspectors should have radios in order to communicate. It is a good practice for at least one of the crewmembers to carry a mobile phone or a radio for use in case of an emergency.

Inspection vehicles such as under-bridge access units, trucks, and vans should be kept away from live traffic. Manlifts, etc. should not be extended over live lanes of traffic. Manlifts should not be operated directly adjacent to live traffic (*i.e.,* on a narrow shoulder) without providing appropriate traffic control. Lanes should be closed if access to structure members over live traffic lanes is necessary. Trucks and vans should be parked away from live traffic. Good places to park include on the edge of or off the shoulder, in the wide part of a large gore area on an adjacent low volume roadway.

Most importantly, should an inspector discover any defect in a structure that would make imminent failure of the structure likely or pose a danger to the public, the inspector has the authority to close the structure. Closing a structure would apply to structures like bridges and culverts that are integral to traveled roadways. When light poles, sound walls, retaining walls, or highway sign bridges are found to be in dangerous conditions, the inspector should assess the danger level and respond appropriately to prevent any injury or property damage. The state police or local sheriff's department can provide assistance in road closures or cordoning off dangerous areas. If any structure is closed, the Inspection Program Manager needs to be contacted immediately after securing the area.

1.4.9 Traffic Control

1.4.9.1 Introduction

Inspection operations that obstruct traveled ways can create unexpected and unusual situations for motorists. Effective traffic control eliminates surprises and routes traffic safely around any hazards, inspection personnel, or equipment. Wisconsin has amended the federal *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) in the Wisconsin Department of Transportation (WisDOT) Facilities Development Manual (FDM). The FDM contains the standards for traffic control procedures and equipment in Wisconsin.

The FDM is available from the WisDOT Bureau of Highway Operations and the federal MUTCD is available on the Internet at http://mutcd.fhwa.dot.gov/. The FDM dictates the minimum traffic control requirements and the proper use of standard traffic control devices for various applications. The FDM also contains plan sheets with plan views of traffic control layouts for typical applications. These plan views detail the types of signs or message boards to use, the geometry of channelization devices, flagger requirements, etc.

Figure 1.4.8.1-1: Barrels Channel Traffic Away From Work Zone.

1.4.9.2 Traffic Control Plan

Traffic control plans are required for any inspection work that will adversely affect the smooth flow of traffic through the work zone. A traffic control plan should be designed by the Inspection Team Leader (ITL), Inspection Program Manager (IPM), or a qualified employee of a traffic control subcontractor. A traffic control plan is a plan view drawing of the proposed work zone that shows where traffic control devices will be placed, what devices will be used, and how they will be oriented. All parties that will be operating in the work zone should review and be familiar with the traffic control plan. If the traffic control plan changes, the changes need to be approved by the ITL or IPM and all affected parties should become familiar with the changes prior to working in the field.

1.4.9.3 Fundamentals of Traffic Control

Traffic control should be designed and implemented to accomplish the four following goals:

1. **Inform the motorist** of changing conditions or hazards ahead.
2. **Control the motorist** and reduce traffic speed in the work zone.
3. **Provide a clearly marked path** to create a smooth traffic flow through the work zone.

4. **Use positive protection** such as crash trucks and attenuators to protect the work zone.

Figure 1.4.8.3-1: Truck Mounted Crash Attenuator.

Inform the Motorist

There should not be any surprises. Proper use of signs can keep the motorist informed of unexpected conditions. Several types of signs are used. They are as follows:

1. **Regulatory Signs:** speed limit, do not pass, etc.

2. **Warning Signs:** this lane ends, work zone ahead, etc.

3. **Guide Signs:** these show motorists the direction they are supposed to travel and tell them how to reach their destination. These signs are only used when a detour is involved.

Control the Motorist

The same safety principles used to design roadways should be applied in the design of traffic control. The geometry, signs, and lights used for traffic control should resemble the unobstructed roadway conditions as closely as possible. This minimizes the confusion for the driver. Channelizing devices are used to guide drivers safely through a work zone. They also warn drivers of hazards ahead in the roadway or just off the roadway. Channelizing devices include but are not limited to traffic cones, barrels, barricades, and wands. Wind and traffic can misalign or knock over channelization devices. Therefore, channelization devices need to be monitored to ensure that they are in place, according to plan, and functional. Speed limit signs can be used to slow traffic in the work zone and increase work zone safety.

Figure 1.4.8.3-2: Directional Arrow Board and Channelizing Barrels.

Provide a Clearly Marked Path

Channelization devices should be used properly to define a clear definite path for drivers to follow. It should be obvious to the driver where he is supposed to go. There should be no confusion. Channelization devices should also provide drivers with a smooth, gradual transition from one lane to another, onto a bypass or a detour or through the narrowing of a traveled lane or shoulder. Abrupt changes in traffic direction and abrupt constriction of the traveled way should be avoided. Warning signs are also used to inform the motorist of unusual or changing conditions. All traffic control devices should be promptly removed once they are no longer needed.

1.4.9.4 Typical Traffic Control Plans

See the WisDOT FDM for typical traffic control plans and traffic control requirements.

1.4.10 Lane Closure System (LCS)

To aid in traffic control on the Wisconsin Highway System, the WisDOT's Statewide Traffic Operations Center (STOC) implemented the Lane Closure System in April 2008. The STOC's mission is to manage congestion and improve transportation safety, mobility, and efficiency on state highways. The LCS is an important and required tool for agencies and engineers requiring access to highly trafficked bridges on the highway system.

The LCS is a web-based system used to track closures and restrictions on Wisconsin state highways. The data accumulated through the LCS is used for traffic analysis and patterning. The LCS data is incorporated into 511 Traveler Information, Inconvenience Maps for public use, and Oversize/Overweight (OSOW) permitting. Finally, the LCS fully integrates historical traffic flow and capacity information which is used to determine available lane closures and the appropriate time at which they can occur to create as little impact on the travelling public as possible.

The LCS coordinates activities throughout the state to prevent back-ups and potential conflicts such as lane closures during special events or conflicting lane closures on the same highway.

As an inspector, lane closures for inspections are common place. However these closures must be accurate and approved with enough time to notify the traveling public. Lane closures involving any planned maintenance on Interstate, state or major state highways, or an emergency closure are required to be entered into the LCS.

Notification requirements to the LCS depend on the roadway system type. The following list indicates when the LCS should be notified for each type:

- 7 days Project start, full roadway closure, or restriction of width, height or weight
- 7 days System ramp closure
- 3 days Lane and service ramp closure

Acceptance of a requested closure depends on the roadway system the closure is on. All freeway and expressway closures are required to be accepted by a WisDOT Regional Traffic Supervisor (RTS).

The LCS is accessed via the internet. Therefore users outside of the WisDOT can log on with a username and password. For more information on and access to the LCS refer to the following URL:

http://transportal.cee.wisc.edu/closures/

The Traffic Operations and Safety Laboratory also provides LCS training. Refer to the following URL to request user privileges and access the training.

https://transportal.cee.wisc.edu/training/applications/WisLCS/

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1.5 REPORTING SYSTEMS

1.5.1 Introduction

There are many different mechanisms available to record, organize, and catalog bridge inventory and inspection data. Over the years, several bridge management systems and tools have been developed to aid structure owners in cataloging their bridge inventories. Wisconsin has developed the Structures Highway Information System (HSIS) to house and maintain data on its inventory of bridges and ancillary structures. Wisconsin uses two distinctly different reporting systems in collecting and managing its structure inspection data: Element Level and National Bridge Inventory (NBI) System.

The WisDOT Highway Structure Information System (HSIS) is maintained by the Bureau of Structures (BOS) located in the Wisconsin Department of Transportation (WisDOT) Central Office and warehouses data on all public bridges in Wisconsin. This system consists of a database of inventory data, Inspection data (NBI and element level data), electronic directory and storage of supplemental information files (approved format such as .pdf, .doc, etc.), and is utilized to create the annual National Bridge Inventory (NBI) file. As mandated by the Federal Highway Administration (FHWA), the NBI file is created annually to submit Wisconsin's Bridge data in the format described in the FHWA Coding Guide for all public bridges in Wisconsin.

Beginning in October 2014, the FHWA required all State DOTs to begin collecting element level condition data in addition to the NBI file. All State DOTs were required to begin reporting element level inspections on NHS bridges to the FHWA by April 2015.

Although a hardcopy paper bridge file may be located at each region for state bridges and located at each county for their local bridges, the electronic file within HSIS shall be designated as the "Official Bridge File" following NBIS Metric #15: Inspection procedures: Bridge Files. It is imperative the bridge file is maintained accurately.

Bridge inspection data must be updated in the bridge file (inspection report completed in HSIS) within 3 months after the month the field portion of the inspection is completed. WisDOT requires the inspection report be created in HSIS within 28 days after the month the in which the field portion of the inspection is completed.

1.5.2 Structure Identification

The Wisconsin Department of Transportation (WisDOT) follows an alphanumeric coding system to identify the various bridge and ancillary structures in the WisDOT right-of-way. In some cases, multiple I.D. plaques are attached to designate the separate components of the structure, such as structure number, lighting circuit, and sequence decal. Refer to the WisDOT Facilities Development Manual (FDM), Chapter 16, Standard Detail Drawings (S.D.D.).

1.5.2.1 Bridges, Small Bridges, Noise Barriers, and Retaining Walls

These structures are identified using a name plaque. Refer to S.D.D. 12a3 in the WisDOT FDM for details of the plaque. The ID code found on the plaque is in the form "X-CC-NNNN-UUUU", where "X" identifies the structure type (B-bridge, C-small bridge, N-noise barrier, L-

High Mast Light pole, or R-retaining wall); “CC” is the two-digit county number; and “NNNN” is the unique four-digit structure number. However, if there are unused leading zeroes, these may be omitted (*i.e.,* B-40-60).

Some longer bridges are subdivided into units. On these structures, the ID code will be “X-CC-NNNN-UUUU” where “UUUU” is the unique four-digit unit identifier.

1.5.2.2 Sign Structures and Overhead Sign/Signal Support

Sign structures and overhead sign supports are identified using a structure plaque. The plaque is either in a vertical or horizontal configuration dependent on whether or not the sign bridge is structure mounted. Refer to S.D.D. 12a4 in the WisDOT FDM for details of the plaque. The ID code found on the plaque is in the form “S-CC-NNNN”, where “S” designates a sign bridge; “CC” is the two-digit county number; and “NNNN” is a four-digit region-provided location number.

If the sign bridge contains lighting, a circuit plaque and sequence decal will also be present. The circuit plaque is mounted to the sign bridge and the sequence decal is mounted to the luminaire. Refer to S.D.D. 10a3 in the WisDOT FDM for details of these plaques. The circuit plaque ID code is in the form “A-B-CD”, where “A” and “B” identify the two circuits that the structure is on; and “CD” identifies the distribution center. The sequence decal code is in the form “AZZ”, where “A” is the circuit pole and “ZZ” is the two-digit luminaire sequence number.

Figure 1.5.2.2-1: Structure Plaque and Circuit Plaque Mounted on a Sign Bridge. (Note that the location number “300” would read “0300” according to current convention.)

1.5.2.3 High Mast Lighting

High mast lights are identified using four separate plaques, the structure plaque, circuit plaque, luminaire sequence plaque, and the north plaque. The structure plaque and circuit plaque are mounted to the hatch door, the sequence decal is mounted to the luminaire and the north plaque is mounted to the light ring. Refer to S.D.D. 10a4 in the WisDOT FDM for

details of these plaques. The ID code found on the plaque is in the form “CC-NNNN”, where “CC” is the two-digit county number, and “NNNN” is a four-digit region-provided location number. The circuit plaque ID code is in the form “A-B-CD”, where “A” and “B” identify the two circuits that the structure is on; and “CD” identifies the distribution center. The sequence decal code is in the form “AZZ”, where “A” is the circuit pole and “ZZ” is the two-digit luminaire sequence number. The north plaque is simply a single letter “N”.

Figure 1.5.2.3-1: Structure Plaque and Circuit Plaque Mounted on a High mast Hatch Door. (Note that the location number “028” would read “0028” according to current convention.)

1.5.3 Element Levels

Each structure element, (*e.g.,* wing walls, steel girders, prestressed concrete beams, decks, slabs) has been assigned a unique element number. Structural elements are listed on the structure inspection form with their associated element numbers.

Each element has a list of possible defects associated with the element and its particular material (steel, concrete, timber, etc.). For instance, the defect “Exposed Prestressing” is found under prestressed concrete elements but absent from reinforced concrete elements.

Elements are then grouped into condition states that reflect the level of element deterioration as based on the defect condition states. All elements have a possibility of four condition states. These condition states are described in detail in Part 2 of this manual as well as in the WisDOT Bridge Inspection Field Manual.

1.5.4 National Bridge Inventory (NBI)

NBI is an acronym for National Bridge Inventory. NBI ratings are based upon the item numbers and conditions set forth in the Specifications for the National Bridge Inventory *(SNBI)*. The NBI reporting system is presently used in Wisconsin to determine the sufficiency number for the bridge and not to rate the individual elements of each structure. An NBI inspection looks at the bridge differently than an element level inspection. Where an element level inspection considers each bridge element separately, the NBI inspection lumps all like-function elements together into a functional group. For example, on a steel girder bridge, the girders, floor beams, and stringers together would be considered the superstructure. Likewise, the abutments, piers and pier caps would be considered the substructure, and so on. Please see the SNBI found on the FHWA website for more information regarding the NBI.

1.5.5 Nonredundant Steel Tension Member

To aid the inspector in performing Nonredundant steel tension member (NSTM) Inspections, Wisconsin has developed NSTM Inspection Report Forms; DT2010 and DT2011. Other supplemental forms can be found in 1.3.5.5. These are supplements to the Bridge Inspection Report entry and can be uploaded into HSIS during inspection data entry. Form DT2010 allows for specific field inspection notes for each member/member component. The final inspection report may have several pages of this form. The inspector may utilize form DT2011 to sketch the bridge and identify the NSTM. Much of the required information for a NSTM Inspection is the same as that required for element level reporting.

Each NSTM shall have its condition recorded. If no deficiencies are noted, an “OK” should be recorded. Additional information should be recorded for serious deficiencies and placed on attachment inspection forms. Such information may be additional narrative, sketches or photographs. The description of such deficiencies should include exact location and detailed dimensions that could help in determining the overall condition rating of the bridge.

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1.6 FIELD QUALITY CONTROL/QUALITY ASSURANCE

1.6.1 Introduction

An Inspection Team Leader has to keep track of a lot of information, from the planning phase of the inspection all the way through to submittal and/or data entry of the completed reports and forms. Given the complex nature of structure inspection work and the low tolerance for mistakes, it is imperative that structure inspectors use quality control and quality assurance methods to reduce the risk of errors or omissions. Refer to Section 1.2.5 for Program Quality Assurance.

1.6.2 Inspection Fieldworks

1.6.2.1 General

NBIS Metric #12: Inspection procedures – Quality Inspections states:

> Each bridge is inspected in accordance with the national recognized procedures in the *AASHTO Manual for Bridge Evaluation (MBE)* contributing to quality assessments, ratings, and documentation, as measured by the following criteria:
>
> - Condition codes within generally acceptable tolerances,
> - All notable bridge deficiencies identified, and
> - Condition codes supported by narrative that appropriately justifies and complements the rating or condition state assignment.
>
> A qualified team leader is at the bridge at all times during each initial, routine, in-depth, nonredundant steel tension member (NSTM) and underwater dive inspection.

The field investigation of a structure should be conducted in a systematic, organized, and efficient manner that minimizes the possibility of any structure element being overlooked. To achieve this objective, consideration should be given to standardizing the sequence for inspection of a structure.

Defects found in various portions of the structure will require thorough investigation to determine and evaluate their cause. The cause of most defects will be readily evident; however, it may take considerable time and effort to determine the cause of some defects and to fully assess their seriousness.

If possible, the inspector should observe bridges, culverts, and bridge-mounted sign structures during passage of heavy vehicle loads to determine if there is any excessive noise, vibration, or deflection. If a problem is detected, the inspector should investigate further until the cause is determined. Careful measurement of line, grade, and length may be required in this case. The seriousness of the condition can then be appraised and corrective action taken as deemed necessary.

Possible fire hazards near the structure should be noted, such as accumulations or storage of debris, combustibles, drift wood, brush piles, weeds, and garbage. If the land under or

around the structure is utilized by the public for other purposes, make a note of it on the report form and comment if any other safety issues are present.

1.6.2.2 Field Measurements

Field measurements are made to provide baseline data on the existing structure components and to track changes, such as crack width and length, which may occur over time.

Measurements may be required on structures for which no plans are available or to verify data shown on plans. Measurements are to be made only with sufficient precision to serve the purpose for which the measurements are intended. Unnecessarily precise measurements lead to a waste of time and a false sense of value of the derived results. The following limits of accuracy are generally ample for field measurement of overall lengths:

Timber Members	Nearest 1/4 inch
Concrete Members	Nearest 1/2 inch
Asphalt Surfacing	Nearest 1/2 inch
Crack Width	Necessary accuracy to identify future growth
Steel Rolled Sections	Necessary accuracy to identify section
Span Lengths	Nearest 0.1 foot

When plans are available for a structure that is to be load rated, dimensions, member types, and member sizes will normally be taken from the plans. However, many of the plans are not as-built plans and may not reflect all changes made to a bridge. Sufficient checking must be done during a field inspection to insure that the plans truly represent the structure before the plans are used in structural calculations. Special attention should be given to changes in dead load, such as alterations in deck geometry, additional overlays, and/or new utilities. Increased dead load may affect the load rating for the structure.

Measurements sufficient to track changes in joint opening, crack size, or rocker position should be made and recorded. Measurements to monitor suspected or observed substructure tilting or movement may also be required. In these cases, it is necessary that permanent markings be made on the structure and recorded in the field notes by the inspector to serve as a datum for future readings. A log of the readings should be kept in the inspection file and updated with the new readings after each inspection cycle. Direct measurements of the surface area and depth and location of defects are preferred to visual estimates of percentage loss.

1.6.2.3 Cleaning

It is a good inspection practice to clean selected areas to allow close, hands-on inspection for corrosion, deterioration or other hidden defects. Debris, vegetation, fungus, marine growth, vines, litter, and many other obscuring coverings can accumulate and hide problem areas. Some bird waste may harbor fungi that are harmful if inhaled. Respiratory protection equipment should be used when deemed appropriate.

On metal structures, particularly on NSTMs, it may be necessary to remove cracked or peeling paint for proper inspection. Laminate or pack rust will require chipping with a hammer or other means to remove the corrosion down to base metal. If the overall paint system on an element is good, provisions should be made to recoat the bare metal areas exposed during the inspection. A coat of spray primer or cold galvanizing would be sufficient.

On concrete structures, leaching, lime encrustation, and debris may cover heavily-corroded steel reinforcing. Debris on piles can obscure heavy spalling or cracking. Vegetation can also obscure large defects such as cracks or spalls. All obscuring debris and vegetation should be moved aside or removed so that the inspector can see the obscured elements.

Timber structures are particularly susceptible to insect damage and decay in areas where debris causes a wet/dry condition. Inspectors should give particular attention to cleaning and carefully inspecting such areas, especially when a wet/dry condition is present near the end grain.

1.6.3 Report Preparation

The inspection report is the only documented permanent record of the inspection. The inspection report is considered a legal document, and all rehabilitation and replacement work decisions are based on the information it contains. Therefore, it is imperative that the inspection report has accurate and thorough information. Reports should include photos, sketches, addenda, or whatever is necessary to adequately and thoroughly document defects. On the other hand, the inspection report should be as concise as possible while still containing all of the relevant information. Do not include sketches, photos, etc. that are not necessary to communicate the nature of a defect. Likewise, when conditions are good/excellent and are so noted by the appropriate rating number, it is not necessary to provide narrative comments.

The HSIS is capable of uploading all necessary photographs for each bridge inspection. Insignificant photographs or additional photos of a particular defect may be uploaded but not selected to appear within the inspection report. Each bridge should have a roadway, and elevation photograph. Refer to 1.4.5 for additional information on the Bridge Inspection Photography Policy.

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1.7 CRITICAL FINDING PROCEDURE AND DOCUMENTATION

1.7.1 Critical Finding

A critical finding (CF) is a structural or safety related deficiency that requires immediate action to ensure public safety. Furthermore, a CF is a deficiency found on a bridge or portion thereof, discovered either by bridge inspection or notification by the public, which critically threatens the structural stability of the bridge and/or the public safety, and is of such severity that requires immediate follow-up action. Deficiencies such as damage, corrosion, section loss, settlement, cracking, deflection, delamination, loss of bearing, or any condition that threatens the integrity of primary structural element(s) or poses an imminent threat to public safety are considered a CF. Critical finding must be resolved as soon as possible but no later than 30 days of the finding.

1.7.2 Critical Finding Designation

The criteria for designating a critical finding are listed below. It is a requirement to report any finding or event meeting this criterion as a critical finding.

- Structural conditions or deficiencies that pose an immediate threat to public safety which results in the immediate partial or full closure of the structure.
- An Nonredundant Steel Tension Member (NSTM) inspection condition to be rated in serious or worse condition (Item B.C.14 ≤ 3).
- Deck, superstructure, substructure, or culvert component to be rated in the critical or worse condition (Items B.C.01, B.C.02, B.C.03, or B.C.04 ≤ 2, respectively).
- Channel or scour condition to be rated in the critical or worse condition (Item B.C.09 or B.C.11 ≤ 2, respectively).
- Immediate load restriction or posting, or immediate repair work to a bridge, including shoring, in order to remain open.
  (1) An immediate posting is when instead of a partial or full closure, the inspector, with consultation with the PM, bridge owner, and possibly the load rating engineer, decides to immediately post the bridge before an official review or rating is completed. An immediate posting is considered a short-term response.
  (2) An immediate repair is one that requires immediate action to ensure public safety. An immediate repair can also be a full or partial depth hole in the deck, if the inspector determines a partial closure is required until the repair is complete.

Potential events/incidents which may lead to a critical finding designation are as follows:

- Structural deficiency of primary structural element(s) which threatens the overall integrity of the structure (Bridge closed ASAP; this may require bridge replacement or major rehabilitation)
- Structural deficiency of primary structural element(s) that requires a partial lane and or shoulder closures for an extended duration (Partial closure ASAP; partial closure shall remain until repairs, rehabilitation, or replacement can occur).
- Component condition requires the immediate load restriction or posting (Load posting signs shall be installed no later than 30-days after the owner is notified – Owner is responsible to have Load Posting Verification Activity completed).
- Other Safety deficiencies (Movement, Natural Disaster, Bridge Hits, etc.)
- Scour around a substructure unit that could conceivably cause the failure of the structure.
- Structural review meeting the criteria above.
- Non-destructive evaluation meeting the criteria above.

1.7.3 Critical Finding Procedure

1.7.3.1 Immediate Action:

Upon identification of a critical finding, the inspection team leader (TL) will immediately assess the safety of the bridge and impact to the public. The TL may decide to close all or a portion of the bridge until further analysis can be performed. The immediate actions taken by the inspector may vary with the circumstance. The TL may require remedial work be performed or a load posting installed immediately depending on the seriousness of the finding. A critical finding actions must be resolved as soon as possible, typically in less than 30 days. This does not necessarily mean the deficiency has been corrected. It means an action has been taken to address any immediate safety concern.

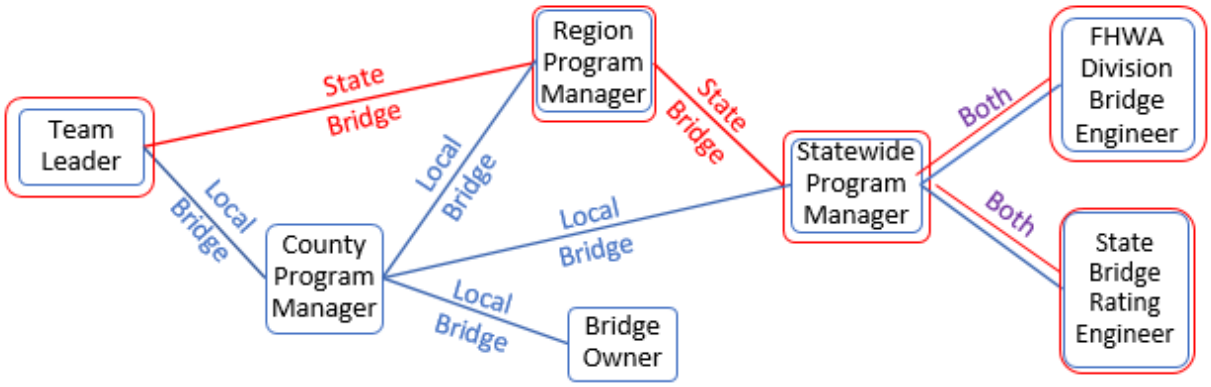
If the action is a load posting or change of an existing load posting, the signage must be installed as soon as possible, but no later than 30 days after the determination of the need for the posting.

1.7.3.2 Critical Finding Notification

1.7.3.2.1 Notification Process

Once the immediate safety concern has been addressed, the TL will notify the County (CPM) or Region Program Manager (RPM) with jurisdiction over the bridge, who will notify the WisDOT Statewide Program Manager (SPM). The CPM will notify the owner of the structure and the RPM. The SPM will notify the FHWA Division Bridge Engineer and the State Bridge Rating Engineer. Direct communications through telephone notification and a follow-up email is required to properly document discussions.

Notification Tree:



During the notification process, the inspector, PM, owner, and others will discuss a plan of action, including any actions to be taken (temporary, underway, and planned), and a communication plan going forward.

Contact information for program managers can be found on the WisDOT inspection website located here at this link Designated Inspection Program Managers.

Notifications should be documented in HSIS on the Critical Findings tab. The PM with jurisdiction over the bridge and others involved with the Wisconsin bridge inspection program are auto populated with email address that will be automatically sent when comments are added and the "send" button is clicked. Creating a notification in HSIS does not fulfill the primary communications requirement of the Critical Findings process – a telephone call is required.

1.7.3.2.2 FHWA Notification

National Highway System (NHS) – The Statewide Program Manager will notify the FHWA Division Bridge Engineer within 24 hours of discovery of any critical finding on the NHS identified as the following:

- Structural conditions or deficiencies that pose an immediate threat to public safety which results in the immediate partial or full closure of the structure.
- NSTM inspection condition to be rated in serious or worse condition (Item B.C.14 ≤ 3).

Monthly Status Report - FHWA Division Bridge Engineer will receive from WisDOT a monthly, or as requested, a written status report for each critical finding until resolved. A critical finding should be resolved within 30 days so it should not appear on more than 2 monthly reporting intervals. The report will contain:

1. Owner
2. NBI Structure Number
3. Date of Finding
4. Description and Photos of critical finding (referenced and available on HSIS under the inspection and critical finding activity)
5. Description of completed, temporary, and or planned corrective actions to address critical finding (referenced and available on HSIS under the inspection and critical finding activity)
6. Status of corrective action (Active/Completed)
7. Estimated date of completion if corrective actions are active
8. Date of completion if corrective actions are completed

1.7.3.3 Additional Actions

Additional action may need to be taken beyond addressing the safety concern. The PM and the owner must determine the next actions. This would include monitoring any temporary measures, updating/improving traffic control measures, completing permanent repairs, planning for rehabilitation or replacement, and other. Temporary repairs and traffic control must be monitored for worsening or changing condition and proper function. Temporary repair is not an acceptable substitute for permanent repairs. Actions take must be recorded in HSIS on the Critical Findings tab.

1.7.3.4 Analysis and Design

The determination of a critical finding may require structural calculations to determine load-carrying capacity or an engineered repair to mitigate the structural deficiency. Although the State Bridge Rating Engineer is part of the notification process, each County Inspection Program (for local structures) is responsible to independently perform any necessary load ratings or repair designs (stamped by a WI licensed PE). The State Bridge Rating Engineer is available for consultation and oversight to the counties. For critical findings on state

structures, Bureau of Structures will perform, or have performed, any necessary load ratings and repair designs.

The determination of a critical finding may require structural calculations be performed to determine load-carrying capacity or an engineered repair be designed to mitigate the structural deficiency. Although the State Bridge Rating Engineer is part of the notification process, each County Inspection Program (for local structures) is still responsible to independently perform any necessary load ratings or repair designs. Bureau of Structures will then serve as oversight to the counties. For critical findings on state structures, Bureau of Structures will perform, or have performed, any necessary load ratings and repair designs.

1.7.3.5 Close-out Inspection

A routine or special inspection must be completed to update the condition and inventory data after the final actions. Bridges that remain closed or restricted until a rehabilitation or replacement project is completed will also require a close-out inspection. Document and photograph the traffic control and bridge closure system installed. Permanent traffic control measures, including barriers when required, must be in place. Include assessment 9036 Bridge Closure System. The anticipated timeframe of the rehabilitation or replacement must be documented on the inspection report and the critical findings tab. The close-out inspection information can be documented in the same inspection report the CF was identified or in a separate special inspection report.

1.7.4 Critical Finding Documentation

The critical finding must be documented in the Highway Structures Information System (HSIS) using the Critical Finding Activity Type under the Inspection Tab as soon as practicable, but within 30 days of the finding. Form DT2026 was developed to assist the inspector in documenting the critical information and plan of action. This document can be used to help complete the Critical Findings tab in HSIS. The completed DT2026 can be uploaded, but is not required, into HSIS as supporting documentation. The information needed includes the following:

- Date of Critical Finding
- Description of the critical finding - include a detailed narrative of exact location, size, and severity of all structural deficiencies that warrant a critical findings designation. Provide additional information on how the critical finding was discovered and the reason the deficiency exists (mainly for bridge hits).
- Any actions undertaken to resolve the critical finding (temporary, underway, planned, immediate final, and long-term final
- Repair plans, design/load rating calculations.
- Action completion dates; estimated completion dates
- Notifications and contacts completed
- Photographs and/or sketches of the structural deficiencies
- Photographs of the installed traffic control/restrictions

Create an inspection (Special, Routine, NSTM, or UW-Dive) report in HSIS with a Critical Finding Activity within 30 days of the CF. Create only one Critical Finding Activity for the same

event regardless of any additional inspection reports. Upload photos and any inspection documentation in an open inspection. Continue to update the Critical Finding tab with all temporary, underway, planned, and final actions including the completion dates. The Critical Finding tab will remain open until a Final Action is completed and the PM reviews the CF. This inspection report that identified the CF can be completed in HSIS once all information about the inspection has been enter – the CF does not need to be finalized to complete the inspection. However, any photographs or documents related to the critical finding must be included with an inspection report in HSIS. The final immediate action must address the immediate safety concern, including installation of the proper traffic control and needed barrier.

Critical Finding Action Definitions

- Temporary – This action is selected to describe a temporary action taken to secure the scene. Examples include restricting traffic using temporarily traffic barrels, a temporary repair, or blocking a lane with a vehicle while trying to determine the next action.

- Underway – This action is selected to describe any actions that are in the process of being completed. An example would be repair work that has started and is in progress. This action would typically be selected when the repair will take multiple days, weeks, or longer to complete.

- Planned – This action is selected to describe any expected future actions that are not the Final-Immediate. Examples would be additional inspections, reduced inspection interval, or repairs being planned/designed. If bridge rehabilitation or replacement is the final planned action, the anticipated timeframe must be documented as a Final-Long Term Action.

- Final-Immediate – This action is selected when the action has resolved the safety concern, the traffic restriction or bridge closure is in place, the posting signage is installed, and/or the permanent repair has been completed. A close-out inspection is required to document a final repair.

- Final-Long Term – This action is selected when the final long-term action is a bridge rehabilitation or bridge replacement. A close-out inspection is not required with a final long-term action. An initial or special inspection will be completed once the rehabilitation or replacement is completed.

EXAMPLE CF: A hole is found in a bridge deck in the driving lane. The *temporary action* is to restrict traffic from driving in the area of the hole. An *underway action* would be to place a steel plate over the hole. A *planned action* would be to complete a concrete patch. The *Final-Immediate action* is to complete a permanent concrete patch and remove the traffic restriction. A *Final-Long Term* action may be to put this bridge in the program for rehabilitation. Upon completion of any action, enter a completion date in the Critical Findings tab next to the action completed. If the patch is a permanent repair and no additional actions are needed, a close-out inspection is required.

CF Action Documentation Review – Upon the Final-Immediate action being completed and documented, the team leader (TL) will notify the inspection program manager (PM) with jurisdiction over the structure. The PM will review the CF actions documented in HSIS. If the actions are acceptable and complete, the PM must update the complete date on the CF tab in HSIS. . A Final – Long Term action can be added by the PM.

1.7.5 Traffic Control

The critical finding may require traffic on the bridge to be restricted. The traffic control and signage for full closures, partial lane closures, or shoulder closures shall be in compliance with the Manual on Uniform Traffic Control Devices (MUTCD) and the Wisconsin MUTCD. These manuals combine to provide guidance on the installation and proper use of traffic control devices. Local municipalities should contact the county highway department for assistance if acceptable barriers or signs are not immediately available. Photographs of the traffic control for the restriction/closure must be included in the CF documentation and loaded into HSIS.

Properly installed traffic control improves the safety of the traveling public and reduces the liability of the bridge owner. Example traffic control layouts for different roadways and situations are shown in the WisDOT Wisconsin Work Zone Field Manual. The layouts shown may need to be modified depending on field conditions and available traffic control devices. Traffic channelizing devices may need to be installed the length of the span for proper delineation. Traffic barriers and channelizing devices that can be easily moved or driven around may result in a non-compliance determination by FHWA's National Bridge Inspection Program, as well as a risk to the traveling public.

Flow Chart of the Critical Finding Process

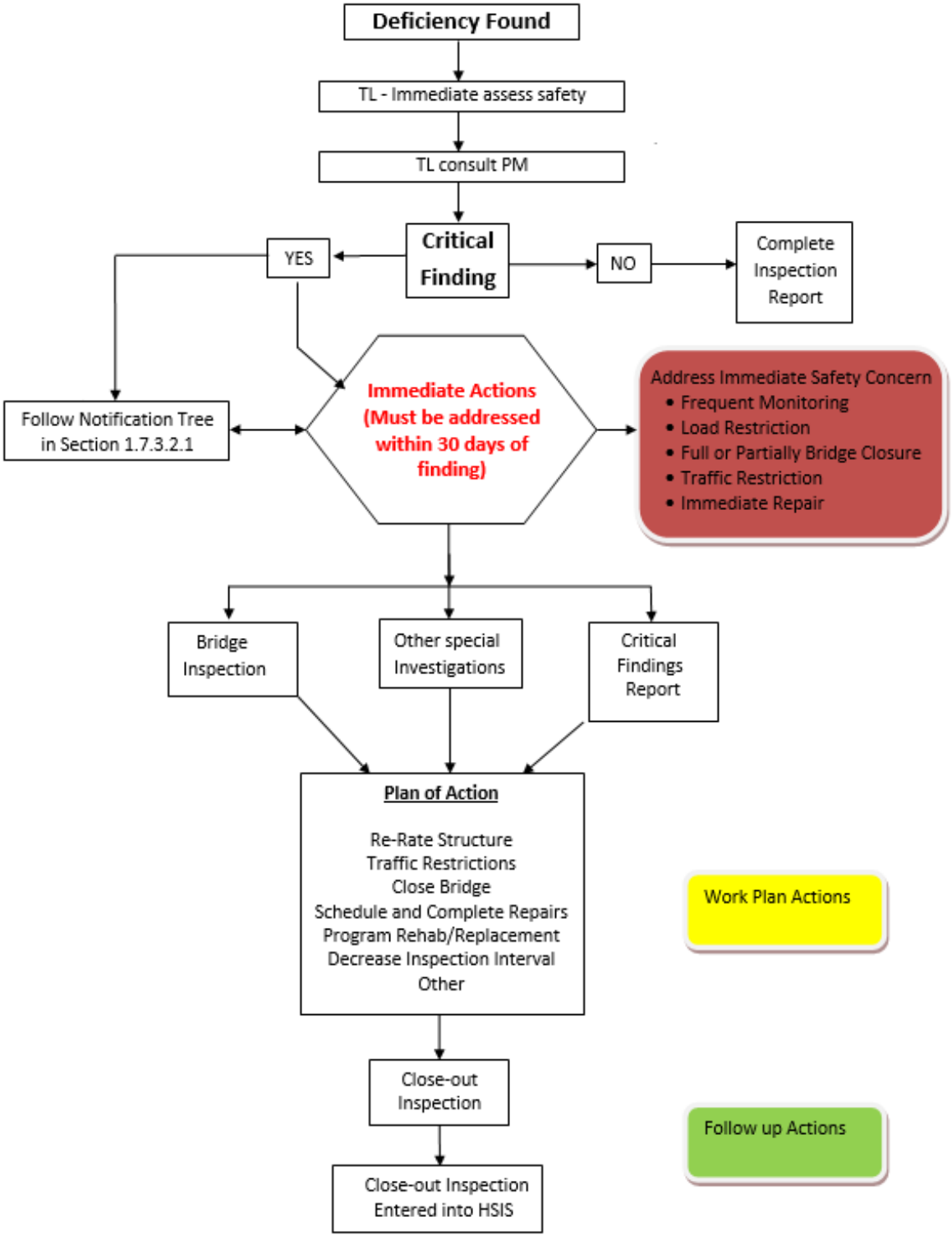


Figure 1.7.1-1: Emergency Notifications and Follow-up Documentation Flow Chart.

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Appendix A: Deck Scanning Policy 2

APPENDIX A: DECK SCANNING POLICY

This deck scanning policy applies to state-owned bridges, offering guidance on the use of non-destructive testing (NDT), semi-destructive testing (SDT), and non-destructive evaluation (NDE). Any means or methods not described in this policy are to be performed at the discretion/approval of the Bureau of Structures (BOS). This policy is directly related to the scope of work covered under the deck survey master contract and also includes sounding/chaining of bridge decks performed through routine inspections. This policy is necessary to determine accurate scope of deliverable projects, certify structure work concepts for various funding programs, and refine deterioration models used in the Wisconsin Structures Asset Management System (WiSAMS). The following bullet points are applicable to all NDT, SDT, and NDE methods described in this policy:

- All deck scanning methods are performed on 100% of the wearing surface(s), or scanned using sensors at appropriate spacing to deliver accurate results on 100% of the wearing surface(s).

- All results (required values and final reports) are to be entered/uploaded into the Highway Structures Information System (HSIS) using the DECK EVALUATION activity type. All final reports, in addition to meeting all contractual requirements, will also include a description of the NDT/SDT systems utilized, NDE methods for determining results, and any factors that may affect the quality of results.

- Condition-based criteria references the most recent Structures Inspection Field Manual using element/defect number, condition state (CS) number, and percent (%) of total element quantity. Example notation of condition criteria that would trigger an early deck scan on a bare wearing surface is 3210(CS2+CS3+CS4) > 1%. This means that the summation of quantities in CS2, CS3, and CS4 of defect 3210 (Debonding/Delaminations/Spalls/Patched Area/Pothole) is greater than 1% of the total quantity of the bare wearing surface.

Thermography (Infrared, IR)

Deliverables

- Estimated quantities and percentage of total deck (i.e. not a range such as 5-10%, but an actual value such as 6%). Very small values may utilize tenths of percent, but this level of accuracy is not required on most values.

- Date and time of scan

- Weather type during scan and temperature (^{o}F)

- Level selection: Level 0 (aerial) or Level 1 is the default level. Level 2 is performed within 2 years of any deck-related project work. The purpose of the level 2 scan is to estimate deck repair quantities and locations for plan development. Level 2 may also be performed if BOS requires additional data for programming decisions. Level 3 is not

recommended as this scope of work should be determined by state personnel, but may be utilized in certain situations with BOS approval.

- Level 0 (aerial)
  - Quantities and percentage of total deck for various defects, including: Delamination, Debonding (if technology allows), Concrete Patching, and Asphalt Patching.
  - HD aerial photo and corresponding thermography photo (may be stitched from multiple photos)
- Level 1
  - Quantities and percentage of total deck for various defects, including: Delamination, Debonding, Concrete Patching, Asphalt Patching, and Spalling.
- Level 2
  - All deliverables of Level 1
  - Plan view of defect locations
- Level 3
  - All deliverables of level 2
  - Plan view of deck prep locations
  - Quantities and percentages of deck prep

<u>When to perform IR on a bare deck</u>

- Regions may elect to perform chain dragging NDT in lieu of deck scanning. The chain dragging will need to be performed on the entire wearing surface. Chain dragging may be performed on a more frequent basis than the IR frequency, but not less frequent than the IR frequency.
- First IR when deck is 15 years old or when one of the following defect thresholds are recorded on a bridge inspection: 3210(CS2+CS3+CS4) > 1%, 3220(CS3+CS4) > 1%, or 3220(CS2) > 25%.
- Perform an IR every 5 years after initial IR. There is not an upper limit on condition for stopping IRs (do not stop when delamination reaches 25%).
- Frequency or level of IR may be modified based on historical data and projected future work; Level 0 or Level 1 is the default level.

- When a different wearing surface exists, refer to the appropriate IR schedule using the headings below.

When to perform IR on a deck with a thin polymer overlay

- First IR when overlay is 5 years old.
- Perform an IR every 5 years after initial IR. There is not an upper limit on condition for stopping IRs.
- Frequency or level of IR may be modified based on historical data and projected future work; Level 0 or Level 1 is the default level.

When to perform IR on a deck with a concrete or PPC overlay

- Regions may elect to perform chain dragging NDT in lieu of deck scanning. The chain dragging will need to be performed on the entire wearing surface and at the same frequency IR would be required.
- First IR when overlay is 10 years old or when one of the following defect thresholds are recorded on a bridge inspection: 3210(CS2+CS3+CS4) > 1%, 3220(CS3+CS4) > 1%, or 3220(CS2) > 25% with overlay more than 5 years old.
- Perform an IR every 5 years after initial IR. There is not an upper limit on condition for stopping IRs.
- Frequency or level of IR may be modified based on historical data and projected future work; Level 0 or Level 1 is the default level.

When to perform IR on a deck with an asphalt overlay

- First IR when overlay is 5 years old.
- Perform an IR every 3 years after initial IR. There is not an upper limit on condition for stopping IRs.
- Frequency or level of IR may be modified based on historical data and projected future work; Level 0 or Level 1 is the default level.

When to perform IR on the underside/soffit of a deck

- No requirements defining use at this time.
- Under deck thermography is performed on a trial basis and will be utilized at BOS discretion.
- Under deck thermography results have not been validated, but may help estimate the amount of full depth repair required. This is a level 2 scan.

Chloride Ion Testing

Deliverables (minimum)

- Core location
- Testing depth (must include testing at rebar depth)
- Water soluble chloride by weight of concrete (lb/cy) using either ASTM C1216 or AASHTO T260

When to test a bare deck

- If deck is older than 10 years, BOS may require testing prior to programming a Thin Polymer Overlay application.
- If deck is older than 15 years, BOS may require testing prior to programming a PPC Overlay application.

When to test a deck with a concrete or asphalt overlay

- No testing required.

When to test a deck with a thin polymer or PPC overlay

- Test prior to reapplication of Thin Polymer or PPC Overlay.

Ground Penetrating Radar (GPR)

Deliverables (minimum)

- Date and time of scan
- Weather type at time of scan
- Date of last rain event prior to scan
- Average deck reading (change in amplitude) at top mat of reinforcement (decibels)
- Maximum deck reading (change in amplitude) at top mat of reinforcement (decibels)
- Threshold used to determine deteriorated/contaminated concrete (decibels)
- Quantities and percentage of deck at or above the threshold value
- Plan view of GPR results showing range of deteriorated/contaminated concrete values
- Average overlay thickness (if non-PCC overlay exists)
- Average depth of rebar (top mat)

When to perform GPR on a bridge deck

- No requirements defining use at this time.
- GPR is performed on a trial basis and will be utilized at BOS discretion.
- GPR may be performed within 2 years of any deck-related project work and corresponding Level 2 IR.
- GPR may also be performed earlier in the life cycle to measure predictive abilities of the results.

Impact Echo (IE)

Deliverables (minimum)

- Date and time of scan
- Scan interval spacing in both directions (ft)
- Concrete velocity used in analysis or method that was used to obtain velocity
- Average deck reading (kHz)
- Threshold reading used to determine each defect identified in the scan, such as: shallow delamination, deep delamination, cracks or other deterioration (kHz)
- Quantities and percentage of deck delamination, cracks and other deterioration.
- Plan view of IE results

When to perform IE on a bridge deck

- No requirements defining use at this time.
- Recommend limiting use to cementitious wearing surfaces only.
- IE is performed on a trial basis and will be utilized at BOS discretion.
- IE may be performed within 2 years of any deck-related project work and corresponding Level 2 IR.
- IE may also be performed earlier in the life cycle to measure predictive abilities of the results.
- Limited area IE may be used to validate other deck scanning methods at the discretion of the contractor.

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Appendix C: SRM Baseline condition report for steel twin tub girders 2

APPENDIX C: SRM BASELINE CONDITION REPORT FOR STEEL TWIN TUB GIRDERS

BRIDGE INSPECTION REPORT
SRM Baseline Condition Report
for Steel Twin Tub Girders

Structure No:	
Team Leader:	
WI Inspector No:	
Date:	

Inspection team leader to complete this form upon review for conditions that might affect the applicability of the method to establish redundancy per SIM Section 1.3.5.7. Include this completed form with the inspection report in HSIS. See specific inspection procedures in the inspection report for location of members critical to the alternate load path.

Yes	No	Structural review required if any of the following applies to SRM	Comments
☐	☐	Fatigue crack that has not been effectively arrested.	
☐	☐	A primary load carrying member in CS4.	
☐	☐	Unresolved impact damage to primary load carrying members, such as stress risers (cracks, sharp reentrant corners) or large out-of-plane distortion.	
☐	☐	CS3 or CS4 distortion in the bottom flange or web of the tub girders in the negative moment region.	
☐	☐	CS3 or CS4 distortion in the bottom flange or web of the exterior diaphragms.	
☐	☐	Missing bolts in field splices or external diaphragm to girder connections.	
☐	☐	Cracks, distortion, or CS4 section loss in the external diaphragms and/or missing bolts in diaphragm-to-girder connections.	
☐	☐	Exposure to intense, prolonged, or high temperature fire.	
☐	☐	The position or number of striped lanes has changed.	
☐	☐	Change in deck overburden or other increase in dead load to the superstructure.	
☐	☐	Removal of external diaphragms (not allowed).	

Additional Notes:

_____
_____
_____
_____
_____

If checking Yes above, notify the inspection program manager and in HSIS, select the Action box on the Element tab for the SRM and select Structural Review activity type as part of the inspection report.

Inspection program manager _____ Date Notified _____

Link to SRM Baseline Condition Report for Steel Twin Tub Girders:

SRM Baseline Condition Report for Steel Twin Tub Girders

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Appendix D: SRM Baseline condition report for Floorbeams (spaced greater than 14')..... 2

APPENDIX D: SRM BASELINE CONDITION REPORT FOR FLOORBEAMS (SPACED GREATER THAN 14')

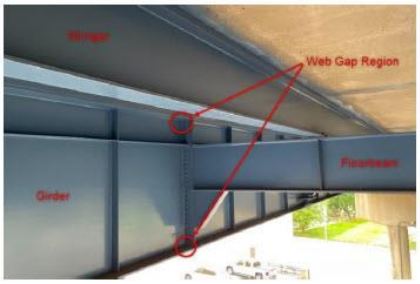
BRIDGE INSPECTION REPORT
SRM Baseline Condition Report
for FLOORBEAMS (spaced greater than 14')

Structure No:	
Team Leader:	
WI Inspector No:	
Date:	

Inspection team leader to complete this form upon review for conditions that might affect the applicability of the method to establish redundancy per SIM Section 1.3.5.7. Include this completed form with the inspection report in HSIS. See specific inspection procedures in the inspection report for location of members critical to the alternate load path.

Yes	No	Structural Review required if any of the following applies to SRM	Comments
Complete the section below for In-Depth inspections requiring hands-on access.			
☐	☐	Out-of-plane distortion induced crack in the web gap region exists but has not been effectively mitigated.	
	☐ N/A		
☐	☐	Fatigue cracks in the floorbeam, floorbeam connection to girder, or stringer have been identified but not effectively mitigated.	
	☐ N/A		
Complete the section below for all inspections involving floorbeams and stringers			
☐	☐	Floorbeam in CS4.	
☐	☐	Stringer(s) within adjacent longitudinal bays in CS4.	
☐	☐	Deck within adjacent longitudinal bays in CS4.	
☐	☐	Stringer or floorbeam distortion is CS3 or CS4.	
☐	☐	Missing bolts/rivets in stringer or floorbeam field splices.	
☐	☐	The position or number of striped lanes has changed.	
☐	☐	Change in deck overburden or other increase in dead load to the superstructure.	

Selection of "Yes" to any item above applies to the floorbeam in question (or the nearest floorbeam when selected "Yes" for stringer) as well as the immediately longitudinally adjacent floorbeams.



If checking Yes above, notify the inspection program manager and in HSIS, select the Action box on the Element tab for the SRM and select Structural Review activity type as part of the inspection report.

Inspection program manager _____ Date Notified _____

Link to SRM Baseline Condition Report for Floorbeams (spaced Greater Than 14'):

SRM Baseline Condition Report for Floorbeams (spaced Greater Than 14')

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2.1 GENERAL

This chapter describes the mechanics, components, construction materials, and classifications of bridges. This part of the Manual addresses subjects common to fixed and movable (Complex) bridges.

The chapters are broken down into basic bridge components (i.e., Deck, Superstructure, Substructure, etc.). Within each chapter the bridge components are broken down into construction material (i.e., Concrete, Steel, Timber, Masonry, etc.). Under each material section are listed the appropriate elements. The elements are described and element level inspection procedures are discussed for the inspector's use. Each element has an associated list of material defects which determine the Condition States of a portion or the entire element depending on the unit of measurement for that element. Refer to Appendix A at the end of Part 2 for full descriptions of the element defects and the appropriate Condition States for each defect.

Refer to Part 3 of this Manual for a complete treatment of components unique to movable (Complex) bridges.

2.2.1 Introduction

Fixed bridges are by far the most common structures which carry the traveling public (both vehicular and pedestrian) over roadways, railways, waterways, and valleys. Movable bridges are common over navigable waterways where the height of a fixed bridge would otherwise restrict marine traffic. It is the responsibility of the agencies that own these structures to uphold the public's confidence in the infrastructure by knowing the condition of their structures and by maintaining them in a safe and cost-effective manner.

According to Federal Highway Administration Policy, any highway structure over 20 feet in length is defined as a "bridge". However, Trans 212.02(2) of the Wisconsin Administrative Code more clearly defines a bridge as:

> [A] structure, including supports, erected over a depression or an obstruction, such as water, a highway, or a railway, having a track or passageway for carrying traffic or other moving loads, and having an opening measured along the centerline of the roadway of more than 20 feet between the under coping of abutments or spring lines of arches, or extreme ends of the openings for multiple boxes. It may include multiple pipes where the clear distance between openings is less than half of the smaller contiguous opening.

An abutment under coping is its front face. Along public roads, structures 20 feet or less in length are considered to be Ancillary structures and are covered in Part 4 of this manual. Other highway structures are also defined in other parts of this Manual.

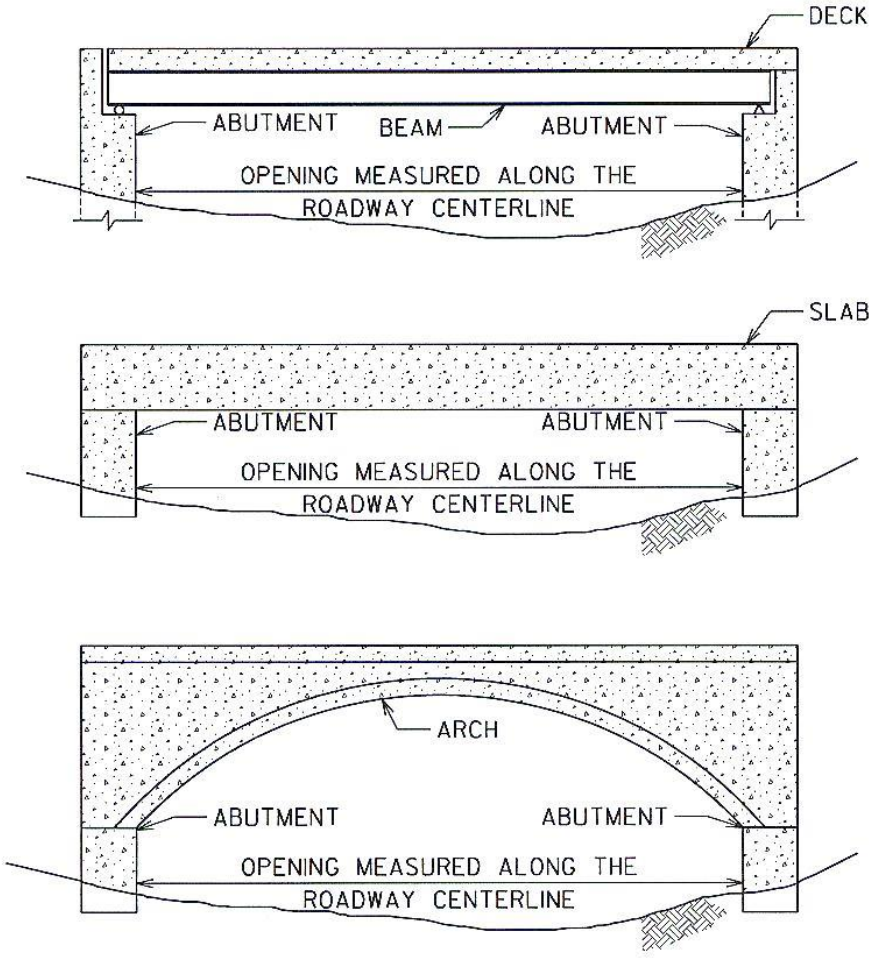

Figure 2.1.1-1: Opening Measurement Used to Define a Bridge.

In Wisconsin (2017), there are about 5300 structures classified as bridges maintained by the State and about 8,900 maintained by local government agencies. The ages, materials, span lengths, design details, etc. result in a wide variety of bridge styles in public use. Regardless of their status, the bridges continue to endure daily traffic fatigue and the deteriorating effects of deicing salts and weather. Since design codes dictate that bridges conservatively incorporate safety factors in their design, bridges are able to tolerate some amount of overload and deterioration. At some point the wear becomes excessive, and the bridge's ability to safely carry traffic loads may need to be evaluated.

It is critical to know when a structure or component thereof has deteriorated to such an extent that it is unable to support the loads it was designed to carry. One of the responsibilities of a bridge inspector is to recognize this condition. To make this judgment, as well as to write a meaningful inspection report, an inspector must be knowledgeable about structural mechanics, be able to identify the components of a bridge and know their functions, and understand the behavior of different materials. Understanding the critical areas of components along with the common deficiencies found in each material is paramount in inspecting structures.

2.2.2 Overview of Bridge Mechanics

A bridge inspector must know how a bridge functions to recognize and judge how a defect affects the load-carrying ability of a member and eventually the entire bridge. This section briefly describes bridge loads and bridge geometric classifications with the associated behavior of each. A more complete discussion may be found in Chapter 5 of the *Bridge Inspector's Reference Manual*.

2.2.2.1 Design Loads

Bridges are designed to withstand a variety of loads. These loads can be divided into three general categories: dead, primary transient (live) and secondary transient (live).

Dead loads are the permanent self-weight loads of the bridge. Dead loads include the weight of the girders, deck, railings, diaphragms, overlays, etc. Utilities mounted on a bridge are also considered dead loads.

Primary transient (live loads) are the temporary gravity loads that act on the structure. These include all moving vehicles (trucks, cars, trains), with their associated impact loads, and pedestrians. Current American Association of State Transportation and Highway Officials (AASHTO) specifications for bridges establish the design live loads for highway bridges. Depending on a bridge's use, different design trucks are specified. Major highways maybe designed for an alternate vehicle configuration as well. It should be noted that the design vehicles do not represent actual trucks found on highways, but were developed to represent an approximate live load for consistent design. The maximum pedestrian load on sidewalks or pedestrian bridges is 85 pounds per square foot.

Dead and primary live loads are gravity loads and exert downward forces on the bridge. Secondary loads are all remaining loads which act on the bridge, many of which act in a lateral direction. These loads include:

1. **Earth pressure** – Lateral soil pressure on abutments and retaining walls.
2. **Buoyancy** – Upward forces on substructures when submerged in water.
3. **Wind load on the structure** – Pressures due to wind blowing on the sides of girders, parapets, arches, etc.
4. **Wind load on the primary live loads** – Lateral pressures transferred to the bridge due to wind blowing on the sides of vehicles traveling across the bridge.
5. **Longitudinal forces** – Forces parallel to the bridge span due to vehicles braking and accelerating.
6. **Centrifugal forces** – Forces transverse to curved bridges due to vehicles traveling around a curve.
7. **Rib shortening** – Forces in arches and frames caused by dead load deformations.
8. **Shrinkage** – Forces within concrete members caused by member dimensional changes due to curing.

9. **Temperature** – Forces within members caused by member dimensional changes due to temperature fluctuations.

10. **Earthquake** – Lateral and vertical forces caused by seismic events.

11. **Stream flow pressures** – Lateral forces caused by river or stream flow acting on bridge members.

12. **Ice pressure** – Lateral forces caused by river or stream ice flow hitting bridge components.

13. **Curb loads** – Lateral forces due to vehicle wheels.

14. **Rail loads** – Lateral forces due to errant vehicles.

2.2.2.2 Simply Supported Spans

A simply supported span is the most basic type of bridge and the easiest for an engineer to analyze and design. It forms a stand-alone, single span, with its beams or trusses commonly supported with one fixed end and one movable end. The movable end bearing devices allow the bridge to rotate and expand under load and temperature changes. On some bridges, the beam ends are encased within a concrete diaphragm over the abutments, hiding the bearing devices. Simply supported bridges can be very forgiving structures should one of the supports settle, since the bearings act like hinges to allow for unrestrained, free movement. Simply supported bridges can also be very unforgiving structures should the beams ever fail between the supports. A failed beam section may act as a third hinge, allowing the beam to collapse.

Figure 2.1.2.2-1: Simply Supported Span – Prestressed Concrete Girders.

Most commonly, simply supported bridges are single span structures with the beam ends bearing on abutments. Alternatively, a bridge may be built up of a series of simply supported spans, with the beam ends supported on either abutments or piers. Each pier will therefore support two lines of bearings. To allow for unrestrained rotation of the bridge ends, joints are normally provided in the deck above all abutments and piers with simply supported spans.

When a simply supported span is loaded, it deflects downward between its supports and rotates at its bearings. For the most basic case of a uniformly loaded beam, shear forces (vertical forces within the beam) are highest at the supports, and zero at midspan. Point loads produce uniform shears of varying magnitudes along the beam. For any type of loading, the amount of bending is zero at the supports. Bending is a maximum at midspan (for uniform loading) or directly under the load (for point loading).

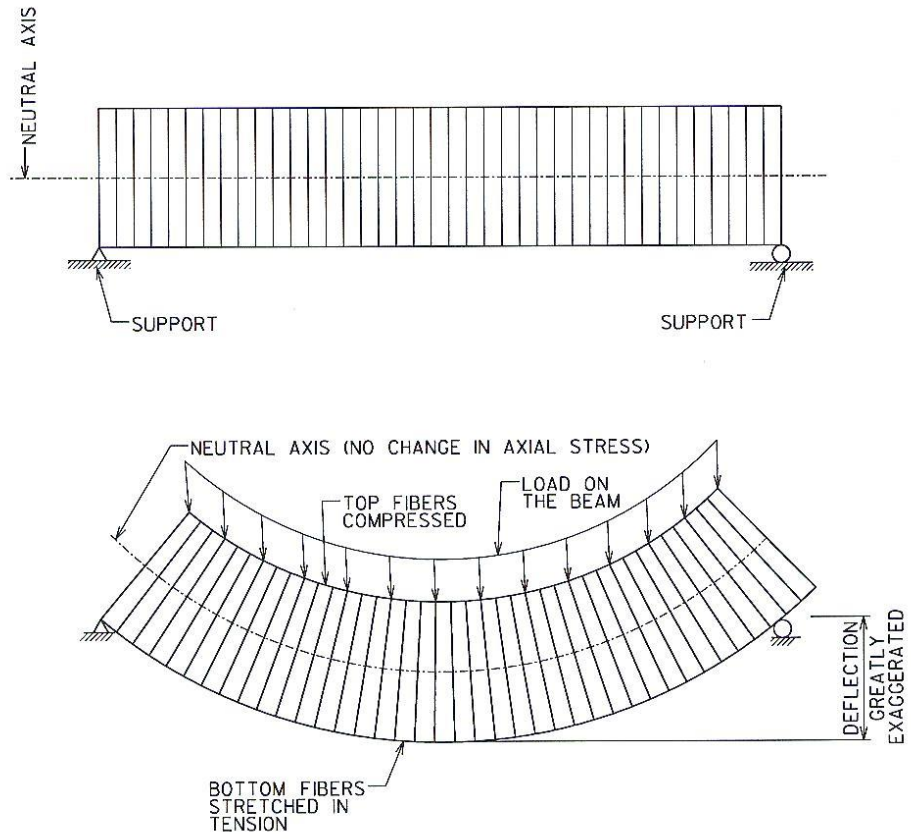

Figure 2.1.2.2-2: Simply Supported Beam Unloaded (Top), and Loaded (Bottom).

An important mechanical concept to understand as it relates to bridges is beam bending. Bending is measured in units of length multiplied by force (defined as a moment). Most commonly in the U.S., moment is currently measured in foot-kips or foot-pounds. For example, suppose a person wanted to hold out horizontally a 5-pound hammer with a 1-foot-long handle in one hand. The person's wrist would need to resist a 5-pound X 1-foot = 5 foot-pound bending moment. If the hammer weighed 10 pounds, a 10-pound X 1-foot = 10 foot-pound bending moment would need to be resisted. Similarly, a 5-pound-hammer with a 2-foot-long handle would produce a 5-pound X 2-foot = 10 foot-pound bending moment.

As it relates to a beam, bending is classified as either a positive or negative. A deflected beam having a concave curve (producing a "smiling face") will be resisting positive bending moments. A deflected beam having a convex curve (producing a "frowning face") will be resisting negative bending moments. When a beam is resisting positive bending moments,

the fibers at the top surface of the beam are shortened and experience compressive stresses, while the fibers at the bottom surface are stretched and experience tensile stresses. Conversely, when a beam is resisting negative bending moments, the fibers at the bottom surface of the beam are experiencing compressive stresses, while the fibers at the top surface are experiencing tensile stresses. In both situations, there is a point somewhere between the beam's top and bottom that does not change length. Since there is no length change, there are no stresses generated. This point on the beam's cross-section is known as the neutral axis. Simply supported spans will always deflect in a concave shaped curve and will therefore only experience positive bending moments. As a result, the top fibers, or flanges, of the beams will always be in compression, and the bottom fibers, or bottom flanges, will always be in tension.

2.2.2.3 Continuous Spans

Continuous spans are more complex in their behavior, design, and analysis than simply supported spans. Continuous spans are essentially beams or trusses with supports at their ends, but also with one or more intermediate supports. In other words, the beams are continuous over the supports. As with simply supported spans, the supports are most commonly pins, rockers, rollers, or bearing pads which allow the bridge to rock, rotate, and expand under load. Continuous bridges can be very forgiving structures should the beams ever fail between the supports. To illustrate this, if the middle span of a three-span bridge fails and creates a hinge, the end spans can act as anchor spans. In doing so, the end spans can prevent a collapse by acting as levers with pivots at the piers to hold up the failed middle span. Continuous bridges can also be unforgiving structures should one of the supports settle. Since continuous spans have no internal hinges, support settlements act like additional loads to the bridge. Depending on which support settles, the settlement load may overstress and possibly fail the beams.

One advantage of using continuous spans versus multiple simple spans is that the beam depths will be smaller, deflections will be less, and spans can be made longer. Also, each pier under a continuous span will have only one line of bearings, resulting in cost savings in multi-span structures. Because the beams are continuous over the intermediate supports, deck joints are not required – yet another economic advantage. This also provides a serviceability advantage in the form of a smoother riding surface for the traveling public. Disadvantages of continuous bridges include the transverse deck cracks that develop over the piers and the increased costs of design, fabrication, and erection.

Figure 2.1.2.3-1: Continuous Spans – Steel Girders.

When a continuous span bridge is uniformly loaded, it deflects downward between the supports and rotates at its supports. Shear forces are at a maximum at the supports and zero at or near midspan. Positive bending moments are at a maximum between supports, while negative bending moments are at a maximum directly over the interior supports. The deflected shape of the bridge will therefore have a concave curve between supports and convex curvature over the supports. Point loads produce more complex patterns of shears and bending moments. Briefly, however, point loads will deflect the beam downward in the loaded span; produce uniform shears of varying magnitudes, and produce maximum bending moments at the point of the load. For any loading type, the bending moment is zero at locations where the curvature changes from concave to convex and also at the end supports.

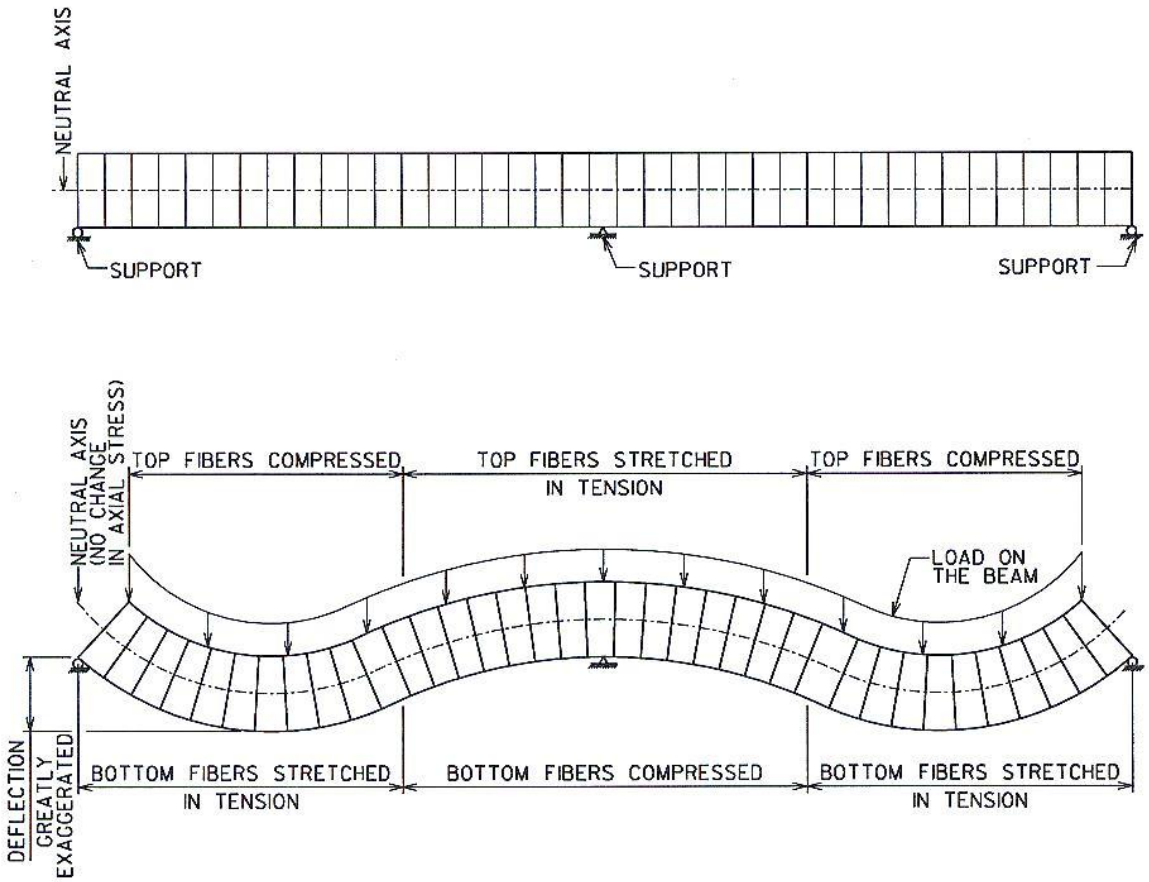

Figure 2.1.2.3-2: Two-span Continuous Beam Unloaded (Top), and Loaded (Bottom).

2.2.2.4 Cantilever Spans

A cantilever span has one end that is free to deflect and rotate, and one end that is fixed against deflection. The fixed end is ideally fixed against rotation, but in reality a small amount of tilt will occur. The deflected shape of a cantilever span will always be convex. Therefore, bending moments are always negative and vary from zero at the free end to a maximum at the fixed end. At the free end, shear forces may be zero, but are usually relatively high since tips of the cantilevers are commonly used to support the end of another span. Shear forces are a maximum at the fixed end. The free ends of cantilever spans are always locations for expansion joints.

Figure 2.1.2.4-1: Steel Cantilever Span.

Cantilever spans will rarely form an entire bridge but are typically portions of a structure. Usually, fixed cantilevers are simply extensions of continuous spans, with the continuous spans providing anchorage for the cantilever. Long span truss bridges frequently employ cantilevers to form part of the main crossing. Cantilever spans are built on either side of the obstruction, extending out towards midspan. The tips of the two cantilevers sometimes meet in the middle, but more commonly an independent simply supported structure is suspended between the tips of each cantilever. On some large girder bridges, short cantilever spans may be used to join a series of independent continuous bridges to form a single, long structure. The cantilever span free end is sometimes called a ship lap joint, since the supported span of one structure laps over the cantilever supporting span.

Advantages of using cantilever spans include a positive bending moment reduction in the adjacent spans and eliminating the use of expensive shoring for main span construction of truss cantilevers.

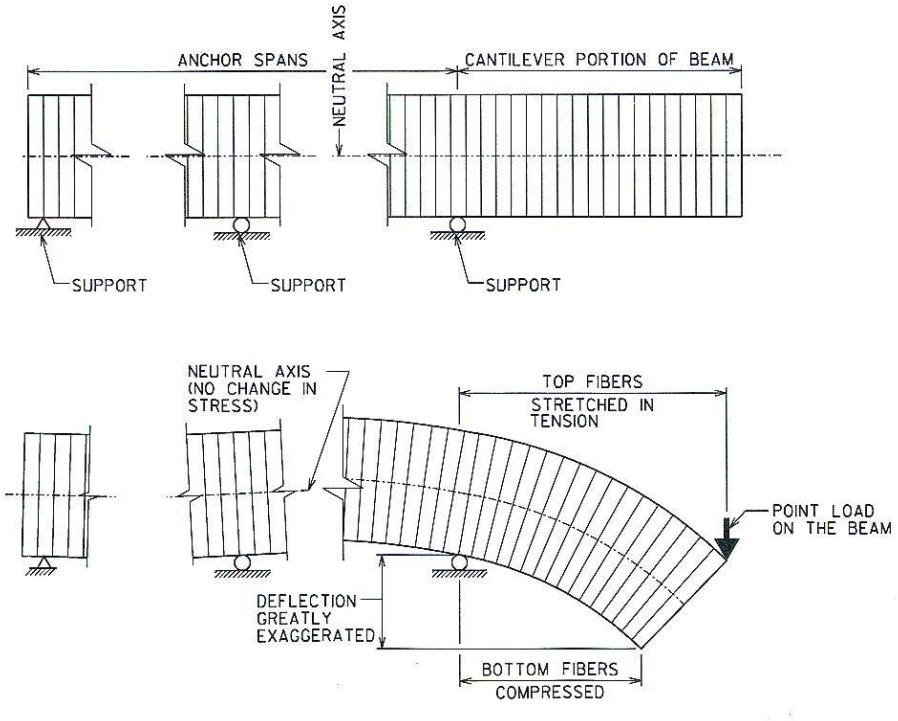

Figure 2.1.2.4-2: Cantilever Beam Unloaded (Top), and Loaded (Bottom).

2.2.2.5 Span Definition Overview

The following figure will help to summarize the definitions of simple, continuous, and cantilever spans.

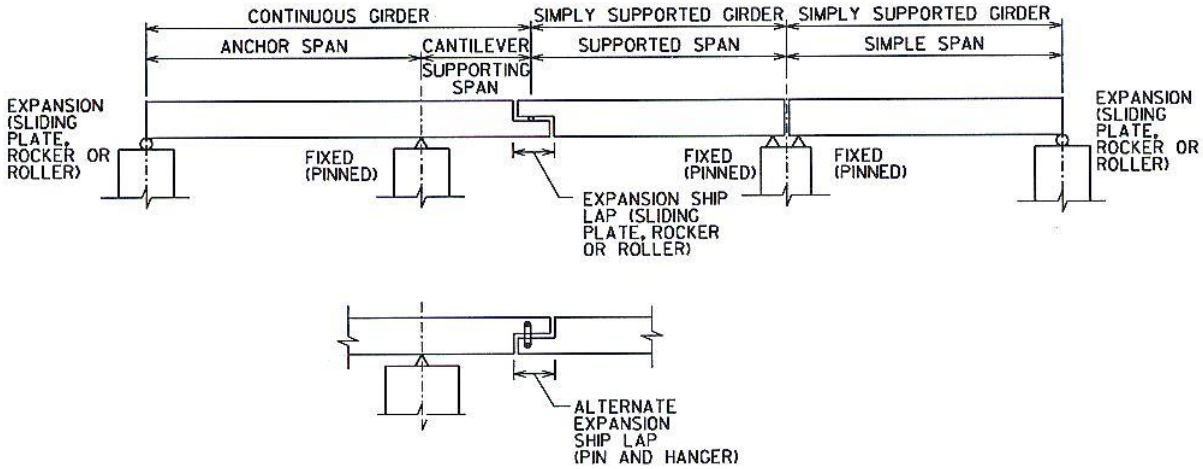

Figure 2.1.2.5-1: Span Definition Overview.

2.2.3 Fixed Bridge Components

There are three basic components common to most fixed bridges. These components are the deck, superstructure, and substructure.

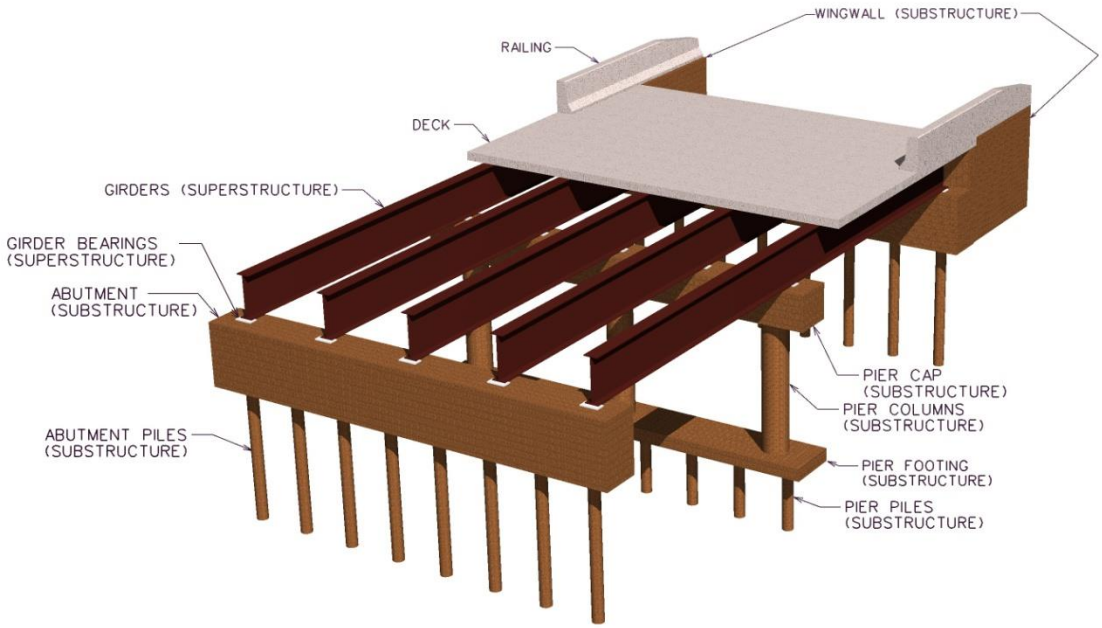

Figure 2.1.3-1: Bridge Components.

2.2.3.1 Deck

A deck is the bridge component that directly receives the vehicular wheel loads in the case of a roadway, or foot traffic loads in the case of a pedestrian bridge. A deck's purpose on all bridges is to provide a smooth driving or walking surface. On most bridges, it functions to transfer the live loads, superimposed dead loads (overlays, sidewalks, parapets, etc.), and its own weight laterally to the beams/girders/stringers/floorbeams. Sometimes, the deck acts compositely to become part of the beam's top compression flange. On concrete slab bridges, the deck itself is the main load-carrying member, delivering all live and dead loads directly to the substructure units.

2.2.3.2 Superstructure

The superstructure supports the deck and all of the live and dead loads applied to it, delivering these loads to the substructure units. There are three main types of bridge superstructures: beam bridges (slabs, beams, girders, and trusses), arch bridges, and cable-supported bridges (cable-stayed, suspension). The differences between the types are in how each superstructure delivers loads to the bridge supports. Beam type superstructures act primarily in bending, arches in compression, and cable-supported in tension.

2.2.3.3 Substructure

Substructure units of a bridge support the superstructure and deliver all of the bridge live and dead loads to the foundation soil or rock. Substructure units include the abutments, wingwalls, piers/bents, and, in the case of suspension bridges, the cable anchorages. Abutments provide superstructure support at the ends of the bridge, while piers and bents provide intermediate support. In addition to providing support for vertical loads, other loads must be resisted by the substructure, such as horizontal loads due to lateral pressures (soil, wind, current, and impact) and temperature expansion effects. Substructure components must therefore function as both compression and bending elements.

2.2.4 Materials

Several materials can be used to construct the various components of a bridge. Each has its own advantages and disadvantages with respect to material properties, weight, durability, cost, and appearance. It is of primary importance for a bridge inspector to understand the material properties, to understand the durability of each material, to recognize the seriousness of the material's deterioration and its causes, and to know the best remedial actions for each material.

2.2.4.1 Concrete

Concrete is a building material used since the ancient Roman Empire. It is a mixture of Portland cement, water, and aggregate (sand and stone). When cement is mixed with water, a chemical reaction takes place that produces a strong, durable, construction bonding material. Aggregates, which typically comprise approximately 75 percent of a concrete mix by volume, are used as an inexpensive filler material. In addition, aggregate improves a concrete's abrasion and weather resistance. Entrained air increases workability while the concrete is being placed and improves its durability against freeze/thaw once the concrete has cured.

Physical Properties

Plain concrete weighs about 145 pounds per cubic foot, and concrete reinforced with steel bars weighs approximately 150 pounds per cubic foot. Entrained air within the cement paste allows the absorption and passage of water, making concrete a porous material. It expands and contracts with increasing and decreasing temperatures, respectively.

Mechanical Properties

Concrete is a material used for its compressive strength, which generally ranges from 2,500 pounds per square inch to 8,000 pounds per square inch. Its tensile and shear strengths are poor, being only about 10 percent and 12 percent, respectively, of its compressive strength. Since it is a brittle material, a beam made out of plain concrete will snap like a piece of chalk under relatively small loads.

To make concrete ductile and usable, reinforcing steel bars (often referred to as rebar) are cast within the concrete mass to create a heterogeneous material (one with a varying composition within its volume) known as reinforced concrete. The reinforcing steel is able to resist the tensile forces that the concrete is unable to withstand. The concrete and reinforcing

steel also work together in carrying a member's shear forces. Reinforcing steel bars that transfer shear forces are placed as vertical stirrups in beams and as horizontal ties in columns. Reinforcing steel resists the tensile forces generated as a member undergoes bending. As an example, a simply supported reinforced concrete beam experiences only positive bending moments, generating compressive stresses near the top of the beam and tensile stresses near the bottom. As the beam is ultimately loaded, the concrete's tensile capacity near the bottom surface will be exceeded, and the concrete will crack. A simply supported reinforced concrete beam will therefore have its reinforcing steel placed near the beam's bottom surface to resist the tensile load. Concrete near the top surface resists the compressive loads.

Reinforcing steel is "deformed", or has transverse ribs, to provide a mechanical interlock with the concrete. In prestressed or post-tensioned concrete, high strength steel strands or bars replace deformed reinforcing steel. For prestressed concrete beams, the strands are pulled into tension, concrete cast around them, and the strands are released (cut) after the concrete is cured. The released strand's tensile force introduces compressive stresses into the concrete. For post-tensioned concrete beams (usually made in the field), concrete is cast around ducts placed near the beam's tension surfaces. After curing, post-tensioning strands or rods are placed into the ducts, anchored at one end, and stretched into tension by jacking at the other end of the beam. Locking or anchoring the jacked ends keeps the strands/bars in tension, introducing compression into the concrete. Prestressing/post-tensioning minimizes or eliminates net tensile stresses in the concrete. With the concrete and steel working together, a strong, ductile, and durable construction material is created.

Concrete Deterioration

Concrete can experience many types of defects, and the causes of each can vary. It is important for the inspector to understand these causes so that a proper evaluation and recommendation can be made.

Cracks

Cracks are linear fractures in the concrete that may extend partially or completely through a member. Due to the nature of reinforced concrete, it is not possible to prevent crack development. Prestressed concrete, on the other hand, should not develop any cracks (other than shrinkage cracks) under normal use. Cracks are described by their length, width, type, and orientation.

Crack Widths

The *Manual for Bridge Element Inspection* classifies cracks as insignificant, moderate, or wide. As shown in Field Manual under the material tabs for reinforced concrete and prestressed concrete. Insignificant cracks are visible, but cannot be measured with a ruler. By definition, insignificant cracks are less than 0.012 inches wide. Structurally, insignificant cracks are not a concern. Moderate cracks have widths from 0.012 inches wide up to 0.05 inches and may be measured with a finely-divided ruler or crack comparator card. Moderate cracks cause no distress to the structure, but will allow water and salts to enter the concrete mass with a path to the steel reinforcement. Moderate cracks should be regularly monitored to track any crack growth. Wide cracks are widths greater than 0.05 inches.. Depending on their location, wide cracks can be structurally significant and the condition could warrant a

structural review, see the field manual condition state 4 guidance in the reinforced concrete and prestressed concrete sections.

Figure 2.1.4.1-1: Map Cracking in Reinforced Concrete.

Figure 2.1.4.1-2: Moderate Longitudinal Crack in a Concrete Deck.

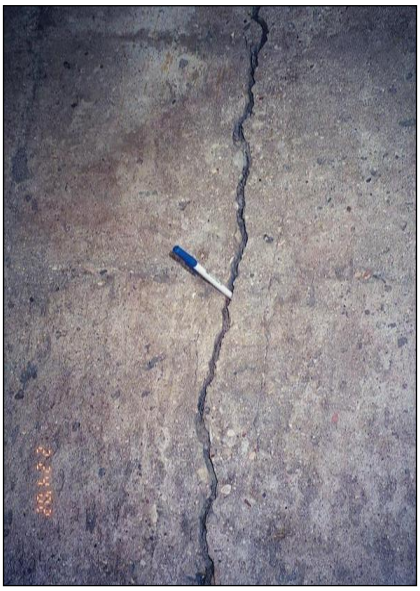

Figure 2.1.4.1-3: Wide Vertical Crack in an Abutment.

Crack Types

Cracks are grouped into two categories: structural and nonstructural. Structural cracks are caused by dead and live loads and include flexural and shear cracks. Flexural cracks always develop on the tension surface of reinforced concrete members. Flexural cracks are most commonly seen on undersides of beams between the piers, but are also found on continuous beam tops and decks above the piers due to negative bending moments. Columns under fixed supports may develop flexural cracks when expansion/shrinkage of the superstructure bends the pier. Flexural cracks are oriented perpendicular to the length of the member. The longest cracks are located in areas of the highest bending moment, sometimes extending up to about 75 percent of the member's depth. Flexural crack lengths shorten as bending moments are reduced. Shear cracks, if present, will always develop near piers and abutments and are always oriented diagonally. These cracks begin near the bearing and extend up towards midspan at an approximate forty-five degree angle.

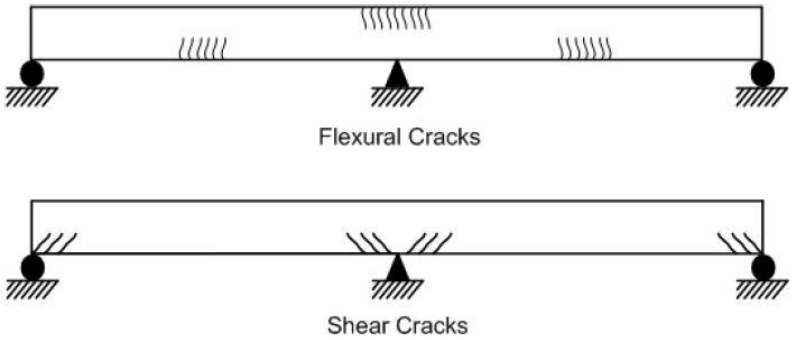

Figure 2.1.4.1-4: Flexural and Shear Cracking in a Beam.

Structural cracks develop when tensile stresses acting on the member exceed the tensile strength of concrete. Reinforcing steel embedded within the concrete picks up all tensile forces immediately upon cracking. Normal loading generally causes uniformly spaced insignificant or fine cracks in reinforced concrete. These are not generally viewed as a problem. Wide cracks accompanied with excessive deflections, or any flexural cracks found in prestressed beams, suggest that the bridge has been overloaded. Foundation movement or settlement can also cause flexural cracks in superstructure or substructure elements.

Nonstructural cracks do not generally affect the load-carrying capacity of the member. All concrete develops small, shallow, nonstructural cracks due to drying and shrinkage during the curing process. After curing, concrete creep (increased deflections or shortening under sustained loads) and seasonal temperature changes expand and contract the concrete, causing further random cracks. These long-term effects can have more serious consequences should movable bearings become locked up, restraining concrete movement and increasing the crack widths. A nominal amount of reinforcing steel placed transverse and longitudinal to the member controls widths and lengths of shrinkage and temperature cracks.

Crack Orientation

Depending on the cause of cracking, cracks can travel in several directions within a structural member. Their patterns may be described as follows:

1. **Map cracking** – nonstructural temperature/shrinkage cracks which travel randomly within the member. Several usually occur within any given area, giving the surface an appearance of a road map.

2. **Random cracks** - individual meandering cracks, usually found on decks.

3. **Transverse cracks** – cracks oriented perpendicular to the bridge centerline. May be structural flexural cracks on slab bridges or nonstructural temperature/shrinkage cracks on bridge decks.

4. **Longitudinal cracks** – cracks oriented parallel to the bridge centerline or horizontal members such as beams, girders, slabs, decks, or parapets.

5. **Horizontal cracks** – cracks oriented transversely to vertical members such as structural flexural cracks in pier shafts, pier columns, walls, and abutments.

6. **Vertical cracks** – cracks oriented vertically to vertical members such as pier shafts, columns, walls, and abutments.

7. **Diagonal cracks** – cracks angled with respect to the member centerline, commonly structural shear cracks.

8. **D-cracks** – letter “D” shaped cracks found at the edge of deck or slab joints.

9. **Radial cracks** – cracks oriented in a circular pattern; may indicate a punching shear type failure in a bridge deck.

Delaminations

Delamination is the separation of concrete at or near the level of the reinforcing steel, and is caused by reinforcing steel corrosion. Rust, the corrosion product of steel, has a volume approximately 7 to 10 times that of the original steel. Since concrete has only limited tensile strength to counteract this expansive force, it cracks internally in a plane along the layer of reinforcing steel. The presence of delaminations indicates that chlorides, salts, or other corrosive chemicals (generally carried as a solution in water) have been absorbed by or traveled through cracks in the concrete and reached the reinforcing steel.

Figure 2.1.4.1-5: Delaminations on Concrete Pier.

Detection of delaminations is often performed using a method called “sounding”. The most common form of sounding on concrete is using hammer tapping. A technique similar to hammer tapping, called chain dragging, is used to detect top of bridge deck delaminations. For each method, a loud popping or hollow sound is generated when the hammer hits or the chain drags over a delaminated area. Thin delaminations are more readily detected than thicker ones, as thicker delaminations will begin to sound more like undamaged concrete. Sometimes slight surface discolorations, surface cracks or rust stains indicate the presence of a delamination. Delaminations should be outlined with a lumber crayon or spray paint for monitoring the deterioration rate or for removal marking.

One high-tech method for delamination detection is infrared thermography. It is mostly used for large area deck investigations or high traffic situations. A truck-mounted infrared camera detects concrete defects by reading differences in deck surface temperatures. These temperature differences are generated because a crack within the concrete is a location of heat transfer discontinuity. Defects are then recorded on videotape and mapped on a bridge deck plan sheet. Because this method relies on temperature differentials, certain weather conditions must be met in order to properly and accurately carry out this testing. For other methods for determining concrete delamination please refer to Part 5 in the manual.

If it can be performed safely, removal of small delaminated areas may be done during an inspection. Removing the delamination allows the base concrete to dry out, slowing down the reinforcing steel corrosion process. Additionally, removing concrete in a controlled manner eliminates the possibility of the delamination suddenly falling onto the public or private property. Large areas of delamination, however, should be removed as a maintenance action.

Spalls

When a delamination is removed or falls off of the base concrete, the resulting depression is called a spall. Spalls are typically circular or oval in shape, often exposing the surface of the corroded reinforcing steel. Causes of spall formation include freeze/thaw action from trapped water, friction from thermal movement, expansive effects from reinforcing steel corrosion, and impact fractures.

Figure 2.1.4.1-6: Spalled Concrete on Pier Cap.

Reinforcing/Prestressing Steel Corrosion

The high alkaline environment of concrete temporarily protects the underlying reinforcing steel from corrosion. As the structure ages, however, chlorides from deicing salts, carbon dioxide, and other potentially corrosive atmospheres may reach the steel reinforcement through diffusion or cracks in the concrete cover. These chemicals, along with moisture and oxygen, form an electrolytic cell that corrodes the steel. As the corrosion advances, it expands, delaminating and spalling the concrete.

Heavy reinforcement corrosion can lead to a structural capacity loss in a member. As corrosion advances, the cross-sectional area of the reinforcing steel is reduced. Since the reinforcing steel must resist the same load, its stresses increase. Excessive section loss may lead to yielding or fracture during an overload. Fortunately, however, there are usually several reinforcing steel bars carrying the tensile loads in any one member. This results in redundancy and a lesser likelihood that a total member failure will occur. Corrosion of prestressing strands, however, is typically more detrimental than reinforced concrete. This is due to the higher stresses already existing in the strands. As the strand corrodes the strength

of the prestressed component is drastically reduced. Forensic studies done after a beam failure in 2005, determined that lightly corroded prestressed strands experienced a 20 percent reduction in tensile capacity where heavily pitted strands experienced up to a 29 percent reduction in tensile capacity.

Figure 2.1.4.1-7: Reinforcing Steel Corrosion.

Efflorescence and Leaching

Efflorescence, informally referred to as leaching, is a white deposit on the concrete surface caused by the crystallization of calcium carbonate from the cement paste and other recrystallized carbonate and chloride compounds. Water traveling through the concrete dissolves these salts and usually deposits them along cracks where the water exits. Efflorescence indicates that water and dissolved chemicals are able to pass through and contaminate the concrete.

Light efflorescence does not affect the compressive strength of the concrete paste. Heavy efflorescence, which is sometimes accompanied with rust stains from reinforcing steel corrosion, indicates a reduction in the compressive strength of the concrete and reduction in the structural member's overall strength.

Figure 2.1.4.1-8: Heavy Efflorescence on an Abutment and Slab.

Chloride Contamination

Concrete is typically exposed to chlorides, especially in Wisconsin, from deicing salts. Chlorides can also be found in acid rain or in some cases contaminated water during concrete mixing. Chloride ions attack and destroy the passivation layer surrounding reinforcing steel. With the passivation layer removed the reinforcing steel is vulnerable to corrosion. Using a practice no longer allowed, in the 1960's salt was added to concrete during mixing to prevent water from freezing. This caused accelerated corrosion in rebar which ultimately led to concrete cracking.

Carbonation

Carbonation is the reaction between carbon dioxide in the atmosphere and the compounds in cured concrete. Carbon dioxide reacts with cement reducing the alkalinity. If the extents of the carbonation reaction extend to or beyond the reinforcing steel, the lowered alkalinity jeopardizes the passivation layer surrounding the reinforcing steel. Once compromised the steel is vulnerable to corrosion. While a slow process, cracking in concrete allows carbon dioxide to penetrate further into the concrete.

Scaling

Scaling, also known as surface breakdown is the gradual loss of surface aggregates and cement paste caused by the chemical degradation of the cement bond. It is classified by ACI as light, medium, severe, or very severe. Light scaling indicates cement paste loss of up to 1/4 inch deep accompanied with surface exposure of course aggregates. Medium scaling indicates cement paste loss of from 1/4 inch to 1/2 inch deep, exposing the sides of the course aggregates. Severe scaling indicates cement paste loss of from 1/2 inch to 1 inch deep with clear exposure of the course aggregates. Finally, very severe scaling indicates a loss of course aggregate with depths greater than 1 inch. Reinforcing steel may be exposed.

Figure 2.1.4.1-9: Scaling on the Underside of a Concrete Box Girder Due to Fire.

Typically located along the gutter lines of the deck, scaling is common when deicing chemicals are used. Ponded salt water along the gutter, caused by clogged drains or scuppers, is the usual culprit for the chemical attacks. Other bridge components can sometimes be subjected to scaling by way of sulfate compounds found in soil and water. Fire damage, distinguished by a blackened concrete surface, can also cause scaling, since temperatures above 700 degrees Fahrenheit will weaken the cement paste.

Pop-outs

Pop-outs are small conical shaped depressions in the concrete surface caused by coarse aggregates which expand during moisture absorption. Shale is one common expansive aggregate, although in Wisconsin, cherts are generally responsible. Pop-outs may also be caused by reactive aggregates and high alkali cement. Pop-outs can normally be discerned by seeing the aggregate particle's fractured surface in the bottom of the depression. Pop-outs themselves are not a structural concern, although the pop-outs may eventually cause a rougher ride or help to speed the rate at which water can reach the underlying reinforcing steel.

Figure 2.1.4.1-10: Pop-out on Concrete Deck.

Honeycombs (Poor Consolidation)

Caused by congested reinforcing steel or improper concrete vibration during construction, honeycombs are voids within the concrete mass. Small honeycombs are not detrimental to the strength of the member, whereas large honeycombs may be structurally significant. Severe honeycombing may even leave some of the reinforcing steel within the member uncovered.

Only voids adjacent to the concrete surface are inspected by visual means. Current construction practice is to fill large surface voids with mortar.

Collision Damage

Trucks, over-height vehicles, derailed trains, and marine traffic may strike and damage concrete piers, abutments, or girders. A large section of concrete may be cracked or chipped off a structural member, exposing reinforcing steel or prestressing strands. If damaged sections are similar to spalls, patching may be used as a repair method. Damaged prestressed beams with exposed or broken strands can be repaired, but generally the beams are replaced. Very serious impacts can knock out pier columns or girders.

Abrasion and Wear

Concrete disintegrates over time with continuous external mechanical forces acting on its surfaces. Surface abrasion is caused by the erosive action of silt or sand found in fast-flowing rivers, streams, or surf zones. It is usually associated with piers, pilings, and abutments. Freeze thaw can accelerate abrasion damage. Wear is the scraping effect on the deck, curb, and parapets due to snow plows, street sweepers and friction caused by daily tire traffic. Severe wear is noted as revealed polished aggregate. This can create a safety concern on bridge decks if the deck is wetted.

Fire Damage

Though concrete is considered a fire-resistant material, fire can cause some deterioration. This will mainly be seen as a discoloration on the concrete surface, but pop-outs, scaling, delaminations, and spalls may be caused by the intense heat expanding the aggregate and reinforcing steel.

Overload Damage

Overload damage is caused from overstressing a structural component. Unlike cracking defects which are typically gradual over time, bridge components experiencing overloading may suffer serious structural cracking in as soon as a single loading. A concrete element that has undergone overloading may exhibit sagging, spalling or working flexural cracks at tension areas and/or diagonal shear cracking near support points.

2.2.4.2 Steel

Carbon steel is often referred to simply as steel and has been a widely-used building material since the later part of the 19th century. It has many advantages, including a high strength/weight ratio, ductility, elasticity, reliability, and versatility. Steel is differentiated from iron by the inclusion of carbon and other alloying elements. Many standard cross-sectional steel shapes are produced by rolling mills, such as angles, channels, and wide flange sections. Nonstandard shapes may be created by welding, bolting, or riveting together combinations of flat plates and/or smaller rolled sections.

Physical Properties

Steel weighs approximately 490 pounds per cubic foot. The chemical composition of steel varies widely depending on the type of steel. Carbon steel is a combination of iron, carbon (0.15 percent to 1.7 percent of carbon by weight), manganese (up to 1.65 percent), silicone, and copper (both up to 0.60 percent). For improved mechanical properties, weldability, and corrosion resistance, high-strength, low-alloy steels are produced. These steels may include the alloying elements used for carbon steels, plus others such as columbium, vanadium, chromium, and nickel. Still higher strength steels are available by using higher percentages of alloying elements and heat-treatments. These are known as quenched and tempered alloy steels, and they are not commonly used for bridge structures.

Unprotected structural steel will readily corrode or rust. Painting and galvanizing (the application of a sacrificial coating of zinc) are the most common methods used to protect steel against atmospheric corrosion. Some steels, known as weathering steels, oxidize to produce a dense protective coating (patina) which inhibits further corrosion. When used in the proper environment, these steels require no painting and develop an even-textured, dark red-brown appearance.

Mechanical Properties

Steel is a material used for its high tensile and compressive strengths. Depending on the steel type, yield strengths (the greatest stress a material can withstand without being permanently deformed) most commonly range from 33,000 pounds per square inch to

50,000 pounds per square inch. Tensile and compressive yield strengths are the same. Shear strength is about 60 percent of its yield strength.

When stressed below its yield point, steel is elastic, and a loaded beam will readily spring back into shape once the load is removed. Should high loads or impacts be applied which produce stresses beyond yield, steel is ductile and can withstand excessive deformations without failure. However, steel may become brittle due to the effects of welding, heat treatment, fatigue or very low temperatures.

Fatigue is a material failure under cyclic (repeated) loading. It only affects parts of a member's cross-section subject to varying tensile stresses or stress reversals (alternating compressive and tensile stresses). Steel strength is reduced if it is subject to a large number of stress fluctuations or reversals. This strength reduction depends on the number of load cycles, the magnitude of the stress fluctuation, and the type of detail involved. The strength reduction is small for base metal. Reductions in allowable strengths may be required when discontinuities (such as changes in cross section, cuts, tack welds, and rough edges) exist. Small discontinuities may be removed or improved by grinding to a smooth profile.

Steel Deterioration

Steel can experience many types of defects. Steel's susceptibility to fatigue damage makes reporting these defects all the more critical. The following information is a brief overview of causes of fatigue cracking in steel. An excellent resource for a more complete discussion of fatigue, as well as for example photographs and diagrams, is the *Manual for Inspecting Bridges for Fatigue Damage Conditions* by Yen, Huang, Lai, and Fisher.

Fatigue Cracks

Fatigue cracks are of primary concern since their growth can lead to sudden catastrophic failures. It is therefore extremely important for an inspector to understand where fatigue cracks are likely to occur and know the signs that indicate a crack.

Fatigue is material failure or fracture under a sufficient number of stress fluctuations. The stress fluctuations are caused by repeated member loading, such as trucks repeatedly driving over a bridge day in and day out. Each load cycle causes a stress loading and unloading in the primary supporting members. The stress generated by this cyclic loading is much lower than the steel's ultimate, or breaking, stress. Eventually, the steel will begin to crack. For example, consider a steel wire capable of suspending a permanent 100-pound weight just prior to breaking. This same wire may only be able to suspend a 75-pound weight, applied and removed several thousands of times, before it would finally break. Bending a paper clip back and forth several times until it breaks is an example of a fatigue failure.

Fatigue crack development depends on several factors. These factors include load frequency, stress type, stress range, and detail type.

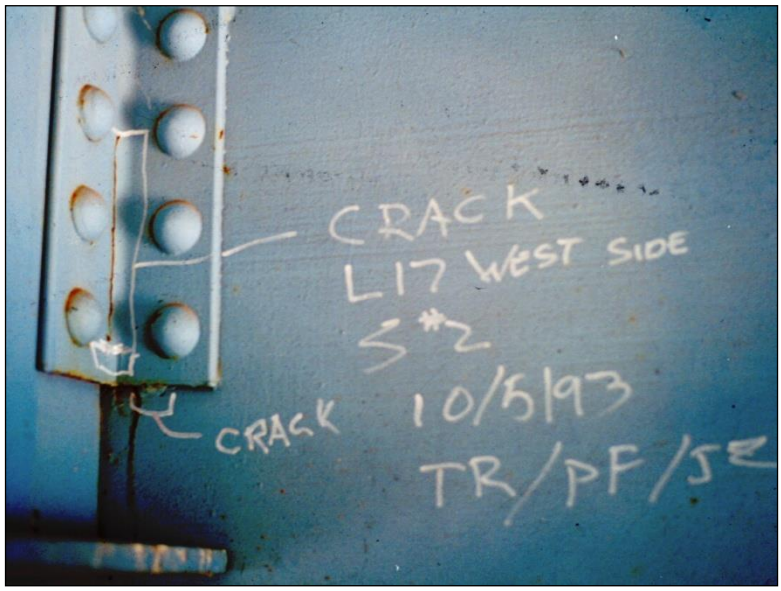

Figure 2.1.4.2-1: Fatigue Crack through Connection Angle and Tack Weld.

Load Frequency

Steel is more likely to crack under a high number of load applications. Interstate bridges that experience truck loadings on a daily basis are more prone to fatigue damage than rural or local bridges that may not carry truck traffic for days on end. Also, older bridges more than likely have seen a greater number of truckload applications than recently built structures.

Load frequency is generally a concern for highway and railroad bridges where heavy vehicles commonly use the structure. It is generally not a concern for pedestrian bridges where the frequency of design loads is relatively low.

Stress Type

Fatigue is most often a concern when the type of stress during each load cycle is either tension-tension or reversed (compression-tension). Several bridge members or components thereof are susceptible, including the tension chords, diagonals, verticals of truss bridges, the tension flanges of beam/girder bridges, and hangers of suspended girders or trusses. Beam webs can also be subject to fatigue cracking when cross-frames or diaphragms distort the web out-of-plane.

Cyclic compression-compression stresses can also be a concern, but any cracking in these locations is usually due to residual tensile stresses in the steel from welding or uneven cooling after rolling. Cyclic compressive stress is rarely a concern for bridges.

Stress Range

Another factor for steel's susceptibility to fatigue is the stress range. Steel is more likely to crack when the range from maximum to minimum stress during each load cycle is high. For example, a detail where the stress varies from –10 kips per square inch compression to 15 kips per square inch tension (stress range = 15 kips per square inch - (-10 kips per square

inch) = 25 kips per square inch) is more prone to fatigue cracking than the same detail which experiences stresses ranging from 10 kips per square inch tension to 30 kips per square inch tension (stress range = 30 kips per square inch - 10 kips per square inch = 20 kips per square inch). Maximum tensile stress is not a consideration when considering fatigue life.

Maximum yield strength is also not a consideration. Steel having yield strength of 50 kips per square inch does not have more fatigue capacity than steel with yield strength of 36 kips per square inch.

Detail Type

Secondary member attachments, weld flaws, welded cover plate ends, bolt holes, tack welds, etc. create a material discontinuity and introduce an interruption in the stress flow. Each discontinuity acts as a notch in the steel member, creating a sudden rise in the stress level. Smooth transitions are less susceptible to cracking; whereas sharp transitions, such as transverse welds at the end of a cover plate or gouges in the flange, are very susceptible.

Figure 2.1.4.2-2: Transverse Weld at a Cover Plate End.

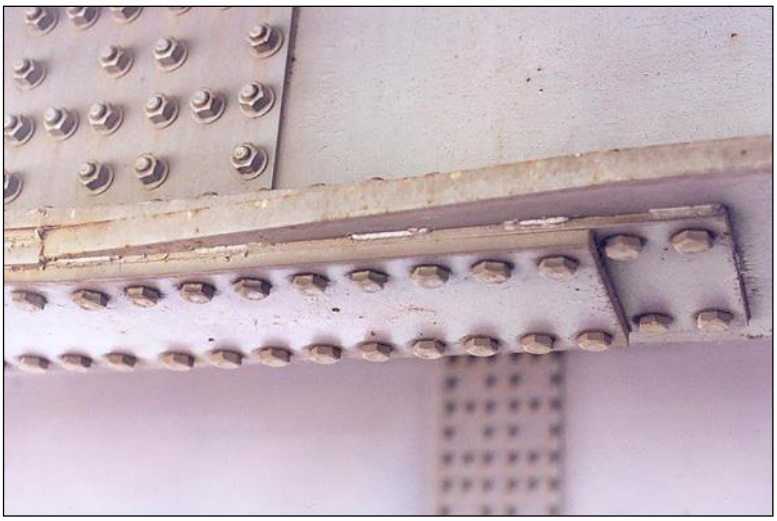

Figure 2.1.4.2-3: Tack Welds at a Splice Plate/Bottom Flange Interface.

Notches force the stress flow to suddenly “bend” around a corner. This sudden direction change produces a rise in the stress which may, on the microscopic level, reach yield. Repeated loading to yield will eventually fracture the steel, creating a crack (similar to the repeated bending of a paper clip mentioned above). Once a crack forms, the cross-sectional area of the member is reduced, leading to higher stresses and further crack growth. If the crack is left without arresting the crack, and occurs within a tension or stress reversal zone, the member could eventually tear itself apart.

The AASHTO bridge specifications have categorized several steel bridge details with respect to their susceptibility to fatigue cracking. All were categorized with respect to beam or girder in-plane bending and axial loading of members or member components. See Appendix D for these details.

Several inspection techniques are used to detect fatigue cracks. The most common and most important method is visual examination, as this is usually how cracks are found in the first place. While it would be ideal to examine every square inch of every element experiencing tensile stress variations or stress reversals, this is unnecessary and extremely uneconomical. Since fatigue cracks develop at discontinuities, it is reasonable to inspect local suspect details only. Telltale visual signs suggesting a crack include rust drips (bleeding), rust powder (due to rubbing along the crack), and small, usually rusty cracks. Some cracks could be very large, and in extreme situations they may even open and close under traffic loading. Another sign is a fine line of discoloration in the paint at the toe of a connecting weld, or on the surface of a weld. The *Manual for Inspecting Bridges for Fatigue Damage Conditions* contains several photos and diagrams illustrating these signs. Suspected cracks may be more easily confirmed by using a magnifying glass. All suspect cracks should be reported, and their location marked directly on the member with a permanent felt tipped marker or paint stick.

After visual detection, some small cracks may require confirmation through the use of nondestructive evaluation methods. Most commonly, magnetic particle or liquid (dye) penetrant testing is used because these methods are easy to administer, are inexpensive, give fairly quick results, and require fairly low-tech equipment. More advanced nondestructive techniques may be used to establish the extent of a crack within the base metal, since magnetic particle and dye penetrant testing will only reveal crack size at the metal’s surface. The most common high-tech method of examination is ultrasonic testing. Other more expensive methods include acoustic monitoring, eddy current, and radiography (x-ray/gamma ray).

Confirmed cracks should be re-examined during subsequent inspections until the cracks are eliminated or repaired. Shallow cracks may be ground or drilled out. Holes (sometimes referred to as “arrest holes”) may be drilled at the tips of long cracks to eliminate the sharp rise in stress in this area. Longer cracks typically require the drilling of arrest holes plus bolting splice plates to the member.

Although the base metal away from component details is generally not a fatigue concern (such as tension flange areas located between diaphragm connections), crack initiation points may still exist in the form of tack welds, welded flange splices not ground smooth, and notches resulting from traffic impact or fabrication carelessness. These flaws should be examined closely and reported. The flaws should also be re-examined during subsequent

inspections until they are eliminated or repaired. Tack welds or butt welds with the reinforcement left in place can generally be ground smooth, eliminating the defect. Notches are typically ground smooth to form a smooth, tapering transition in the plate.

A detail contributing to the cracking of many fabricated girders is the web-gap. This cracking and associated detail is not related to the in-plane bending details indicated in the AASHTO stress categories. Web-gap cracking is produced by out-of-plane, or sideways, web bending. A web-gap is created when a horizontal gusset plate is notched to fit around a transverse, vertical connection plate. The horizontal gusset plate, used to connect lateral bracing, is then welded directly to the girder web. The small space created between the transverse connection plate and horizontal gusset plate notch is called the web-gap. Forces in the lateral bracing/horizontal gusset plate push and pull on the girder web, causing it to bend sideways (out-of-plane). This bending is restrained due to the stiffness created by the transverse connection plate. As a result, the web is forced to deflect and bend over the very short gap distance, generating extremely high local stresses. After numerous load cycles, a vertical web crack within the web-gap will develop, creating a material discontinuity and notch with respect to in-plane bending. This effect is shown in Figure 2.1.4.2-4. If left without drilling out the crack tips, this crack can grow into the tension flange to create a critical situation.

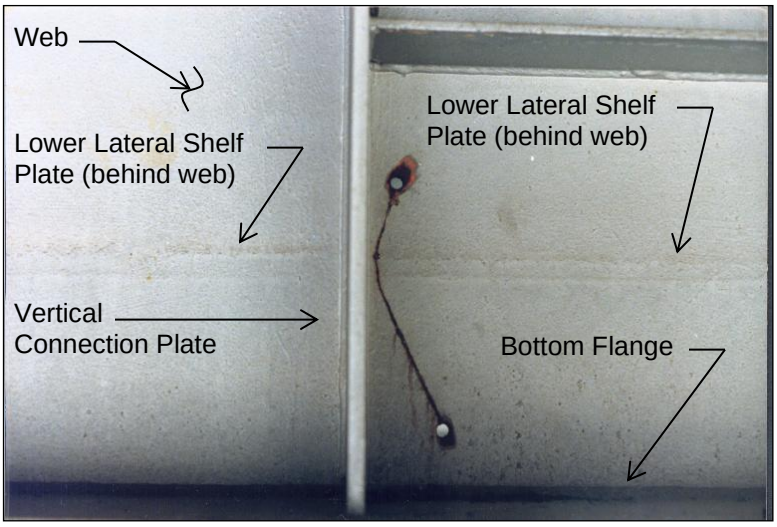

Figure 2.1.4.2-4: Girder Web Gap Fatigue Crack (Note Drilled Crack Arrest Holes at Crack Tips). Note the Outline of the Lower Lateral Shelf Plate Located on the Opposite Side of the Web.

Corrosion

Corrosion, or rust, is the most visible type of steel deterioration. It is a chemical reaction in which the iron in the steel combines with oxygen and moisture in the air to form ferric oxide and ferric hydroxide. More specifically, it is an electrochemical process between an area having a tendency to corrode (the anode) and an area with a lower tendency to corrode (the cathode). The anode and cathode may be located on the same piece of steel. A liquid electrolyte (water) must be present to allow for the flow of metal ions, and a conductor (the

steel itself) must be present to allow electron flow from the anode to the cathode. Iron in the steel dissolves in the water to form iron ions. These ions react with oxygen in the air to form rust at the anode. Electrons that flow from the anode to the cathode combine with other ions in the water, typically hydrogen ions, to form hydrogen gas. As a result, the cathode is left undamaged.

Figure 2.1.4.2-5: Steel Corrosion at Girder Splice.

The corrosion process can be stopped by preventing the electrolyte (water) from coming in contact with the base metal. In most steel bridge structures, this is done by the application of a protective paint coating. In lieu of painting, a zinc galvanizing coating may be applied. Since zinc has a greater tendency to corrode than iron, it becomes the anode and the steel becomes the cathode.

Once the base steel is exposed to the atmosphere, there are several causes for corrosion. The most common cause is the environment, and this primarily affects steel in contact with water or soil. Impurities in water, such as deicing chemicals, bird waste, atmospheric pollutants, and acids in the soil, produce ionic solutions which create more efficient electrolytes. The more efficient the electrolyte, the greater the rate of corrosion.

Other less common causes for corrosion include fretting, stress corrosion, bacteria, and DC currents. Fretting is the rubbing of two closely-fitted steel parts. Pitting and a red deposit occur at the interface. Stress corrosion occurs when a metal is loaded in tension. The tensile stress exposes an increased amount of surface area at the metal's grain boundaries, leading to corrosion and cracking. Waterborne bacteria, heavy clay, contaminated waters, etc., can destroy the steel's protective coating and sometimes corrode the steel itself through a process called microbial induced corrosion. Finally, stray DC currents from sources such as welding equipment, substations, and railway signal systems can also speed the rate of the electrochemical process.

The three commonly recognized stages of corrosion are light, moderate, and heavy. Light or surface rust is a loose form of corrosion usually appearing as a dark orange or light brown

color. Its effect is not serious, as it does not oxidize away a measurable amount of the steel. It can also be observed as light pitting or spotting on the paint surface. Moderate corrosion has a medium to dark brown color and a scaly appearance. Shallow pits may have formed in the steel surface. There is no paint left to protect the steel, and small amounts of section loss may have occurred. Heavy corrosion is easily recognized by its dark brown to almost black color and laminated rust (laminated rust occurring between two plies of plates is referred to as “pack rust”). These rust laminations may be easily removed with a chipping hammer, exposing large deep pits in the steel surface. Heavy corrosion causes reductions in plate thickness that can easily be seen with the naked eye or can even cause through-thickness section loss. Corrosion allowed to reach the heavy stage can present several problems for a steel member.

Loss of Section and Capacity

If left unchecked, corrosion will remove a significant quantity of a member’s cross-sectional area. This can be serious for members or member components designed to be highly loaded. Regardless if the member is new or severely corroded, the forces within the member remain unchanged. Member stresses, however, can increase significantly.

Figure 2.1.4.2-6: Corrosion and Loss of Section - Crushed Beam at Abutment.

For example, suppose a member has an area without corrosion of 3 square inches and carries a load of 60,000 pounds. Stress in the original member is 60,000 pounds ÷ 3 square inches, or 20,000 pounds per square inch. If corrosion removes one-third of the original cross-sectional area, leaving only 2 square inches of steel, the remaining stress increased by 50 percent to 60,000 pounds ÷ 2 square inches, or 30,000 pounds per square inch.

It is clear that for a given load, stress is inversely proportional to cross-sectional area. Continuing with the procedure above, a 50 percent reduction in area will lead to a stress of 40,000 pounds per square inch. If steel with yield strength of 36,000 pounds per square inch were used for the member, an overstress and possible failure may occur.

Creation of Stress Risers

Corrosion causes surface discontinuities in the form of rough edges and surface pits. Discontinuities act as notches. When stress “flow” is forced to bend around a notch, local stresses in the immediate vicinity are greatly amplified. Under repeated loading, the notch may act as a crack initiation point, and the high stress concentrations coupled with repeated loading will eventually tear the steel apart. As a result, a corroded member is more prone to fatigue damage than members with smooth surfaces without corrosion.

Pack Rust

Corrosion can occur between two plates that are riveted, bolted, or pinned together, such as field splices, bracing connections, cover plates, pin and hanger bars, bearings, etc. If the plates are not clamped tightly together or paint does not properly seal the edge of the faying surface, moisture is able to seep in between the plies. This moisture can remain for extended periods, since it is difficult for this area to dry out quickly. One particular problem area is where rivets or bolts are spaced too far apart and are unable to tightly clamp all surfaces of the plates together. When corrosion begins, it expands to separate the plates, creating prying forces on the fasteners. Advanced corrosion will exhibit stratified rust, plate bending, and popped rivets or bolts.

Figure 2.1.4.2-7: Pack Rust Prying Apart Flange Splice Plates.

Fixity at Moving Components

Corrosion that forms at expansion devices such as pins and hangers, sliding plate bearings, and pinned bearings can become so excessive that these devices “lock up”. When this occurs, unintended loads are introduced into the structure which could cause overstresses and possible component failure.

<u>Overloads</u>

An overload occurs when live loads crossing the bridge are so great that the structural members are stressed beyond their design strength. Compression members or components

may become unstable, or tension members may be stressed beyond their elastic limit. When these events occur, the member will permanently deform, referred to as plastic deformation. In tension components, a sign that an overload has occurred includes an area of “necking down”, where the member is permanently stretched and cross-section is reduced. Other signs are total member elongation or even fracture. For compression members, symptoms of overloading are buckled members that form either a single curved bow or double curved “S” shape. An indication that a compression member component has buckled is acute waviness in the plate.

Impact Damage

It is not uncommon for older bridges to have lower vertical clearance than bridges built by today’s standards. Some of the bridges may show signs of vehicular or ship impact damage. This damage normally occurs above the roadway or navigable channel on the fascia girder. Indications include dislocated and distorted members and scrape marks on the flange undersides. Distorted members or components may act as stress risers, making a normally sound component a fatigue prone detail.

Figure 2.1.4.2-8: Girder Bottom Flange Impact Damage.

Fire Damage

Unprotected steel has poor resistance to extended heat exposure. Although it is incombustible, it will become significantly weakened by fire. Yield strengths of steels normally used in bridge construction are reduced by about 23 percent at 800 degrees Fahrenheit, 37 percent at 1,000 degrees Fahrenheit, and 63 percent at 1,200 degrees Fahrenheit. Melting points of the many steel alloys vary, but the melting temperature of about 2,800 degrees Fahrenheit for pure iron is a good approximation.

2.2.4.3 Timber

Timber has been a widely-used building material for all of recorded human history. It is likely that the first constructed bridge was a tree intentionally felled across a chasm. Timber as a bridge construction material has many advantages, including excellent resistance to fatigue and impact loading, resistance to deicing chemicals and freeze thaw effects, a favorable strength to weight ratio, and ease of construction. Timber is readily available and relatively inexpensive, and it is a renewable resource. Traditionally, solid sawn lumber was used for all bridge components due to the abundance of large, old growth trees. Many of these old growth trees have since been cut down. To compensate, current manufacturing processes allow very large timber members to be created by glue-laminating several smaller pieces together. These members can be assembled into many types of structures including slabs, beams, trusses, arches, trestles, and even suspension bridges.

Physical Properties

Wood used for bridge construction weighs about 50 pounds per cubic foot, although this value can vary widely, depending upon the wood species and moisture content. It is a hygroscopic material that absorbs and loses moisture as the humidity of the air changes. These moisture content fluctuations cause the wood to expand and shrink. Wood is an orthotropic material, meaning that its physical properties parallel to the grain are different than that perpendicular to the grain. It is resistant to many chemicals. Heavy wood members are even resistant to fire by the formation of a char layer that acts to insulate the underlying sound wood.

One negative aspect of wood is that features related to tree growth, such as knots, splits, checks, and shakes, can adversely affect a member's strength. A second is that high moisture content can negatively affect the wood's strength. Finally, wood without a preservative treatment has limited resistance to decay or insect attack. Fungi, termites, carpenter ants, powder-post beetles, marine borers, and caddis flies are the most common sources for timber deterioration.

Mechanical Properties

Wood is a material used for its tension, compression, and bending capabilities. Its mechanical properties vary greatly from species to species. Because wood is an orthotropic material, the mechanical properties also vary depending on its principal axes of anatomical symmetry. Ultimate strengths for the most common properties used in design range from 6,600 to 17,500 pounds per square inch for tension parallel to the grain, 1,700 to 10,100 pounds per square inch for compression parallel to the grain, 3,900 to 20,200 pounds per square inch for bending, and 500 to 2,600 pounds per square inch for shear parallel to the grain. Similar properties in wood's other two orthogonal directions (perpendicular to the grain in the radial and tangential directions) will have different ultimate strength values.

For most engineering applications, wood is elastic, and a beam loaded and stressed below its ultimate strength will spring back into shape once the load is removed. Recovery from its deformed loaded shape may not be immediate, and if the load had been in place long-term, recovery will take a longer period of time. Wood is susceptible to creep. Creep is a gradual deformation under a sustained load. Over a period of years, the initial deflection of a wood beam can eventually double under high permanent loading.

Should high dynamic impact loads be applied, wood is a very resilient material. It can sustain loads up to twice the amount that would produce failures if applied statically.

Fatigue is a material failure under cyclic (repeated) loading, affecting only parts of a member's cross-section subject to varying tensile stresses or stress reversals (alternating compressive and tensile stresses). Fatigue damage is not a concern for wood since its fibrous structure is resistant to cyclic loads.

Timber Deterioration

As with any material, timber is susceptible to deterioration and damage. Untreated wood can suffer structural damage due to biological attack. Treated or untreated wood is susceptible to structural damage from mechanical or atmospheric sources, such as vehicle impact, overloads, and cracks due to drying, and from fire.

Biologically Induced Damage

Biological damage to wood is caused by living organisms. The main defense against such attacks is by treating the wood with chemical preservatives.

Fungi

Decay caused by fungi is the primary reason for timber bridge replacement. Brown rot and white rot are the two fungi most responsible for structural damage. They feed upon the cellulose and lignin that make up the wood's cell wall configuration and give it its strength. Brown rot makes the wood dark brown and crumbly. Because its enzymes can diffuse into the wood far from their source, brown rot can weaken the wood substantially in the early stages of its attack. White rot makes the wood white and stringy. Though the enzymes do not migrate into the wood as with brown rot, white rot uses more of the wood cell wall as a food source, causing more severe, localized decay.

Other fungi may be present on a timber member, but do not cause any serious structural damage. However, their presence indicates that conditions are right for the growth of brown or white rot. These "indicator" fungi include molds, which have cottony or powdery appearances and vary from white to black in color. Stains may appear as specks, spots, streaks or patches of varying color on the wood surface. Soft rot does actually attack the wood, but only to just below the surface making the wood soft and spongy.

Conditions must be right for fungi to survive. There must be sufficient oxygen and a minimum of 20 percent moisture present in the wood. Fungi must have a food source, and this is the wood itself. Temperatures must range between 32 degrees and 90 degrees Fahrenheit, with the rate of biological activity being higher at warmer temperatures. Essentially, these are the same conditions required for human survival. Areas on a bridge favorable for fungi growth commonly exist at connections, supports, splices, and the waterline or ground line. Other areas of a bridge usually lack one of the components necessary for fungi survival, such as a lack of oxygen and excessive moisture in submerged timber piles. The only way to stop the natural decay due to fungi is by using wood preservatives, which poison the fungi food source.

Parasites

Wood parasites are insects, mollusks or crustaceans that live within or feed upon the timber member.

Termites: Termites are insects recognized for their ability to damage wood and have a northern habitat range within Wisconsin. Termites found in Wisconsin feed on damp wood usually in contact with the ground. Their tunnels contain no exit holes, so a termite-damaged timber may look as good as sound timber. A sharp tap on a timber's surface, however, will easily punch a hole into the termite-damaged interior. Signs of termite activity are mud tubes that run from the ground to the wood member.

Timber damage due to termites on frequently used bridges is rare due to the constant vibrations of daily traffic.

Carpenter Ants: Carpenter ants do not feed on wood, but tunnel through a timber's interior for shelter. As with termites, they damage the interior of a timber so that infestation is not readily apparent. They will, however, leave a pile of sawdust at the entrance tunnel, signaling their presence.

Powder-Post Beetles: Larvae of powder-post beetles feed on the interior of timbers, creating tunnels and many exit holes. A powdery residue is often packed into these exit holes, indicating their presence.

Caddisflies: Caddisflies are insects found in fresh or brackish water. Caddisfly larvae use timber piles for protection by boring holes into the sides of the timber. They are attracted to timber experiencing fungal decay. Since they do not eat the wood, they can also be found in piles treated with creosote.

Marine Borers: Marine borers are saltwater-borne parasites and do not affect timber bridges found in Wisconsin.

Mechanically Induced Damage

Mechanical damage is usually caused by the live loads acting on the bridge.

Overloads

Overload damage occurs when live loads acting on the bridge are so great that the structural members are stressed beyond their ultimate strength. Compression members may buckle into a single curved bow or double curved "S" shape, crack or even break. Tension and bending members will crack or fracture. Cracks due to overloading will generally occur perpendicular to the grain, splintering the wood. Overloads may not only be caused by traffic but also by foundation settlements.

Impact

Impact damage, most commonly caused by vehicular collisions, normally occurs above the roadway or navigable channel on the fascia girder. It can also occur on the main load-carrying members of through trusses and arches. Stream debris or ice floes may also impact

bridge members situated within the channel. Indications of impacts include damaged timbers and scrape marks.

Abrasion/Mechanical Wear

Abrasion is most often seen on timber bridge decks and is caused by vehicle tires and snow plows. These forces can cause ruts in the deck that hold water, further weakening the wood. Mechanical wear occurs from fasteners rubbing against their holes at loose connections.

Atmospheric Related Damage (Weathering)

The weathering effects of moisture, light, and heat can adversely affect the physical properties of wood. Moisture loss in wood can cause dimensional instabilities of a timber, resulting in warps, cracks, and shrinkage.

Warping

Warping of a wood member is caused by uneven drying, allowing one side of the timber to shrink at a different rate than the other. As a result, the member may bow, twist or cup.

Cracks

Moisture loss in a timber will often cause the member to shrink, causing cracks. Cracks create openings into a timber's untreated interior, allowing biological agents to enter and begin the decay process. Cracks may also reduce the timber's strength. Checks are cracks oriented parallel to the grain and perpendicular to the annual growth rings. Splits are advanced checks that extend completely through the member.

Some cracks, called shakes, may form before a tree has been felled. Shakes are oriented parallel to both the grain and annual growth rings.

Loose Connections

Improperly dried wood may shrink during service, resulting in loose connections. Loose connections may allow biological agents to enter the untreated timber interior. Mechanical wear due to the fastener rubbing against the wood may also take place under live load.

Fire Damage

Though fire can completely consume a small timber member, large timber members often used for bridges offer some resistance to fire. Fire consumes wood at a rate of about 0.05 inch per minute for the first 30 minutes, and about 0.021 inch per minute thereafter. As wood burns, a black char layer is formed. This char helps to insulate the underlying unburned wood, slowing the consumption rate. Large timbers have enough volume to develop this protective char layer, leaving a core of undamaged wood. Though the remaining wood core does not have the strength of the original undamaged timber, it is often enough to prevent a total collapse.

2.2.4.4 Other Materials

Stone Masonry

Stone masonry is rarely used in modern construction as a structural material for bridges. However, it was used extensively for abutments and piers in the 1800s and early 1900s. It was also used during this time to build arch superstructures and culvert structures. Different types of stone were used, depending on local availability. These are most commonly limestone, sandstone, and granite.

Stone masonry is classified as rubble masonry, square-stone masonry or ashlar. Rubble masonry is rough cut stones used for random coursed or roughly coursed construction. Square-stone masonry is roughly squared and dressed and may be laid in random or coursed construction. Ashlar masonry is precisely squared and finely dressed and may be laid in random or coursed construction.

Physical Properties

Stone masonry weighs between 135 pounds per cubic foot (sandstones) to 170 pounds per cubic foot (granites). It expands and contracts with increasing and decreasing temperatures, respectively. It may be porous and therefore absorbs moisture. Limestone, common in Wisconsin, tends to be more absorptive than most other stones. Stone is a durable material, and different stone types will each have a different durability.

Mechanical Properties

Stone masonry is a material used for its compressive strength. Depending on the material type and where it had been quarried, compressive strengths generally range from 6,000 pounds per square inch (limestones and sandstones) up to 60,000 pounds per square inch (granites). Its tensile strengths are poor, ranging from about 2 to 13 percent of its compressive strength. Mortar is used to form a bedding material for each masonry unit, bond individual stone units together, seal joints against moisture penetration, and seal irregularities on the masonry unit’s surface. Mortar itself is almost always weaker than stone.

Stone Masonry Deterioration

The three main causes of stone masonry deterioration are splitting, spalling, and weathering.

Figure 2.1.4.4-1: Masonry Deterioration/Spalls at an Arch Bearing.

Splitting

Splitting refers to the seams or cracks that may form in rocks. This common type of deterioration may occur due to freezing water within small seams and pores, due to volume changes from seasonal temperature fluctuations, and due to the wedging force of plant roots growing into crevices and joints. Structural overloads may also cause the stone masonry units to split.

Figure 2.1.4.4-2: Split Masonry Unit with Failing Mortar Joints.

Spalling

Small pieces of rock which break out or chip off of the stone masonry unit are called spalls. Sources of spalling are the same as those that produce splits. Vehicle impacts may also produce spalls.

Weathering

Weathering is the degeneration of the rock surface into small granules. Causes that chemically attack the stone include lichens and ivy, as well as gasses and solids dissolved in water, including deicing agents. Windborne or waterborne particles can cause abrasion as well.

Cast Iron

Cast iron is a material not used in modern construction as a structural material for bridges. However, it was used for the compression members and bearing castings of very old bridges. Cast iron is produced by pouring molten iron into a mold and letting the metal solidify.

Figure 2.1.4.4-3: Cast Iron Arch.

Physical Properties

Cast iron has a gray color due to the presence of graphite particles distributed throughout the metal. It weighs about 450 pounds per cubic foot. The chemical composition of cast iron varies widely depending on the type. However, the most common gray cast iron is composed mainly of iron, carbon (2.0 percent to 4.0 percent of carbon by weight), and silicone (up to 2.8 percent). Other elements include sulfur, phosphorus, and manganese.

Unprotected cast iron will corrode, but tends to be more corrosion-resistant than steel. It is usually painted to protect against atmospheric corrosion.

Mechanical Properties

Cast iron is a material used for its high compressive strength. The tensile yield strength of gray cast iron is about the same as its ultimate tensile strength of 20,000 pounds per square

inch to 30,000 pounds per square inch. Its compressive yield strength varies from approximately 80,000 pounds per square inch to 100,000 pounds per square inch.

The free carbon and slag in cast iron make it a brittle material, make it difficult to weld, and gives it poor resistance to shock and impacts. It has poor resistance to fatigue loading, but good damping and machinability properties.

Cast Iron Deterioration

Types of cast iron deterioration are similar to those found on steel (see Section 2.1.4.2).

Wrought Iron

Wrought iron is a material that is not used in modern construction as a structural material for bridges. In fact, it is no longer produced in the United States. It is, however, still found on many old bridges as tension members. Wrought iron is produced by mechanically rolling or working relatively pure iron into a desired shape, producing a fibrous material with properties in the worked direction similar to steel.

Physical Properties

Wrought iron weighs about 480 pounds per cubic foot. The chemical composition of wrought iron is mainly of iron, slag (iron silicate, up to 3.0 percent by weight), and phosphorous (approximately 1.2 percent). Other elements include sulfur, manganese, and carbon.

Unprotected wrought iron will corrode, but tends to be more corrosion-resistant than steel. Its fibrous nature produces a tight rust less likely to flake and scale as in structural steels.

Mechanical Properties

Wrought iron is a material used for its high tensile strength. Its yield strength is about 30,000 pounds per square inch. When wrought iron is worked or rolled, the slag distributed throughout the metal is elongated into fibers. Because of this, it is an anisotropic material, having different mechanical properties with respect to the direction of the slag fibers.

The slag fibers give wrought iron many desirable properties. It surpasses steel in its ductility, fatigue strength, and corrosion resistance. It has good machinability properties and good impact and shock resistance. Wrought iron is also weldable.

Wrought Iron Deterioration

Types of wrought iron deterioration are similar to those found on steel (see Section 2.1.4.2).

Cast Steel

Cast steel is a material not normally used in fixed bridge construction, but it has been used for tracks in movable bridges. Cast steel is produced by pouring molten metal into a casting mold directly from the steel-making furnace, and letting it solidify.

Physical Properties

Several types of cast steel exist, including carbon steel, low alloy steel, alloy steel, and stainless steel. Cast steel weighs about 490 pounds per cubic foot. The chemical composition of cast steel varies widely depending on the type. However, carbon steel castings are composed of iron and carbon (0.2 percent to over 0.5 percent of carbon by weight). Unprotected cast steel will corrode.

Mechanical Properties

Cast steel is a material used for its machinability qualities. The yield strengths of carbon steel castings commonly used for bridges range from 35,000 pounds per square inch to 95,000 pounds per square inch, and tensile strengths range from 60,000 pounds per square inch to 120,000 pounds per square inch. Low alloy and alloy bridge castings are stronger, and can have yield and tensile strengths up to 135,000 pounds per square inch and 140,000 pounds per square inch, respectively

Cast Steel Deterioration

Types of cast steel deterioration are similar to those found on steel (see Section 2.1.4.2).

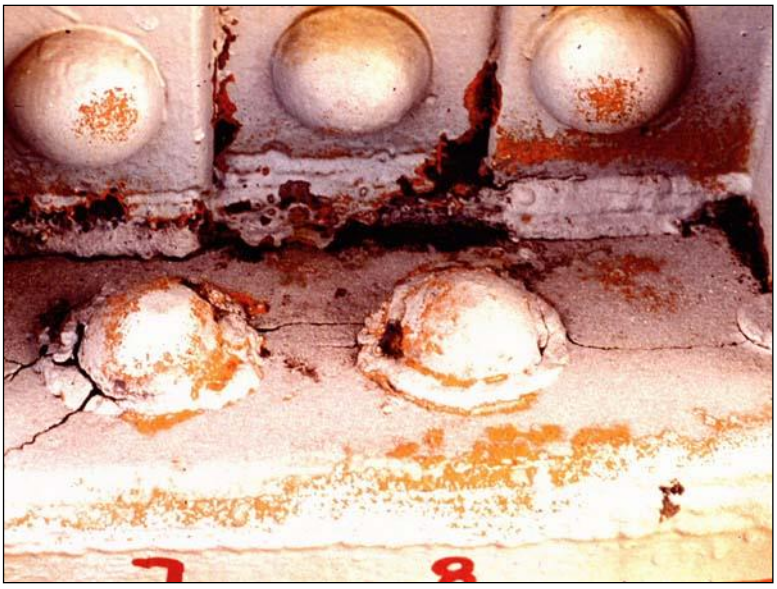

Figure 2.1.4.4-4: Fatigue Cracks on a Steel Casting.

Aluminum

Aluminum is not normally a material used as a structural material for vehicular bridges. However, it may be found as part of the deck, superstructure or substructure on pedestrian bridges where fatigue is not a concern.

Physical Properties

Aluminum weighs about 175 pounds per cubic foot. The chemical composition of aluminum varies widely depending on the type. However, the most common aluminum used for structural applications is composed mainly of aluminum, magnesium, silicon, copper, and chromium. Unprotected aluminum is much more corrosion-resistant than steel. Because of this, painting is usually unnecessary to protect it from the atmosphere.

Mechanical Properties

Aluminum is a material used for its high tensile and compressive strengths. Depending on the aluminum type, yield strengths range from 40,000 pounds per square inch to 48,000 pounds per square inch, giving it a high strength to weight ratio. Tensile and compressive yield strengths are the same. Shear strength is about 60 percent of its yield strength. The modulus of elasticity values of aluminum are approximately one third that of steel. Modulus of elasticity is a material property relating material stress to elongation. Since modulus of elasticity values are inversely proportional to elongations, an aluminum beam will deflect three times as much as a similarly shaped steel beam carrying the same load. In addition, aluminum's fatigue strength is approximately one-third that of steel, making it less desirable when fatigue is a concern.

When stressed below its yield point, aluminum is elastic, and a loaded beam will readily spring back into shape once the load is removed. Should high loads or impacts be applied which produce stresses beyond yield, aluminum is ductile and can withstand excessive deformations without failure. Although weldable, aluminum experiences significant base metal strength reductions of up to one half the yield strength in the vicinity of welds.

Aluminum Deterioration

With the exception of atmospheric corrosion, types of aluminum deterioration are similar to those found on steel (see Section 2.1.4.2).

Composites

Composites are plastics/resins reinforced with carbon, glass or other fibers. In Wisconsin, their use as main load carrying members for bridges is limited. Wisconsin has several research bridge decks that utilized composites for concrete deck forms, some in combination with deck reinforcement. These can be tracked using the special component feature.

Composites can have a high strength-to-weight ratio, can attain strengths similar to steel, and have excellent corrosion and impact resistance. There are numerous types of plastics available, but their properties can lack consistency from producer to producer. Also, costs are presently very high (although prices may come down as more is being produced).

Composites exhibit viscoelastic (nonlinear) behavior. This means that at any given strain magnitude, temperature changes can produce marked changes in strength and deflection. Similarly, load duration and magnitude affect a composite's strength and deflection. Viscoelastic behavior is less pronounced in composites than in plain, unreinforced plastics/resins.

Though fatigue data does exist for some of the composites, no method is available to characterize fatigue strengths of composites.

2.2.5 Classification of Fixed Bridges

Bridges are classified according to their superstructure type. The use of each type generally depends on the distance the bridge must span, although more than one type can be used for the same span length. Other factors include depth of channel or ravine to be crossed, required underpass clearance, horizontal curvature, and aesthetics.

2.2.5.1 Slab

Slab bridges are the simplest bridge structures, constructed using reinforced concrete or timber to span distances from 20 feet to about 45 feet. Longer spans are usually continuous, often with a haunch (thickened slab) over the piers. Spans up to 75 feet may be obtained if the slabs are prestressed. The slab acts as a very wide beam spanning between substructure elements. Because live loads are applied directly to their top surfaces, slab bridges (including flat slabs, haunch slabs, and rigid frames) are unique in that the superstructure is the same element as the deck.

Figure 2.1.5.1-1: Heavily Deteriorated Single Span Concrete Slab.

Figure 2.1.5.1-2: Three-Span Concrete Slab with Haunches.

2.2.5.2 Beam/Girder

Beam/girder bridges rely on the use of two or more primary elements acting in bending to support the deck and traffic. Beam/girder bridges are usually constructed of steel or concrete, although timber is also sometimes used.

The shortest steel bridges are typically constructed using standard hot-rolled “I” shapes, referred to as beams. Spans are generally limited to the range of 20 feet to 90 feet. Longer steel spans require deeper sections that are not produced by rolling mills. For these situations, welded, or historically bolted or riveted “I” shapes fabricated from plates are used. When a shape is built-up by welding, or historically bolting or riveting together plates and structural shapes, it is sometimes referred to as a girder instead of a beam. Still longer spans may consist of steel plates built-up into a four-sided rectangular or trapezoidal closed shape, known as a box girder. Due to the box girder’s inherent resistance to the effects of torsion, box girders are commonly used for horizontally curved spans. Box girders are capable of reaching very long span lengths.

Figure 2.1.5.2-1: Steel Multiple Girder Bridge.

Figure 2.1.5.2-2: Prestressed Concrete Multiple Girder Bridge.

Many types of concrete beam bridges exist, since concrete can be cast into many different shapes. The earliest examples are cast-in-place structures, usually of single spans. The rectangular beams and deck were typically cast simultaneously, forming “T” beams with the deck acting as the top flange. On many older structures, the fascia beams protrude above the deck. These beams are part of the superstructure and double as bridge parapets.

Prestressed concrete girders, cast at an off-site plant and delivered to the site, gained acceptance in the 1950s. By compressing the girder with steel tendons prior to the application of dead load, internal tensile stresses in the concrete are greatly reduced or eliminated. This technology has allowed concrete to span much farther than conventionally reinforced concrete. Prestressed girders come in a variety of geometric cross-sections, including “I”, “C”, “T”, “Bulb-T”, and closed box shapes.

For longer spans or horizontally curved structures, post-tensioned concrete boxes may be used. Rather than being prestressed at a plant, the girders are either cast-in-place, or segments thereof are precast at a plant and lined up to form a girder on site. Both contain ducts through which post-tensioning rods or tendons are passed. The rods/tendons are then pulled into tension, compressing the girder.

Rectangular timber beams may be either sawn or glue-laminated and are used for fairly short span bridges. Solid sawn beams are most often associated with older structures built during a time when large timber members were plentiful. To create the same size or larger beams nowadays, multiple strips of wood are glued together to build up a laminated, or glulam, member.

2.2.5.3 Truss

A truss is a structure whose members are arranged to form triangles. Each member is classified as either a top chord, bottom chord, vertical or diagonal. Most, but not all, trusses have vertical members. The mechanics of this arrangement is such that each member is acting as either a pure tension or pure compression member. Two parallel trusses (rarely more than two) form the main load-carrying system of a bridge superstructure. Each may be thought of as a very deep beam with holes cut into the web, the top chord acting as the top flange, and the bottom chord acting as the bottom flange. Thus, under positive bending, the top chord is in compression, and the bottom chord is in tension. The reverse is true if the truss undergoes negative bending. A truss' diagonal and vertical members usually have one end connected to the top chord and the other to the bottom chord. The diagonal members deliver the "shear" loads of the imaginary deep beam to the supports by means of tension or compression, depending on their orientation within the truss.

Truss bridges are mostly constructed using steel members, while very old trusses used timber and wrought iron. Loads are delivered to trusses by way of the deck bearing on stringers (longitudinal beams), which in turn bear on floor beams (transverse beams). The floor beams frame into the sides of the trusses, usually at the panel points. Timber trusses are used only for fairly short spans. Simply supported steel trusses were used for spans from about 100 feet upwards to 800 feet. Cantilevered and continuous spans ranged from 500 to 1,500 feet.

Figure 2.1.5.3-1: Parker Truss Bridge.

Figure 2.1.5.3-2: King Post Truss Bridge.

Figure 2.1.5.3-3: Pratt Truss Bridge.

2.2.5.4 Arch

Arch bridges generally transmit their loads to the ground by diagonally pushing on their supports, rather than bearing vertically as with slabs, beams or trusses. To resist this push, or thrust, buttresses are built at the arch ends, or the ends are tied together with a tension member in a manner similar to an archer's bow. Arches resist a combination of compression, bending, and shear. The relative magnitude of each depends upon the shape of the arch. True arches are parabolic in shape and are subjected only to compressive forces. For practical reasons, most arches are not designed or built as true parabolas and therefore must transmit bending and shear, in addition to compression.

Tied arch bridges are similar to arches with one significant variant. Rather than the diagonal thrusting forces being absorbed by the foundations, tied arches transfer the outward thrusting force to ties that run from one end of the arch to the other, like the string in an archer's bow. The ties are entirely in tension. The tied arch foundations, therefore, only carry the dead and transient loads.

Arches have been used for a wide range of bridge spans, from under 50 feet up to 1,700 feet. Almost all types of construction materials have been used to build arches, including stone masonry, wood, cast iron, steel, and concrete. They may be comprised of two or more parallel ribs or of a single curved member called a barrel. Arch ribs can be constructed with steel, concrete, or wood. Usually, only stone masonry and concrete are used to form barrel arches.

The space between the flat deck and curved arch is called the spandrel. Open spandrel arches use columns placed within the spandrel to transfer the live loads and deck dead loads to the arch ribs. Closed spandrel arches usually use earth fill retained by spandrel walls to transfer the live loads and deck dead loads to barrels. Other closed spandrels do not use earth fill, but leave the spandrel unfilled or vaulted. Deck live and dead loads are delivered to the barrel by way of the spandrel and interior walls.

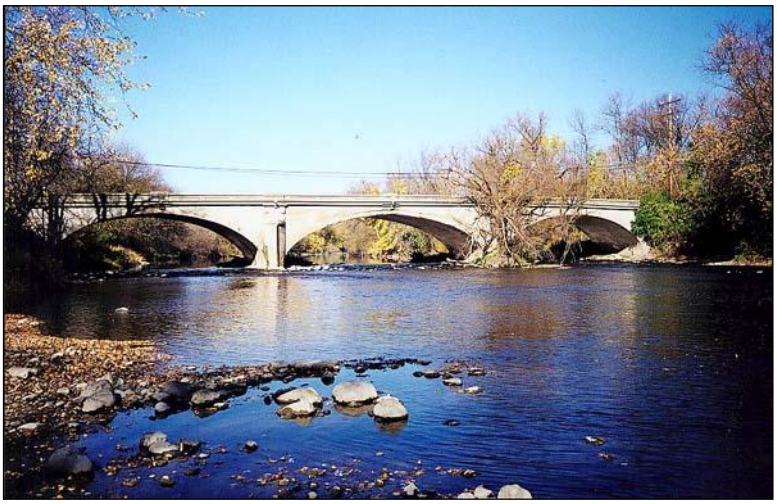

Figure 2.1.5.4-1: Closed Spandrel Concrete Arch Bridge.

Figure 2.1.5.4-2: Steel Open Spandrel Arch.

2.2.5.5 Rigid Frame

Rigid frames are similar to arches in that they transmit their loads to the ground by diagonally pushing on their supports. However, since they consist of horizontal members rigidly connected to the tops of inclined members, they have the advantage of reducing span length

while allowing more head clearance for traffic traveling underneath. Rigid frames primarily resist bending and shear forces, with compression loads occurring mostly in the vertical or slanted legs.

Only steel and concrete are used for rigid frame bridge construction. Steel rigid frames are usually multi-span structures known as “K-frames”, with main spans ranging from 50 feet to 200 feet. They are built of welded plate girder construction and require a minimum of two frames placed parallel to the roadway to carry the deck and live loads. Concrete rigid frames can be built as single or multi-span structures, with a main span of each up to 50 feet in length. Single span concrete rigid frames are commonly slab type structures, similar to barrel arches. Multi-span bridges are usually built of multiple frames similar to steel rigid frames.

Figure 2.1.5.5-1: Steel Rigid Frame.

2.2.5.6 Cable-Stayed

Cable stayed bridges have superstructures supported by diagonal cable tension members. Each cable stay is connected to a pylon (tower) located at the main pier. Superstructures may be built of steel or concrete.

Figure 2.1.5.6-1: Cable-Stayed Pedestrian Bridge.

The cable stays act as spring supports, causing the superstructure to act as a multi-span bending member. Since the cables are sloped relative to the roadway, the cable stays also introduce compressive forces into the superstructure. Compression forces are also introduced into the pylon at each cable stay connection point. Because of the possible unequal live loading from span to span, variable cable stay forces also cause the pylon to bend. Back stay cables, either anchored to the approach spans or anchorage block on land, help to balance the forces of the main span cables, thereby minimizing pylon bending.

Cable stayed bridges are not very common in Wisconsin. Currently, there are only a handful of structures that are used as either pedestrian bridges or vehicular bridges. When used for highway bridges, they can be used for spans from 200 feet to 2,000 feet.

2.2.5.7 Suspension

Suspension bridges are uncommon in Wisconsin, currently existing only for pedestrian structures. When used for highway bridges, suspension bridges can span distances in excess of 1,800 feet. They use vertical cable hangers to suspend the superstructure from two or more main cables. The main cables are draped over towers and terminate at heavy anchor blocks. The main suspension cables can have diameters exceeding 3 feet for long roadway spans. The main cables exert large compressive loads on the tops of the towers and may introduce some bending due to unequal live loading from span to span.

The distance between adjacent hanger cables is relatively small, and therefore only a relatively shallow superstructure is required for strength. However, this results in a very flexible structure, giving rise to large deflections. Because of this, deep stiffening trusses, girders, or box girders are normally used to more evenly distribute the live loads among the hangers, thereby reducing deflections. Stiffening trusses and girders for highway suspension bridges are normally built of steel.

Figure 2.1.5.7-1: Pedestrian Suspension Bridge.

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2.2 CONDITION EVALUATION PROCEDURES

2.2.1 Introduction

The biggest challenge in any bridge inspection program is to relate the material distress found on a bridge to its effect on the structure's strength and safety. Another challenge is to create uniformity between all bridge inspectors for rating the structural condition of a bridge.

It is expected that all qualified bridge inspectors have a basic understanding of bridge mechanics and of how deterioration of a certain bridge component will affect (or will not affect) the bridge's performance and public safety. So that all bridge inspectors will more or less come to the same conclusions regarding the structural condition of a bridge and causes for its distress, two standard inspection methods are currently used in Wisconsin: Element Level inspections and National Bridge Inventory (NBI) inspections. Both are currently used for all inspections.

2.2.2 Element Level Inspection

Element Level bridge inspections have been used since the early-1990s. This inspection method developed because of the need for bridge owners to manage and allocate the limited funds available for maintenance, repair, and rehabilitation of their structures. It identifies problems at particular components of a structure and recommends measures to be taken to ensure the longevity of a bridge. The intent of Element Level inspections is to capture all major bridge components in an effective simple way while providing agencies with an appropriate amount of flexibility. The Element Level inspection method breaks the bridge down into several elements, such as the railing, deck, girders, diaphragms, abutments, pier columns, etc. Each element is inspected for defects. The defects are assigned a Condition State based upon its state of deterioration. The various defects quantified on an element then determine the Condition State of the element as a whole. Condition States are numeric ratings that reflect the state of deterioration of an element. Every element has the same possible number of condition states, four which are comprised of the following overall rating: Condition State 1 – Good, Condition State 2 – Fair, Condition State 3 – Poor and Condition State 4 – Severe. Each defect condition state uses standard phrases to describe the deterioration. The standard phrases increase uniformity and consistency of bridge inspection reports. Many elements, depending on their unit of measure, can be split into several different defects and assigned different Condition States to reflect the differing categories and differing rates of deterioration that often exist on any one bridge element.

One of the end results of performing Element Level inspections is the generation of a database for the bridge management system computer software. By developing a database over time, bridge deterioration rates based upon material, geographic location, age, usage, type of crossing, prior rehabilitation or preventive actions, etc. can be estimated. The software modeling capabilities allows comparisons between the effectiveness of preventive and corrective actions, predictions of estimated future deterioration, and life cycle costs. In this way, informed decisions can be made regarding prioritizing funds, when (or when not) to take action, and what type of action to take for the maximum benefit of money spent.

Per the Federal Highway Administration, all State and Federal agencies, were required to collect element level data for every NHS Bridge by October 1, 2014 and submitting this data

to the FHWA by April 1, 2015. Wisconsin has been performing element level inspections since the early 1990's.

The American Association of State Highway Transportation (AASHTO) has released an updated version of its element level inspection manual titled, *Manual for Bridge Element Inspection, 1st Edition.* Elements are broken down into three categories: National Bridge Elements (NBEs), Bridge Maintenance Elements (BMEs) and Agency Defined Elements (ADEs).

- National Bridge Elements (NBEs) are the primary structural components which determine the general overall condition and safety of members carrying the primary load. These are to remain unchanged from one agency to another across the nation in attempt to facilitate and standardize bridge element condition descriptions.

- Bridge Maintenance Elements (BMEs) are more generic and can be modified by an agency as needed. They are defined with modifiable recommended condition assessment language and are purposefully fairly general in order to allow flexibility for the agency to develop specific local bridge preservation practices and standards. Additional BMEs can also be created by the agency as needed. These added elements should take into account such factors as element performance, deterioration rates, feasible actions, preservation costs, and also the practicality of said elements.

- Agency Developed Elements (ADEs) are elements an agency can create for added flexibility. These may be sub-elements of NBEs or BMEs or may be elements independent of national standards and recommendations.

- All elements have associated material defects. Those elements comprised of the same material share the same defect. Defects are a refinement to the element level inspection process to further break down and more accurately reference element deterioration. WisDOT has the opportunity to create its own defects if the Department has a deficiency it would like to capture.

2.2.3 National Bridge Inventory (NBI) Inspection

The Federal Highway Administration (FHWA) *Specifications for the National Bridge Inventory* (*SNBI*) is the basis for the NBI inspection condition ratings. The *SNBI* provides guidance for rating several components of the bridge. By using standard phrases related to each rating code, increased uniformity and consistency can be achieved. Aside from establishing a component's physical condition, rating data is used in a variety of analyses and decisions performed by Wisconsin Department of Transportation (WisDOT) and the FHWA. The data helps in determining the sufficiency of a bridge to remain in service, rehabilitation and replacement eligibility, etc. Based on this inspection method, funding for maintenance, repair, and rehabilitation is determined on an individual bridge basis.

In contrast to the Element Level inspection, NBI inspection results rate as a whole only major bridge components without being specific as to where, how much or what type of deterioration exists. Each bridge component is assigned a numeric rating code ranging from 9 to 0, with 9 being "excellent condition" and 0 being "failed condition". The ratings represent the overall physical condition of the component as compared to the day it was built. It is

purely an evaluation of the bridge component's material and its state of deterioration and not an evaluation of its ability to carry current legal loads. Sound engineering judgment is required in order to provide the most accurate condition of the primary structural components. For more information see the SNBI manual found on the FHWA website.

2.2.4 Agency Submittal Requirements

Both NBI Condition Ratings and Element Level Condition States of NBEs are required to be reported to FHWA. Wisconsin has been cataloging data from both inspection methods for several years.

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DECKS AND SLABS

2.3.1 Introduction

Decks are structural components of a bridge, and they serve several functions. First, they provide a smooth riding surface for traffic crossing the bridge. Second, they distribute the bridge live loads (such as vehicle wheel loads) laterally to the girders, stringers, and floor beams. Third, reinforced concrete decks may also function as part of the top flange of steel open girder, steel box girder, prestressed concrete and concrete box girder bridges. They always serve as the top flange of reinforced concrete T-beam bridges.

Decks receive a considerable amount of abuse from sources such as wheels, snow plow blades, and road debris that cause wear and abrasion; truck overloads that can cause overstress within the deck material; deicing chemicals that cause corrosion; freeze/thaw weathering effects that can cause material breakdown; and parasites/fungi that cause biological material deterioration. To protect the deck from these sources of deterioration, wearing surfaces, such as overlays or running boards, are often placed on top. Sidewalks and medians are also considered a protective wearing surface for the deck or slab.

Decks should not be confused with slab bridges. Decks are elements separate from the superstructure. They are placed on top of girders, stringers or floor beams, typically spanning laterally between these superstructure elements. Slabs do not bear on separate superstructure elements. Slabs are superstructures themselves, spanning longitudinally from substructure unit to substructure unit.

The slab acts as a single, wide beam spanning from substructure unit to substructure unit. There are no individual beams with this type of bridge, and the slab also acts as the deck. Slabs are used for simple spans of about 45 feet or less. Continuous slab bridges can be built with slightly longer span lengths. To attain greater negative bending strength on continuous bridges, the slab may be thickened (haunched) over the piers. The main reinforcing steel is placed parallel to traffic and located towards the bottom of the slab in positive bending regions, and towards the top of the slab in negative bending regions. On older and more complex structures, continuous cast-in-place slabs may contain voids to lighten the dead load of the bridge.

Slabs represent the simplest superstructures and have been in use for many years. Concrete slab elements are currently the most economical method to span short distances. Slab elements are most commonly constructed using cast-in-place methods, although plant fabricated precast/prestressed or post-tensioned hollow core planks have also been used to create slab bridges. Precast slabs are different than precast box beams in that precast slabs contain two or more voids through them, while box beams have only one. Precast slab may also be designed solid, with no voids in them.

Slabs are most commonly comprised of concrete, although timber slabs do exist. These elements are typically deeper dimension lumber post-tensioned together transversely with rods. Other materials may also be used so long as they are capable of carrying Wisconsin legal loads.

2.3.2 Reinforced Concrete Elements

Concrete decks and slabs are by far the most common element an inspector will encounter. They may or may not be covered with an overlay.

2.3.2.1 Reinforced Concrete Deck (Element 12)
Reinforced Concrete Slab (Element 38)

Element Level Inspection

These elements define all reinforced concrete bridge decks and slabs regardless of the wearing surface or protective systems used. Deck and slab elements shall be used to evaluate only the edge and bottom surfaces of the respective element. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). Timber planks used along running wheel lines shall be included under the wearing surface Element 510 – Wearing Surface (Other). Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects.

Concrete overlays are sometimes difficult to distinguish from the bare structural concrete deck. Therefore, the inspector should make it a habit to review the Construction History on the Element Level Inspection form. The Construction History should list any overlays that were added as well as the year the overlay was constructed. Likewise, if an inspector realizes that an overlay was added to a bridge but the Construction History does not reflect the change, the inspector shall make note of the overlay in the Construction History, update the overburden data, and check the rerate flag.

Wearing Surfaces (including overlays, sidewalks, and raised medians) are separate elements from reinforced concrete deck, slab and top flanges. Thin overlays are defined as being less than 1-inch in thickness, and may be materials such as Portland cement, epoxies or other polymers. It is important for the inspector to keep in mind that overlays do not increase the structural capacity of the deck. In fact, overlays slightly reduce the structural capacity due to the increased dead load (overburden). For this reason, it is important to provide the correct overburden depth on the inspection report. Think of the overburden depth to be the non-structural thickness that doesn't include the original deck or slab. The milling thickness does not matter, if the overlay thickness is 2.75" then 2.8" should be entered into HSI. Where there is adjacent girders touching each other with a concrete topping then that topping and any additional overlays count toward the overburden depth.

Due to the possible presence of an overlay (top), structural deck (integral structural concrete overlay) or stay-in-place forms (bottom), deck top or bottom surfaces that are not visible for inspection shall be assessed based on the available visible surface. If both top and bottom surfaces are not visible, the condition shall be assessed based on destructive and nondestructive testing, or indicators in the materials covering the surfaces.

By today's engineering practices, concrete decks and slabs in Wisconsin are constructed with epoxy coated reinforcing or stainless steel reinforcement to help protect the reinforcing steel nearest the riding surface from the corrosive effects of deicing chemicals. However, in

the past not all bridges were constructed with these types of reinforcing bars. The deck element will note the type of protective coating (if any) on deck and slab reinforcing. The inspector is responsible for reporting this data, and the inspector will be able to update the element data where he/she is aware of reinforcing bars utilized within the deck or slab element. Refer to the Concrete Reinforcing Steel Protective Systems section in Chapter 6 Part 2 for more information.

Sometimes the inspector will encounter a deck with two configurations. A common example of this occurs on bascule bridges, where the approach spans often have concrete decks (with or without an overlay) and the movable span has a steel grid deck. For this situation, two deck elements are required to be entered on the inspection form, with the square footage distributed appropriately between the elements and broken out into each Condition State as necessary.

On the inspection report form, reinforced concrete deck or slab is recorded in units of “square feet”. The correct method for calculating the deck or slab area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks) and accounting for any flares or ramps present. The overhangs located under parapets and sidewalks are included in the deck or slab measurement. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

The deck or slab evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. This quantifies the element’s condition which is used to track the progression of material degradation and generate quantity/cost estimates for future remedial work.

Problems on deck or slab elements the inspector should look for include:

- Transverse flexural cracks adjacent to and over piers. These are caused by superstructure negative bending.
- Diagonal or transverse temperature/shrinkage cracks. Diagonal cracks are typically located on the corners of skewed decks or slabs.
- Longitudinal flexural cracks caused by deck positive bending between the girders or stringers. Wide cracks may indicate a serious structural overload. Note the maximum crack widths measured on the bridge inspection report.
- Efflorescence, discoloration, and rust staining which will often accompany cracks.
- Indications of corroding reinforcing steel such as rust stains or exposed reinforcement. Since section loss associated with reinforcing steel corrosion can reduce the deck or slab’s strength, measure this loss if possible and record it.

- Delaminations, spalls, and exposed/corroded reinforcing steel. Deteriorated concrete in the flexural zones may cause debonding of the reinforcing steel.

- Loose or spalling concrete located over a traveled way such as a roadway, train track, river, or pedestrian path. These can fall and cause damage or injury. Such loose concrete should be reported and recommended for removal by a maintenance crew if it cannot be safely done during the inspection.

- Cracks, delaminations, spalls, and exposed reinforcing steel located under drains or leaking expansion joints.

- Punching shear failures. Although they are rare, punching failures will show up as a localized area of radial-shaped cracks with the deck bulged downward.

Figure 2.3.2.1-1: Reinforced Concrete Slab w/ Spalling and Exposed Corroded Reinforcing - Condition State 3.

When the inspector is evaluating the bottom of the deck, it is important to assess the amount of deterioration as it is a valuable window used to assess the structural condition of the interior and top of deck. The amount of deterioration on the underside is a good indication of how much moisture is passing through the element.

Temperature and shrinkage cracks are common and will be found on most every concrete deck or slab. However, many types of cracks are more indicative of a structural issue. All types of cracking must be accounted for and a portion of the total quantity must be assigned to a Condition State. Types of cracks commonly encountered include the following:

- Temperature and shrinkage cracks (non-structural). These will be apparent on most concrete decks. These types of cracks almost always show up at the acute corners of skewed decks.

- Transverse flexural cracks (structural) due to negative bending will most likely appear over the piers of continuous superstructures.

- Transverse reflective cracks (non-structural) may appear at closely spaced, regular intervals on plank decks with asphalt overlays. These are caused by differential plank deflections.

- Transverse reflective cracks (non-structural) may appear adjacent to an expansion joint. These cracks suggest that the joint anchorage hardware is beginning to fail.

- Longitudinal flexural cracks (structural). These are caused by negative bending of the deck over the girders or beams.

- Longitudinal reflective cracks (non-structural) may appear along the joints of prestressed concrete channels or box beams placed tight, side by side. This cracking is caused by differential beam deflection.

- Longitudinal construction joints in overlays (non-structural).

Figure 2.3.2.1-2: Cracking of Concrete Deck (Localized Area Over Pier) – Condition State 2.

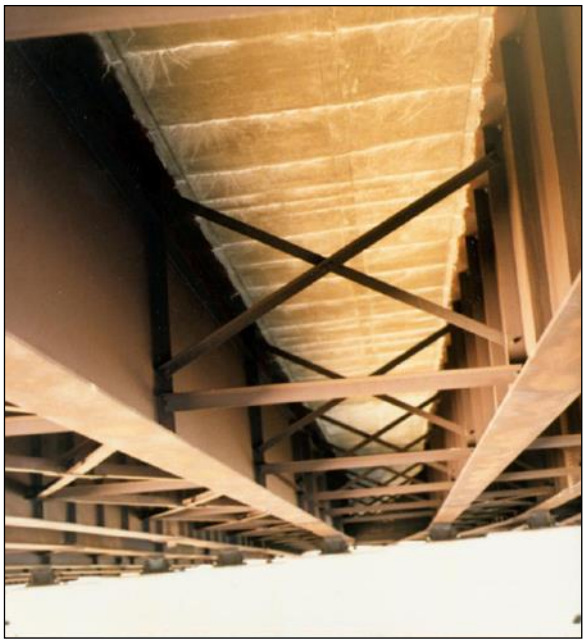

Figure 2.3.2.1-3: Heavy Density Cracking of Concrete Deck w/ Efflorescence – Condition State 3.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/ Spall/ Patched Area (1080)
- Exposed Rebar (1090)
- Cracking (RC and Other) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Chloride Concentration (8905)
- Precast Concrete Connections (8906)

Condition State Commentary

The Condition States for describe repaired areas. Repaired areas are considered "distressed" if the repair is a short-term maintenance action that does not restore the deck's structural integrity (such as filling potholes with asphalt). Therefore these repairs do not improve the Condition State rating of an element. Repaired areas are considered "rehabilitations" if the repair (such as full or partial depth concrete patches) improves the deck's structural integrity, thereby improving the rating after work has been completed.

Figure 2.3.2.1-4: Typical Reinforced Concrete Slab Distress.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.2.2 Reinforced Concrete Top Flange (Element 16)

There are several concrete structures that have concrete superstructures whose top components act as the deck of the structure. In these structures the deck is composite with the girder and acts as the compression flange of girder as well as potentially the wearing surface for vehicles. The following structures are common reinforced concrete superstructures with monolithic top flanges.

1. **Tee beams:** Tee beam bridges were commonly constructed in the early half of the 20th century. They are cast-in-place structures, with the deck cast monolithic with the beams. The “tee” shape is created by the rectangular beam stem below the deck, with the deck forming the top flange. In Wisconsin, the fascia beams on some tee beam bridges are upturned, doubling as parapets. Tee beam bridges are most commonly used for simple spans, although they may be made continuous by haunching the beam stems over the piers. Individual spans may reach 50 feet in length, with the beams spaced from about 3 to 8 feet. Common beam depths range from 18 to 24 inches. The main reinforcing steel is placed longitudinally towards the bottom of the beam in positive bending regions and longitudinally within the deck in negative bending regions. Transverse, vertical stirrups placed along the beams serve as shear reinforcing.

   Historically Wisconsin has evaluated tee beams utilizing the elements reinforced concrete deck and reinforced concrete open girder/beam and continues to utilize this convention over the use of reinforced concrete top flange.

2. **Through girders:** Through girder bridges were constructed in the early half of the 20th century. They are cast-in-place structures, with the deck cast monolithic with the girders. Two girders normally are used, and these very deep girders serve as the bridge parapets. The deck spans between the girders, connected to the lower portion of the girders. Through girder bridges are used for simple spans of 30 to 60 feet. Because the deck must span between the girders, through girder bridge widths rarely exceed 24 feet. The girders themselves are fairly large, being 18 to 30 inches wide and 4 to 6 feet deep. The main reinforcing steel is placed longitudinally towards the bottom of the girders, while the main deck reinforcing steel is placed transversely towards the deck bottom. Transverse, vertical stirrups placed along the girders serve as shear reinforcing.

Figure 2.3.2.2-1: Reinforced Concrete through Girder Bridge.

3. **Channel beams:** Channel beam bridges use precast channel beams as the primary load-carrying members. The channels are placed on the substructure units so that they form an upside down "U", with the vertical legs forming the beams and the horizontal top slab forms the deck. The channels are placed tight side by side and maybe transversely connected so that the individual beams act as a unit under live loads. Grouted shear keys may also have been utilized to help the beams act together. Channel beam bridges are used for simple spans up to about 50 feet. Widths of the individual beams usually range from 3 to 4 feet. The main reinforcing steel is placed longitudinally towards the bottom of the channel legs, while the main deck reinforcing steel is placed transversely in the top slab. Vertical stirrups may be placed along the channel legs and serve as shear reinforcing.

Please refer to Chapter 4 Superstructures, for additional information concerning these bridges. The element discussed in this section deals only with the top flange portion of these superstructures where traffic rides directly on the structural element.

Element Level Inspection

This element shall be used for all reinforced concrete bridge girder top flanges where traffic rides directly on the structural element regardless of the wearing surface or protection systems used. Top flange elements shall be used to evaluate only the edge and bottom surfaces of the respective element. Top Flange elements are considered deck elements. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects. See section 2.3.2.1 for guidance on overburden depth.

The inspector should always review the *Construction History* on the backside of the Element Level Inspection form. The *Construction History* should list any overlays that were added as well as the year the overlay was constructed. Likewise, if an inspector realizes that an overlay was added to a bridge but the *Construction History* does not reflect the change, the

inspector shall make note of the overlay in the *Construction History*, update the overburden data, and check the rerate flag.

On the inspection report form, reinforced concrete top flange is recorded in units of “square feet”. The correct method for calculating the flange area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks), and accounting for any flares or ramps present. The overhangs (not monolithic with the top flange) located under parapets and sidewalks are not included in the flange width measurement. This quantity is for the flange only, all girder components (web, bottom flange, etc.) should be quantified under the appropriate superstructure element. Please refer to Chapter 4 of Part 2 for Superstructure Elements.

The top flange evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

By today’s engineering practices, concrete decks and slabs in Wisconsin are constructed with epoxy coated reinforcing to protect the reinforcing steel nearest the riding surface from deicing chemicals. However, in the past not all bridges were constructed with epoxy coated bars. The inspection report form will note the type of coating on deck and slab reinforcing. The inspector will not be responsible for reporting this element, although the inspector will be able to add the element if he/she is aware of coated bars within the deck or slab element. Refer to the Concrete Reinforcing Steel Protective Systems section in Chapter 6 Part 2 for more information.

When the inspector is evaluating the bottom of the flange, it is important to assess the amount of deterioration as it is a valuable window used to assess the structural condition of the interior and top of the flange element. The amount of deterioration on the underside is a good indication of how much moisture is passing through the element.

If stay-in-place forms (bottom) prevent the inspector from viewing the bottom surfaces, those surfaces shall be assessed and evaluated based on the available visible surface. If both top and bottom surfaces are not visible, the condition shall be assessed based on destructive and nondestructive testing or indicators in the materials covering the surfaces.

Problems on the top flange element the inspector should look for include:

- Transverse flexural cracks adjacent to and over piers. These are caused by superstructure negative bending.
- Diagonal or transverse temperature/shrinkage cracks.
- Longitudinal flexural cracks caused by deck positive bending between the girders or stringers. Wide cracks may indicate a serious structural overload. Note the maximum crack widths measured on the bridge inspection report.

- Efflorescence, discoloration, and rust stains, which will often accompany cracks.
- Delaminations, spalls, and exposed/corroded reinforcing steel. Deteriorated concrete in the flexural zones may cause debonding of the reinforcing steel.
- Indications of corroding reinforcing steel, such as rust stains or exposed reinforcement. Since section loss associated with reinforcing steel corrosion can reduce the deck's strength, measure this loss if possible and record it.
- Loose or spalling concrete located over a traveled way such as a roadway, train track, river, or pedestrian path. These can fall and cause damage or injury. Such loose concrete should be reported and recommended for removal by a maintenance crew if it cannot be safely done during the inspection.
- Cracks, delaminations, spalls, and exposed reinforcing steel located under drains or leaking expansion joints.
- Punching shear failures. Although they are rare, punching failures will show up as a localized area of radial-shaped cracks with the deck bulged downward.

Figure 2.3.2.2-2: Reinforced Concrete Box Girder Top Flange/Deck Localized Rebar Corrosion and Bond Loss.

Temperature and shrinkage cracks are common and will be found on most every concrete surface. However, many types of cracks are more indicative of a structural issue. All types of cracking must be accounted for and a portion of the total quantity of the area must be assigned to a Condition State. Look on the top of the flange (deck side) for any cracking. Types of cracks commonly encountered include the following:

- Temperature and shrinkage cracks (non-structural). These will be apparent on most concrete surfaces. These types of cracks almost always show up at the acute corners of skewed decks.

- Transverse flexural cracks (structural) due to negative bending will most likely appear over the piers of continuous superstructures.

- Transverse reflective cracks (non-structural) may appear at closely spaced, regular intervals on plank decks with asphalt overlays. These are caused by differential plank deflections.

- Transverse reflective cracks (non-structural) may appear adjacent to an expansion joint. These cracks suggest that the joint anchorage hardware is beginning to fail.

- Longitudinal flexural cracks (structural). These are caused by negative bending of the deck over the girders or beams.

- Longitudinal reflective cracks (non-structural) may appear along the joints of prestressed concrete channels or box beams placed tight, side by side. This cracking is caused by differential beam deflection.

- Longitudinal construction joints in overlays (non-structural).

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/ Spall/ Patched Area (1080)
- Exposed Rebar (1090)
- Cracking (RC and Other) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Chloride Concentration (8905)
- Precast Concrete Connections (8906)

Condition State Commentary

The Condition States for describe repaired areas. Repaired areas are considered “distressed” if the repair is a short-term maintenance action that does not restore the deck’s structural integrity (such as filling potholes with asphalt). Therefore these repairs do not improve the Condition State rating of an element. Repaired areas are considered “rehabilitations” if the repair (such as full or partial depth concrete patches) improves the deck’s structural integrity, thereby improving the rating after work has been completed.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.3 Prestressed Concrete Elements

Components are pre-cast away from the bridge site and either pretensioned prior to concrete placement or post-tensioned after the concrete has been placed and cured. This tensioning of the strands creates compressive forces in the concrete component which reduces the amount of tension cracking that would occur in a conventionally reinforced cast-in-place deck.

Another prestressed bridge deck component is the monolithic top flange of a prestressed girder which acts as a driving surface. A common example would be a tee-beam bridge. Similar to a slab, the flange and beam is one unit and act as both the deck and superstructure. These beams are prevalent in Accelerated Bridge Construction (ABC). Pre-cast prestressed concrete bridge components provide an owner with a faster alternative than time-consuming concrete curing of cast-in-place systems. The costs are greater, however, for the component materials, as well as the additional equipment needed for the panel installment.

Prestressed concrete slabs, similar to reinforced concrete slabs, comprise the deck and superstructure. There are no girders or monolithic legs below the slab. The slab element itself spans from one substructure unit to the next. Examples of prestressed concrete slabs include:

- **Solid core prestressed planks:** As a superstructure, these components would be set adjacent to one another with only a joint separating each plank. The planks are either grouted together along a joint on the driving surface or tensioned together transversely by post tensioning. By appearance, these would look very similar to box beams. The inspector will need to use the plans to determine the actual composition of the superstructure.

- **Inverted Tee Beams:** Similar to solid core planks, these beams are set adjacent to one another separated by only a joint. A cross-sectional view of the component explains the name as the beam appears like an upside-down T. The stem of the tee is much wider than most girders as it acts partly as the compression flange. The key formed between the webs of adjacent beams is filled with concrete and typically a reinforced concrete deck is cast over the top of the beams to add structural capacity and further tie the beams together.

- **Post Tensioned Cast-in-Place Slabs:** These superstructures are essentially reinforced concrete slabs only more shallow and constructed with post-tensioning ducts throughout to strengthen the slab after concrete placement. Verification through the bridge plans may be necessary by the inspector.

2.3.3.1 Prestressed Concrete Deck (Element 13)
Prestressed Concrete Slab (Element 8039)

These elements define all prestressed concrete bridge decks and slabs regardless of the wearing surface or protective systems used. The deck and slab evaluation is all inclusive with the defects observed on the bottom and edges of the element and being captured using the defined condition states. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects.

Due to the possible presence of an overlay (top), structural deck (integral structural concrete overlay) or stay-in-place forms (bottom), deck top or bottom surfaces that are not visible for inspection shall be assessed based on the available visible surface. If both top and bottom surfaces are not visible, the condition shall be assessed based on destructive and nondestructive testing or indicators in the materials covering the surfaces.

These decks are not to be mistaken for Prestressed Concrete Top Flange, which are integral prestressed decks and girders where the top flange acts as a deck as well as compression flange of the girder. Prestressed Concrete Decks are typically comprised of precast panels that lie transversely or longitudinally on the girders with a gap between. The gap is filled with concrete to create a composite superstructure. Panels may also be overlaid with concrete creating a jointless deck. This cast-in-place deck would still be considered part of the Prestressed Concrete Deck element.

Element Level Inspection

By today's engineering practices, concrete decks and slabs in Wisconsin are constructed with epoxy coated reinforcing to protect the reinforcing steel nearest the riding surface from deicing chemicals. However, in the past not all bridges were constructed with epoxy coated bars. The inspection report form will note the type of coating on deck and slab reinforcing. The inspector will not be responsible for reporting this element, although the inspector will be able to add the element if he/she is aware of coated bars within the deck or slab element. Refer to the Concrete Reinforcing Steel Protective Systems section in Chapter 6 Part 2 for more information.

Sometimes the inspector will encounter a deck with two configurations. A common example of this occurs on bascule bridges, where the approach spans often have concrete decks (with or without an overlay) and the movable span has a steel grid deck. For this situation, two deck elements are required to be entered on the inspection form, with the square footage distributed appropriately between the elements.

On the inspection report form, prestressed concrete decks or slabs are recorded in units of "square feet". The correct method for calculating the deck or slab area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks), and accounting for any flares or ramps present. The overhangs located under parapets and sidewalks are included in the deck or slab measurement.

The deck or slab evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

When the inspector is evaluating the bottom of the flange, it is important to assess the amount of deterioration as it is a valuable window used to assess the structural condition of the interior and top of the flange element. The amount of deterioration on the underside is a good indication of how much moisture is passing through the element.

Problems on the element the inspector should look for include:

- Transverse flexural cracks adjacent to and over piers. These are caused by superstructure negative bending.
- Diagonal or transverse temperature/shrinkage cracks.
- Longitudinal flexural cracks caused by deck positive bending between the girders or stringers. Wide cracks may indicate a serious structural overload. Note the maximum crack widths measured on the bridge inspection report.
- Efflorescence, discoloration, and rust staining which will often accompany cracks.
- Delaminations, spalls, and exposed/corroded reinforcing steel. Deteriorated concrete in the flexural zones may cause debonding of the reinforcing steel.
- Indications of corroding reinforcing steel such as rust stains or exposed reinforcement. Since section loss associated with reinforcing steel corrosion can reduce the deck's strength, measure this loss if possible and record it.
- Loose or spalling concrete located over a traveled way such as a roadway, train track, river, or pedestrian path. These can fall and cause damage or injury. Such loose concrete should be reported and recommended for removal by a maintenance crew if it cannot be safely done during the inspection.
- Cracks, delaminations, spalls, and exposed reinforcing steel located under drains or leaking expansion joints.
- Punching shear failures. Although they are rare, punching failures will show up as a localized area of radial-shaped cracks with the deck bulged downward.

On the report, the inspector should describe cracks in terms of their location, orientation, length, and maximum width. Indicate also if the cracks are structural or nonstructural.

Temperature and shrinkage cracks are common and will be found on most every concrete deck or slab. However, many types of cracks are more indicative of a structural issue. All

types of cracking must be accounted for and a portion of the total quantity of area assigned to a Condition State. Look on the top of the deck for any cracking. Types of cracks commonly encountered include the following:

- Temperature and shrinkage cracks (non-structural). These will be apparent on most concrete decks and overlays. These types of cracks almost always show up at the acute corners of skewed decks.

- Transverse flexural cracks (structural) due to negative bending will most likely appear over the piers of continuous superstructures.

- Transverse reflective cracks (non-structural) may appear at closely spaced, regular intervals on plank decks with asphalt overlays. These are caused by differential plank deflections.

- Transverse reflective cracks (non-structural) may appear adjacent to an expansion joint. These cracks suggest that the joint anchorage hardware is beginning to fail.

- Longitudinal flexural cracks (structural). These are caused by negative bending of the deck over the girders or beams.

- Longitudinal reflective cracks (non-structural) may appear along the joints of prestressed concrete channels or box beams placed tight, side by side. This cracking is caused by differential beam deflection.

- Longitudinal construction joints in overlays (non-structural).

Maintenance inspection of concrete decks with asphaltic concrete overlays should include the following items:

- Examining the top of the deck, looking for potholes, and impending potholes. Impending potholes usually show up as radial cracks, localized map cracks, or depressions in the overlay.

- Looking for overlay repairs in the form of potholes filled with asphalt.

- Checking to see if potholes expose the concrete deck. If so, the deck should be examined to determine its condition and any problems should be reported.

- Looking for overlay cracks, which may allow water to reach the structural concrete deck below. Many times, cracks exist at the panel joints. The inspector should also look for transverse or D-cracks adjacent to expansion joints, as this situation indicates anchorage loosening or failure of the expansion joint armor. Older overlays may contain random or map cracking. These cracks may have been routed and filled with tar, blocking water movement to the deck.

Cracking not appearing to be temperature or shrinkage cracking is a significant cause of concern for any prestressed elements. Prestressing concrete is intended to keep the element in compression even under legal loading. A crack larger than hairline may be an indication of

overloading. Inspectors should take care to detect and note any structural cracking in prestressed elements.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/ Spall/ Patched Area (1080)
- Exposed Prestressing (1100)
- Cracking (Prestressed concrete) (1110)
- Abrasion/Wear (PSC/RC) (1190)
- Chloride Concentration (8905)
- Precast Concrete Connections (8906)

Condition State Commentary

The Condition States for describe repaired areas. Repaired areas are considered "distressed" if the repair is a short-term maintenance action that does not restore the deck's structural integrity (such as filling potholes with asphalt). Therefore these repairs do not improve the Condition State rating of an element. Repaired areas are considered "rehabilitations" if the repair (such as full or partial depth concrete patches) improves the deck's structural integrity, thereby improving the rating after work has been completed.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.3.2 Prestressed Concrete Top Flange (Element 15)

There are several prestressed concrete girders whose top flanges act as the deck of the structure. In these structures the top flange is composite with the girder and acts as the compression flange of the girder as well as the wearing surface for traffic. These structures include:

- Tee beams
- Through girders
- Channel beams
- Prestressed box beams
- Box girders
- Segmental box girders

Please refer to Chapter 4 Superstructures, for additional information concerning these bridges. The element discussed in this section deals only with the top flange portion of these superstructures where traffic rides directly on the structural element. The element is assessed regardless of the wearing surface if one is present.

Element Level Inspection

This element shall be used for all prestressed concrete bridge girder top flanges where traffic rides directly on the structural element regardless of the wearing surface or protection systems used. Top flange elements shall be used to evaluate only the edge and bottom surfaces of the respective element. Top Flange elements are considered deck elements. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects.

Wearing Surfaces (including overlays) are separate elements from reinforced concrete top flanges. Thin overlays are defined as being less than 1-inch in thickness, and may be materials such as Portland cement, epoxies or other polymers. It is important for the inspector to keep in mind that overlays do not increase the structural capacity of the deck. In fact, overlays slightly reduce the structural capacity due to the increased dead load (overburden).

The inspector should always review the *Construction History* on the backside of the Element Level Inspection form. The *Construction History* should list any overlays that were added as well as the year the overlay was constructed. Likewise, if an inspector realizes that an overlay was added to a bridge but the *Construction History* does not reflect the change, the inspector shall make note of the overlay in the *Construction History*, update the overburden data, and check the rerate flag.

On the inspection report form, prestressed concrete top flange is recorded in units of “square feet”. The correct method for calculating the flange area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks), and accounting for any flares or ramps present. The overhangs (not monolithic with the top flange) located under parapets and sidewalks are not included in the flange width measurement. This quantity is for the flange only, all girder components (web, bottom flange, etc.) should be quantified under the appropriate superstructure element. Please refer to Chapter 4 of Part 2 for Superstructure Elements.

The top flange evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

By today’s engineering practices, concrete decks and slabs in Wisconsin are constructed with epoxy coated reinforcing to protect the reinforcing steel nearest the riding surface from deicing chemicals. However, in the past not all bridges were constructed with epoxy coated bars. The inspection report form will note the type of coating on deck and slab reinforcing. The inspector will not be responsible for reporting this element, although the inspector will be able to add the element if he/she is aware of coated bars within the deck or slab element. Refer to the Concrete Reinforcing Steel Protective Systems section in Chapter 6 Part 2 for more information.

When the inspector is evaluating the bottom of the flange, it is important to assess the amount of deterioration as it is a valuable window used to assess the structural condition of the interior and top of the flange element. The amount of deterioration on the underside is a good indication of how much moisture is passing through the element.

If stay-in-place forms (bottom) prevent the inspector from viewing the bottom surfaces, those surfaces shall be assessed and evaluated based on the available visible surface. If both top and bottom surfaces are not visible, the condition shall be assessed based on destructive and nondestructive testing or indicators in the materials covering the surfaces.

Problems on the element that the inspector should look for include:

- Transverse flexural cracks adjacent to and over piers. These are caused by superstructure negative bending.
- Diagonal or transverse temperature/shrinkage cracks.
- Longitudinal flexural cracks caused by deck positive bending between the girders or stringers. Wide cracks may indicate a serious structural overload. Note the maximum crack widths measured on the bridge inspection report.
- Diagonal or transverse temperature/shrinkage cracks.
- Efflorescence, discoloration, and rust stains which will often accompany cracks.

- Delaminations, spalls, and exposed/corroded reinforcing steel. Deteriorated concrete in the flexural zones may cause debonding of the reinforcing steel.

- Signs of corroding reinforcing steel such as rust stains or exposed reinforcement. Since section loss associated with reinforcing steel corrosion can reduce the deck's strength, measure this loss if possible and record it.

- Loose or spalling concrete located over a traveled way such as a roadway, train track, river, or pedestrian path. These can fall and cause damage or injury. Such loose concrete should be reported and recommended for removal by a maintenance crew if it cannot be safely done during the inspection.

- Cracks, delaminations, spalls, and exposed reinforcing steel located under drains or leaking expansion joints.

- Punching shear failures. Although they are rare, punching failures will show up as a localized area of radial-shaped cracks with the deck bulged downward.

On the report, the inspector should describe cracks in terms of their location, orientation, length, and maximum width. Indicate also if the cracks are structural or nonstructural.

Temperature and shrinkage cracks are common and will be found on most every concrete surface. However, many types of cracks are more indicative of a structural issue. All types of cracking must be accounted for and a portion of the total quantity of area assigned to a Condition State. Look on the top and underside of the flange for any cracking. Types of cracks commonly encountered include the following:

- Temperature and shrinkage cracks (non-structural). These will be apparent on most concrete decks and overlays. These types of cracks almost always show up at the acute corners of skewed decks.

- Transverse flexural cracks (structural) due to negative bending will most likely appear over the piers of continuous superstructures.

- Transverse reflective cracks (non-structural) may appear at closely spaced, regular intervals on plank decks with asphalt overlays. These are caused by differential plank deflections.

- Transverse reflective cracks (non-structural) may appear adjacent to an expansion joint. These cracks suggest that the joint anchorage hardware is beginning to fail.

- Longitudinal flexural cracks (structural). These are caused by negative bending of the deck over the girders or beams.

- Longitudinal reflective cracks (non-structural) may appear along the joints of prestressed concrete channels or box beams placed tight, side by side. This cracking is caused by differential beam deflection.

- Longitudinal construction joints in overlays (non-structural).

Maintenance inspection of concrete decks with asphaltic concrete overlays should include the following items:

- Examining the top of the deck, looking for potholes, and impending potholes. Impending potholes usually show up as radial cracks, localized map cracks, or depressions in the overlay.

- Looking for overlay repairs in the form of potholes filled with asphalt.

- Checking to see if potholes expose the concrete deck. If so, the deck should be examined to determine its condition and any problems should be reported.

- Looking for overlay cracks, which may allow water to reach the structural concrete deck below. Many times, cracks exist at the panel joints. The inspector should also look for transverse or D-cracks adjacent to expansion joints, as this situation indicates anchorage loosening or failure of the expansion joint armor. Older overlays may contain random or map cracking. These cracks may have been routed and filled with tar, blocking water movement to the deck.

Figure 2.3.3.2-1: Localized Spall with Exposed/Corroding Rebar on the Deck Underside of a Post-Tensioned Box Girder.

Cracking not appearing to be temperature or shrinkage cracking is a significant cause of concern for any prestressed elements. Prestressing concrete is intended to keep the element in compression even under legal loading. A crack larger than hairline may be an indication of overloading. Inspectors should take care to detect and note any structural cracking in prestressed elements.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects.

However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/ Spall/ Patched Area (1080)
- Exposed Prestressing (1100)
- Cracking (Prestressed concrete) (1110)
- Abrasion/Wear (PSC/RC) (1190)
- Chloride Concentration (8905)
- Precast Concrete Connections (8906)

Condition State Commentary

The Condition States describe repaired areas. Repaired areas are considered “distressed” if the repair is a short-term maintenance action that does not restore the deck’s structural integrity (such as filling potholes with asphalt). Therefore these repairs do not improve the Condition State rating of an element. Repaired areas are considered “rehabilitations” if the repair (such as full or partial depth concrete patches) improves the deck’s structural integrity, thereby improving the rating after work has been completed.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.4 Steel Elements

Steel decks are sometimes used on bridges because of their much lighter self weight versus concrete decks. They are often used on movable bridges to reduce the required counterweight needed to balance the span. Steel decks have also been used to replace concrete decks on older bridges when an increased live load capacity is desired, or when existing superstructure or substructure elements do not have enough strength to support the heavier dead load of a concrete deck replacement.

There are several types of steel decks:

1. **Grid decks:** These are the most common types of steel decks found in Wisconsin. Conventional steel grids are distinctive for the humming sound they make when traffic drives over them. Grid decks contain several components that are either welded or riveted together. These components include bearing bars, cross bars, and supplementary bars. Openings between these bars may be filled with concrete to improve a steel grid deck's durability.

2. **Corrugated steel flooring:** This deck type uses corrugated steel plates transversely spanning between the girders or stringers. Corrugating steel plate vastly increases its strength and stiffness. The corrugations are smaller than stay-in-place forms, however the plating is thicker, ranging from 0.1 inch to 0.18 inch. After the flooring is fastened to the superstructure, the corrugations are filled with asphalt, concrete or gravel. Reinforcing is not found in this deck type. A problem with this type of deck system is that the flooring can trap and hold water that passes through the topping, making it very susceptible to corrosion. The corrosion often cannot be seen during an inspection because the plate surface experiencing the deterioration is hidden from view. Corrugated steel flooring is not normally used in Wisconsin.

3. **Buckle plate deck:** This is an obsolete style of deck once used on girder/floor beam/stringer floor systems. Buckle plates are dished downward in the middle and fastened on all four sides to the floor system. Reinforced concrete is then placed on top. Drain holes are normally provided in the middle of each plate.

4. **Orthotropic deck:** These decks are an advanced patented deck system often used on long span bridges for their light weight. Orthotropic decks act integrally with the superstructure primary members, reducing the total bridge dead load. An orthotropic deck consists of a flat steel plate with longitudinal stiffeners welded to the plate underside. The floor beams act to stiffen the deck in the transverse direction.

5. **Exodermic Deck:** These decks are a newer type of steel decking system in which a reinforced concrete slab is placed in a steel grid which is tied to a system of inverted steel tee beams. The concrete deck is composite with the lower steel beams through the steel grid. The concrete portion of the deck is designed to take the compressive forces while the steel takes the tensile forces. The concrete also protects the steel grid from environmental factors. These decks are lighter weight than reinforced concrete decks and can be precast to facilitate accelerated bridge construction.

2.3.4.1 Steel Deck – Open Grid (Element 28)

This element defines all open grid steel bridge decks with no fill. Open grid steel decks have high strengths. They are also light in weight because concrete is not used to fill the openings of the grid. Open grids are prefabricated using rectangular bars and delivered to the bridge site in several panels, which are then connected to the superstructure. The tops of the bars may be serrated to provide a skid-resistant riding surface.

Welded grid steel decks normally consist of bearing bars (also called primary or main bars). These are the main load carrying elements of a grid system. They span perpendicular between the stringers or girders and are fastened by welding. Cross bars (also called secondary or distribution bars) run parallel to the bridge. They are placed perpendicular to and on top of the bearing bars. Supplementary bars (also called tertiary bars) may or may not be found on a welded grid steel deck. They run parallel to the bearing bars, but are placed at the same level as the cross bars.

Several types of riveted grid steel decks may be encountered. Some use of bearing bars, cross bars, supplementary bars, and diagonal bars that span between the cross/supplementary bar intersections. Another type consists of main bars and crimp bars. Crimp bars are actually a length of narrow corrugated steel plate placed between the main bars. One edge of crimp bar is positioned at the same level as the top of the main bars. The crimp bar “zigzags” between the main bars, and is riveted at the point where the two bars make contact.

In recent years, the industry has begun to move away with welding and riveting fasteners. Instead the slotted hole connections are used. The holes are cut into the bearing bars and the distribution bars are placed through, rotated and locked into place by a key system.

Although they offer the advantage of a very light deck, open grid decks are constantly exposed to the elements. Even though they are often galvanized (and may even be painted), traffic wear quickly exposes the deck top surface. This leaves the deck vulnerable to corrosion. Open grids also leave the superstructure exposed to roadway debris, rain, and deicing chemicals. This can especially be a problem for movable bearings, which can lock up from pack rust and debris accumulation.

Figure 2.3.4.1-1: Open Grid Steel Deck.

Element Level Inspection

This element is for all open grid steel bridge decks with no fill.

On the inspection report form, Steel Deck – Open Grid is recorded in units of "square feet". The correct method for calculating the deck area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks), and accounting for any flares or ramps present. Overhangs located under parapets and sidewalks not comprised of open grid are not included in the deck measurement.

The steel deck – open grid evaluation is three dimensional in nature with the defects observed on the top surface, bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

For most circumstances, the entire deck will be assigned to only one element. Sometimes the inspector will encounter a deck with two or more configurations. This may be found on movable bridges, as it is not uncommon for a movable span to have open grid, filled grid, and concrete deck on various portions of the span.

In the unique situations when the steel grid has concrete in the wheel lines only, the area of filled deck should be coded under Element 29 – Steel Deck with Concrete Filled Grid. Multiple deck elements may be required to be entered on the inspection form, with the square footage distributed appropriately among the elements.

Maintenance inspection of open grid steel decks should include the following items:

- Examining the bearing bars in the bearing areas at stringers/girders for cracked welds or broken fasteners. Special attention should be paid at the tension areas of the bars.
- Looking for twisted, cracked, broken, or missing bars.
- Checking for corrosion and related section loss. A caliper is useful to measure the remaining section.
- Looking for worn serrations or excessive wear causing section loss. Worn serrations are very slippery, especially in wet weather.
- Listening for any rattling as traffic passes over the deck. Rattling suggests loose, broken, or missing fasteners.
- Looking for broken fasteners on riveted steel grid decks.
- Checking any repair plates placed over the grid to make sure they are still securely fastened. Repair plates tend not to be very durable.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Condition State Commentary

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.4.2 Steel Deck with Concrete Filled Grid (Element 29)

This element defines steel bridge decks with concrete fill either in all of the openings or within the wheel tracks. Concrete filled grid steel decks are similar to open grid steel decks in that a steel grid serves as the deck's structural component. However, the addition of concrete between the bar openings offers much better corrosion protection and a more durable riding surface. The deck system also protects the superstructure below from rain, deicing chemicals, and roadway debris. The main disadvantage is than concrete filled grid steel decks are heavier than open grid decks, although they are still lighter than traditional concrete decks. The concrete fill of these elements may be placed flush with the top layer of bars or is preferably overfilled 1 to 2 inches. The bottom of the fill may be flush with the bottom of the grid or at mid-depth of the main bars (half-filled).

It should be noted that some steel grids contain a bituminous concrete fill. Since the function of bituminous fill on steel grid decks is the same as when conventional concrete is used, bituminous filled steel decks should be inspected and rated in a manner similar as that given in this Section.

Element Level Inspection

This element defines all steel bridge decks with concrete fill either in all of the openings or within the wheel tracks. The steel deck element shall be used to evaluate only the edge and bottom surfaces of the respective element. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). In the case of a concrete filled steel grid, the top surface with concrete would be evaluated using the Wearing Surface – Bare element and the undersurface and sides would be evaluated using the Steel Deck with Concrete Filled Grid element. Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects.

On the inspection report form, Steel Deck with Concrete Filled Grid is recorded in units of "square feet". The correct method for calculating the deck area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks), and accounting for any flares or ramps present. Overhangs located under parapets and sidewalks not comprised of steel grid are not included in the deck measurement.

The Steel Deck with Concrete Filled Grid evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

For most circumstances, the entire deck will be assigned to only one element. Sometimes the inspector will encounter a deck with two or more configurations. This may be found on

movable bridges, as it is not uncommon for a movable span to have open grid, filled grid, and concrete deck on various portions of the span.

In the unique situations when the steel grid has concrete in the wheel lines only, the area of filled deck should be coded under Element 29. Multiple deck elements may be required to be entered on the inspection form, with the square footage distributed appropriately among the elements.

Maintenance inspection of concrete filled grid steel decks should include the following items:

- Checking for grid expansion at the joints and bridge ends. This is often caused by corrosion. The deck panels may bow upward.
- Looking for concrete filler that is cracked, broken out or missing all together.
- Examining the bearing bars in the bearing areas at stringers/girders for cracked welds or broken fasteners. Special attention should be paid at the tension areas of the bars.
- Looking for cracked, broken, or missing bars.
- Checking for corrosion and related section loss. A caliper is useful to measure the remaining section.
- Listening for any rattling as traffic passes over the deck. Rattling suggests loose, broken, or missing fasteners.
- Checking any repair plates placed over the grid to make sure it is still securely fastened. Repair plates tend not to be very durable.

On the report, the inspector should describe cracks in terms of their location, orientation, length, and maximum width. Indicate also if the cracks are structural or nonstructural.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Condition State Commentary

The top surface of the deck should be evaluated under the appropriate wearing surface element. This includes those steel decks that are intended to have a concrete deck. For instance, a concrete filled steel deck grid would be evaluated using the element Wearing Surface – Bare (Element 8000) for the top surface evaluation and Steel Deck with Concrete Filled Grid for the evaluation of the edges and undersurface of the deck.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.4.3 Steel Deck Corrugated/Orthotropic/Etc. (Element 30)

Orthotropic steel decks differ from corrugated steel decks in that the ribs of an orthotropic deck are fastened to the underside of a flat thin steel plate closing off the ribs. In a corrugated deck the corrugations are pressed into a steel plate creating troughs and crests. The deck is then fastened to the superstructure and filled in with gravel, asphalt or other fill material for a driving surface.

Exodermic decks would fall under this element. While an exodermic deck makes use of a steel grid to tie the concrete deck to the structural inverted tee beams, it is a much more robust configuration. Furthermore, the inverted tee beams that bare on the superstructure components are the primary steel decking components. An exodermic deck relies on the grid configuration of its bearing, distribution and tertiary bars to transfer the cast-in-place concrete deck that partially envelops the distribution bars. The CIP deck also contains a single mat of reinforcing bars to further integrate the lower grid deck to the concrete deck. The lower steel portion of the deck is intended to take all tensile loading in bending while the concrete takes only compressive loading.

Element Level Inspection

This element defines all steel bridge decks constructed of corrugated metal filled with Portland cement, asphaltic concrete, or other riding surfaces. The steel deck element shall be used to evaluate only the edge and bottom surfaces of the respective element. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). In the case of a corrugated, orthotropic or other steel deck, the top surface with concrete would be evaluated using the Wearing Surface – Bare element and the undersurface and sides would be evaluated using the Steel Deck Corrugated/Orthotropic/Etc. element. Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects.

On the inspection report form, Steel Deck Corrugated/Orthotropic/Etc. is recorded in units of “square feet”. The correct method for calculating the deck area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks), and accounting for any flares or ramps present. Overhangs located under parapets and sidewalks not comprised of steel deck are not included in the deck measurement.

The Steel Deck Corrugated/Orthotropic/Etc. evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

For most circumstances, the entire deck will be assigned to only one element. Sometimes the inspector will encounter a deck with two or more configurations. This may be found on

movable bridges, as it is not uncommon for a movable span to have open grid, filled grid, and concrete deck on various portions of the span.

Maintenance inspection of corrugated/orthotropic/etc. steel decks should include the following items:

- Observing the driving surface. Heavy cracking or areas of high density potholes may be an indication of debonding of the overlay, pack rust build up between overlay and plate or cracking in the steel plating.
- Examining the ribs and corrugations in the bearing areas at stringers/girders for cracked welds or broken fasteners. Special attention should be paid at the tension areas of the decking.
- Looking for cracked, broken, or missing ribs or corrugations.
- Checking for corrosion and related section loss. A caliper is useful to measure the remaining section if a hole is present within a rib. A D-meter (ultrasonic testing device) is also useful for detecting remaining thickness.
- Listening for any rattling as traffic passes over the deck. Rattling suggests loose, broken, or missing fasteners.
- Checking any repair plates. The inspector should note the location and connection detail to the deck.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Condition State Commentary

The top surface of the deck should be evaluated under the appropriate wearing surface element. This includes those steel decks that are intended to have a concrete deck. For instance, a concrete filled steel deck grid would be evaluated using the element Wearing

Surface – Bare (Element 8000) for the top surface evaluation and Steel Deck with Concrete Filled Grid for the evaluation of the edges and undersurface of the deck.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.5 Timber Elements

Timber decks are normally used for timber superstructures, although they are occasionally found on older steel superstructures. Timber decks may also be referred to as decking or timber flooring. Timber decking is typically considered non-composite due to the inefficient transfer of forces through to the superstructure elements. There are several types of timber decks:

1. **Plank deck:** These are the most common types of timber decks found in Wisconsin. Deck planks are simply sawn timber planks laid flat across the tops of the timber beams or steel stringers. They span transversely between the beams/stringers and are fastened to the superstructure with nails or bolt clamps. Common plank size is 3 to 6 inches thick and 10 to 12 inches wide.

Figure 2.3.5-1: Timber Plank Deck (Top Surface Evaluated as Wearing Surface – Bare).

2. **Nail laminated deck:** This deck type uses sawn planks laid on edge across the tops of the timber beams or steel stringers, creating a very stiff deck. Each plank is placed tight against and nailed to the adjacent one. When used in conjunction with timber superstructures, each plank is toe nailed to the beam. When used in conjunction with steel superstructures, the deck is attached with clamps at regular intervals. Common laminations size is 2 inches thick and 4 to 12 inches wide.

3. **Glued laminated deck:** These decks are similar to nail laminated decks, but the planks are glued together in a factory and shipped to the job site in 3- to 5-foot wide planks. After setting the planks on top of the superstructure, the planks are sometimes clamped together using dowels. The deck is then fastened down to the beams/stringers using nails, bolts, clip angles, or nailers. Glued laminated decks are stronger, stiffer, and more water-resistant than conventional deck types. Because of this, glued laminated decks are used most often for timber decks in new designs.

4. **Stressed laminated deck:** These decks use laminated timbers similar to nail and glued laminated decks. They are different in that external prestressing (post-tensioning) is used to clamp the laminations together by use of high strength steel bars that run the full width of the deck. The individual laminations work together as a unit due to the large frictional forces generated by the prestressing. Normally, the

steel rods passing through the laminations to deliver the prestressing forces are at approximately 2-foot centers.

5. **Structural composite lumber deck:** These decks include laminated veneer lumber (LVL) and parallel strand lumber (PSL). LVL is fabricated by adhering thin sheets of rotary peeled veneer together. PSL is fabricated by compressing and adhering narrow strips of veneer together with the wood grain parallel. These decks are comprised of fully composite tee beams or box beams. The superstructure components are pulled together by use of steel rods or prestressing strands that pass through the top flange (deck).

Because of timber's low resistance to abrasion, wearing surfaces are often used. These surfaces may consist of timber or steel running boards or asphalt. Running boards are placed longitudinal to traffic, normally only along the wheel paths. They are easily replaced once worn. Asphalt wearing surfaces may be placed on any deck type, although the asphalt tends to crack and deteriorate quicker on plank decks due to plank flexibility and differential deflection.

Timber slab bridges are constructed using either glue-laminated or nail-laminated sawn lumber placed longitudinally between supports. The slab acts as a single wide beam spanning from substructure unit to substructure unit. There are no individual beams with this type of bridge, so the slab also acts as the deck.

Slabs are used for simple spans of about 35 feet or less and for continuous spans of slightly greater lengths. Glue-laminated slab depths range from 6-3/4 inches to 14-1/4 inches thick, using individual strips of dimensional lumber 3/4 to 2 inches thick to form 42-inch to 54-inch wide panels. Nail-laminated slab depths range from 8 inches to 16 inches deep, using 2-inch to 4-inch dimensional lumber.

For maximum strength and stiffness, the lumber width is oriented vertically in the completed structure. Timber slabs may have transverse distributor beams attached to their undersides as a method to distribute live loads across the entire bridge width. Steel transverse post-tensioning rods (Element 8165) may also be used for this purpose, as well as to keep the planks in alignment on glue-laminated slabs.

Figure 2.3.5-2: Steel Running Boards on a Timber Deck.

2.3.5.1 Timber Deck (Element 31)
Timber Slab (Element 54)

Decks differ from slabs in that a deck lies atop superstructure components, i.e. girders or stringers. Slabs are both the deck and superstructure component and span from substructure unit to substructure unit.

Element Level Inspection

These elements define all timber bridge decks and slabs regardless of the wearing surface or protective systems used. Deck and slab elements shall be used to evaluate only the side and bottom surfaces of the respective element. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). Timber planks used along running wheel lines shall be included under the wearing surface Element 510 – Wearing Surface (Other). Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects.

On the inspection report form, timber decks, and slabs are recorded in units of “square feet”. The deck or slab evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, and being captured using the defined condition states. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

Transverse distributor beams, often fastened to a timber slab’s undersides, distribute live loads across the entire bridge width. Steel transverse post-tensioning rods may also be used for this purpose. These elements shall not be considered as part of the slab. Transverse distributor beams should be evaluated as Element 8166 – Timber Spreader Beams (refer to Chapter 4). Post-tensioning rods should be evaluated as Element 8165 – Tension Rods/Post-Tensioned Cables (refer to Chapter 4).

It is important to note that the deck element is only concerned for the top flange of structural composite lumber decks. The remaining portion of the superstructure should be coded as necessary to account for the bottom flange and web of each segment.

Element Level Inspection of timber decks and slabs should include the following items:

- Examining the element’s surface for signs of wear and abrasion, weathering, splitting, crushing, and decay.
- Examining the deck’s outside edges for signs of decay.
- Looking for loose or missing planks.

- Checking the underside of the deck at the superstructure bearing areas. Signs of deterioration in these locations include decay and crushing. Crushing is usually the result of decay. Overloads, however, may also cause crushing of sound wood.

- Looking for fastener deficiencies at the superstructure bearing areas. These deficiencies may include corrosion, looseness, or missing fasteners.

- Checking the underside of the deck in tension areas for excessive deflections, fractures, and transverse cracks. These are signs of excessive flexural stresses and overloads.

- Hammer tapping random and suspect areas to evaluate the wood's soundness. Areas of decay and deterioration are usually noted with a hollow sound.

- Perform probe tests in areas suspected to be experiencing decay. This test entails using an awl, ice pick, or pocketknife, to lift a small sliver of wood from the surface. Wood that lifts up and splinters is sound, while wood that breaks up upon lifting the tool is decaying.

- Drilling or boring suspect planks to estimate the extent of decay.

- Checking for overlay (if present) cracking or deterioration. This can be an indication of excessive flexure and a measure of the amount of moisture and chlorides that are seeping to the timber deck below.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shakes/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Condition StateCommentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the underside of a slab is struck by vehicular traffic and exhibits section loss due to the loss of material. The defect would be

reported under the appropriate defect for section loss with the note indicating the section loss was caused by traffic impact.

The Element Level Inspection Condition States for this element are concerned with mechanical and biological damage. The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013. The following are commentary for each Condition State to aid the inspector.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.3.5.1-1: Timber Deck – Condition State 1.

Figure 2.3.5.1-3: Timber Deck (Railroad Ties) – Condition State 3.

2.3.6 Other Material Elements

Other material decks or slabs are not commonly used in Wisconsin bridge construction. This section shall be used when inspecting a deck or slab element that is comprised of a material other than concrete, steel or timber. Fiber reinforced polymer (FRP) decks or panels would be an example of an Other Deck or Other Slab, respectively.

1. **Fiber reinforced polymer (FRP) deck:** These are decks comprised of a modern bridge material. FRP has been used as a repair and retrofit material in the past. These decks are fabricated using pultruded sections (e.g. honeycomb or trapezoidal shaped I-beams). There are three types of FRP decks: Honeycomb sandwich, Solid core sandwich, and Hollow core sandwich. FRP decks require an overlay due to the low slip resistance of the material. The benefit of these decks is a much lighter structure and ease of installation.

2.3.6.1 Other Material Deck (Element 60)
Other Material Slab (Element 65)

These elements define all other bridge decks and slabs composed of other construction materials regardless of the wearing surface or protective system used.

Element Level Inspection

The other deck or slab elements shall be used to evaluate only the edge and bottom surfaces of the respective element. Per WisDOT policy, the top surface of all deck and slab elements (excluding the Steel Deck – Open Grid element) will be evaluated using an associated wearing surface element. Those deck, slab or top flange elements without an overlay or other type of wearing surface will have the top surface evaluated under element Wearing Surface – Bare (Element 8000). Refer to Chapter 6 of Part 2 for more information on Wearing Surface elements and their associated material defects.

On the inspection report form, Other Material Deck and Other Material Slab are recorded in units of “square feet”. The correct method for calculating the deck area is to multiply the structure length by the edge to edge width including any median areas, sidewalk areas (including cantilever sidewalks), and accounting for any flares or ramps present. Overhangs located under parapets and sidewalks are included in the deck measurement.

The Other Material Deck or Other Material Slab evaluation is three dimensional in nature with the defects observed on the bottom surface and edges, including exposed bottom surfaces of sidewalks and medians, and being captured using the defined defects. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of other material decks or slabs should include the following items:

- Examining the overlay for signs of wear and abrasion, cracks, potholes, and impending potholes. This can be an indication that the deck structure itself is deflecting excessively or damaged.
- Examining the deck's outside edges for signs of deterioration.
- Looking for loose or missing segments.
- Checking the underside of the deck at the superstructure bearing areas. Signs of deterioration in these locations include cracking, blistering, fiber exposure, or wrinkling.
- Looking for fastener deficiencies at the superstructure bearing areas. These deficiencies may include corrosion, looseness, or missing fasteners.
- Checking the underside of the deck in tension areas for excessive deflections and cracks. These are signs of excessive flexural stresses and overloads.
- Checking for general deterioration such as discoloration, weather, debonding, etc.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Area (1080)
- Deterioration (1220)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the underside of a slab is struck by vehicular traffic and exhibits section loss due to the loss of material. The defect would be reported under the appropriate defect for section loss with the note indicating the section loss was caused by traffic impact.

The Element Level Inspection Condition States for this element are concerned with mechanical and biological damage. The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection, First Edition 2013. The following are commentary for each Condition State to aid the inspector.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.3.7 Deck National Bridge Inventory (NBI) Condition Ratings

In addition to element level inspection and Condition States, part of every Routine Inspection is rating the deck according to the FHWA General Condition Rating Guidelines. The numeric condition ratings of these guidelines describe existing bridge components in comparison to their as-built condition. Ratings range from 9 to 0, with 9 describing components in excellent condition and 0 describing failed components.

Because only a single number is used to rate the deck, the rating must characterize its overall general condition. The rating should not be used to describe local areas of deterioration, such as an isolated spall or area of heavy leaching. However, widespread spalling or heavy leaching would certainly influence the rating. A proper rating will therefore consider deterioration severity plus the extent to which it is distributed throughout the deck. Median and sidewalk condition are **not** to be considered in the NBI rating for deck condition rating, except insofar as they impact the condition of the deck/slab itself.

NBI ratings are used to evaluate the state of deterioration of the deck material. Material condition is independent of a bridge's design load-carrying capacity. Therefore, postings or original design capacities less than current legal loads will not influence the rating. Similarly, temporary deck support does not change or improve the condition of the deck material. Temporary strengthening methods will therefore not influence the deck rating. Finally, since bituminous overlays are nonstructural in nature, they do not have an influence on the deck rating.

Decks that are built integral with the superstructure (concrete and steel box girders, decks of reinforced concrete tee beam bridges, etc.) are rated as separate components from the superstructure. In other words, since the superstructure is not part of the deck, the superstructure NBI rating should not influence the deck NBI rating.

On slab bridges, the deck is the same structural component as the superstructure. The FHWA Guidelines specifically state that ratings of decks built integral with superstructures (including slabs) should not be influenced by the superstructure rating. However, since the deck NBI rating accounts for inspection findings both on the top and underside, NBI condition ratings of the deck and superstructure must be the same.

The NBI general condition ratings found in the FHWA guidelines apply to decks, superstructures, and substructures. Ratings 9 through 6 apply to components built of any material, while ratings 5 through 0 mention specific defects or deterioration that can be applied to certain materials. Because the NBI general condition ratings apply to a wide range of components and materials, Wisconsin has developed supplemental rating guidelines. These supplemental rating guidelines are used to assist the inspector in properly assigning condition ratings to specific components constructed of the most commonly used materials.

The general condition ratings and Wisconsin supplemental rating guidelines for decks are given below, followed by a summary chart for concrete decks:

Code (Rating) **Description**

N **NOT APPLICABLE**

Wisconsin Supplemental Rating Guidelines:

Used for culverts or filled spandrel arch bridges.

9 **EXCELLENT CONDITION**

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – there are no spalls, delaminations, cracks or scaling present. The electric potential is 0.0 volts, and there is no chloride within the deck.

Steel Grid Deck – there are no noticeable or noteworthy deficiencies which affect the deck condition.

Timber Deck – there are no noticeable or noteworthy deficiencies which affect the deck condition.

Other Deck – the deck is in excellent condition, typically due to new construction. After initial or first routine inspection, most decks/slabs should be rated an 8 due to exposure.

8 **VERY GOOD CONDITION –** no problems noted.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – there are no spalls, delaminations, cracks or scaling present. Minor transverse cracks may be present. There are no areas on the deck with an electric potential greater than 0.35 volts. The chloride content within the deck is less than 2.0 pounds per cubic yard.

Steel Grid Deck – there is no damage to the primary or secondary bars other than surface rust on unpainted decks. The paint system on coated decks is sound. The grid deck is securely fastened to the floor system, and any concrete filler present is sound.

Timber Deck – there is no crushing, rotting, or splitting. The deck is tightly secured to the floor system.

Other Deck – there are no significant problems noted.

7 **GOOD CONDITION –** some minor problems.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – there are no spalls or delaminations present. Cracks which can be sealed with tar or epoxy exist, or light scaling may be present. Up to 10% of the deck area has an electric potential greater than 0.35 volts. Up to 10% of the deck area has a chloride content greater than 2.0 pounds per cubic yard.

Steel Grid Deck – there may be minor damage to the primary or secondary bars, such as small twists or bends. There may be surface rust on unpainted decks, or minor isolated areas of corrosion of painted decks. Any concrete filler present is sound.

Timber Deck – there is minor checking or splitting with a few loose planks.

Other Deck – there is minor surface damage in the form of hairline cracking in resin and scratches. No delaminations are present.

6 **SATISFACTORY CONDITION –** structural elements show some minor deterioration.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – less than 2% of the deck area is spalled or less than 5% is delaminated. Excessive open cracking exists (5-foot maximum spacing) or moderate scaling may be present. Ten to 20% of the deck area has electric potential greater than 0.35 volts. Ten to 20% of the deck area has a chloride content greater than 2.0 pounds per cubic yard.

Steel Grid Deck – there may be some twisted, bent, or cracked bars. There may be some isolated broken welds or loose/broken fasteners. Concrete filler may have broken out at a few localized areas. There is surface rust on unpainted decks, and surface or freckle rust of painted decks.

Timber Deck – some planks are checked or split, but they are sound. There may be some loose or moderately worn planks. Any fire damage is limited to surface charring with minor, measurable section loss. Some areas of wetness are present.

Other Deck – there is some measureable damage over less than 2% of the total area. FRP's showing cracks in resin, scratches, blistering, abrasion, and small delaminations. Fibers at damaged areas are not exposed or buckled. Delaminations are smaller than 4 inches in every dimension and located away from structurally significant details (i.e. tension flange at midspan)

5 **FAIR CONDITION –** all primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – 2% to 5% of the deck area is spalled, or 5% to 20% is delaminated. Excessive open cracks or heavy scaling may exist. Twenty to 40% of the deck area has an electric potential greater than 0.35 volts. Twenty to 40% of the deck area has a chloride content greater than 2.0 pounds per cubic yard.

Steel Grid Deck – there are some twisted, bent, or cracked bars and possibly a few broken or missing bars. There are some broken welds or loose/broken fasteners. Concrete filler may have broken out at a few scattered locations. Some section loss may be occurring due to corrosion, but the section loss is not very measurable. Section loss due to wear may be noticed in the wheel lines.

Timber Deck – numerous (30-40%) of the planks are checked, split, rotted or crushed. Many planks are loose. Fire damage is limited to surface charring with minor, measurable section loss. Some planks (<10%) are in need of replacement.

Other Deck – there is measureable deterioration and/or section loss over 2-10% of the total area. In FRPs, damage showing cracks in resin, scratches, blistering, abrasion, and small delaminations. Fibers in cracks are exposed but not debonded, buckled, or ruptured. Delaminations are less than 8 inches in all dimensions and are located away from structurally significant details.

4 **POOR CONDITION –** advanced section loss, deterioration, spalling, or scour.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – greater than 20% of the deck area is delaminated. Full-depth failures are present or imminent. Excessive leaching exists. Over 60 percent of the deck area has an electric potential greater than 0.35 volts. Over 60 percent of the deck area has a chloride content greater than 2.0 pounds per cubic yard.

Steel Grid Deck – there are numerous cracked, broken, or missing bars. There are numerous broken welds or loose/broken fasteners. Concrete filler has broken out at scattered locations. Measurable surface pitting/section loss is occurring due to corrosion, reducing the grid deck's load carrying capacity. The paint system of coated decks has failed. Measurable section loss due to wear has occurred in the wheel lines.

Timber Deck – the majority (over 40%) of the planks are rotted, crushed, or split. Fire damage exists that has significant section loss, possibly reducing the load carrying capacity of the member. Over 10% of the planks are in need of replacement.

Other Deck – there is significant surface damage over 10-25% of the total area. In FRPs, damage is cracks in resin, scratches, blistering abrasion, and delamination. Fibers in cracks are exposed but not debonded, buckled or ruptured. Delaminations are less than 8 inches in all dimensions but are located near structurally significant details.

3 **SERIOUS CONDITION –** loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – many full-depth failures are present or imminent. Excessive leaching exists.

Steel Grid Deck – there are numerous broken or missing bars. There are widespread broken welds or broken fasteners. Several areas of concrete filler are missing. Section loss has reduced the grid deck's load carrying capacity. Measurable section loss due to wear has occurred in the wheel lines.

Timber Deck – severe signs of structural distress are visible. Extensive plank damage is evident with a reduced deck load carrying capacity.

Other Deck – there is significant surface damage over greater than 25% of the total area warranting structural analysis to determine load capacity. In FRP's damage is deep cracking in resin, scratches, blistering, abrasion, and delamination. Fibers are exposed but not debonded, buckled, or ruptured at damaged locations. Delaminations are smaller than 14 inches in every dimension.

2 **CRITICAL CONDITION –** advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – full-depth failures exist over much of the deck. The deck structural capacity is grossly inadequate.

Steel Grid Deck – there are widespread broken or missing bars accompanied with partial deck failures. There are widespread broken welds or broken fasteners. Much of the concrete filler is missing. It may be necessary to close the bridge until corrective action is taken.

Timber Deck – advanced deterioration with partial deck failure. It may be necessary to close the bridge until corrective action is taken.

Other Deck – there is significant surface damage over the majority of the deck and warranting structural analysis to determine load capacity. If not closely monitored, bridge should be posted for reduced loads or closed. In FRPs, Fibers are exposed, debonded and ruptured or buckled at damaged locations. Delaminations are larger than 24 inches in any dimension.

1 **"IMMINENT" FAILURE CONDITION –** major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structural stability. Bridge is closed to traffic but corrective action may put it back in light service.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – bridge is closed. Corrective action may put it back in light service.

Steel Grid Deck – bridge is closed. Corrective action may put it back in light service.

Timber Deck – bridge is closed. Corrective action may put it back in light service.

Other Deck – bridge is closed. Corrective action may put it back in light service.

0 **FAILED CONDITION –** out of service, beyond corrective action.

Wisconsin Supplemental Rating Guidelines:

Concrete Deck – the bridge is closed. Deck replacement is necessary.

Steel Grid Deck – the bridge is closed. Deck replacement is necessary.

Timber Deck – the bridge is closed. Deck replacement is necessary.

Other Deck - the bridge is closed. Deck replacement is necessary.

DECK RATING	CRACKING	SCALING	SPALLING	DELAM.	ELECTRICAL POTENTIAL (VOLTS)	CHLORIDE CONTENT (LB/CY)
9	None	None	None	None	0	0
8	Minor Transverse	None	None	None	None >0.35	None >2.0
7	Sealable	Light	None but visible tire wear	None	10% >0.35	10% >2.0
6	Excessive (open cracks @ 5-foot maximum spacing)	Moderate	<2%	<5%	10-20% >0.35	10%-20% >2.0
5	Excessive	Heavy	2%-5%	5%-20%	20-40%* >0.35	20%-40%* >2.0
4	Many full depth failures present or imminent; leaching			>20%	Over 60%* >0.35	Over 60%* >2.0
3	Many full depth failures present or imminent; leaching					
2	Full depth failures over much of deck					
1	Bridge Closed. Corrective action may put back in service					
0	Bridge Closed. Replacement necessary					

Note: Values are guidelines only

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2.4 SUPERSTRUCTURE

2.4.1 Introduction

The term “superstructure” is defined as all bridge elements above the substructure units. However, for bridge inspection purposes, “superstructure” refers to all bridge elements, other than the deck, that distribute loads longitudinally to the substructure units. The exception to this definition is the concrete slab, where the deck and superstructure are one and the same.

Members comprising the superstructure are categorized into two groups: primary and secondary. Primary superstructure members are those that directly carry the deck dead loads and live loads to the abutments and piers. The superstructure members include girders, beams, floor beams, stringers, arches, trusses, cables, bearings, and stiffeners. Secondary superstructure members are those that do not directly carry deck dead loads and live loads. The secondary superstructure members are used to provide lateral stability for the primary members and help laterally distribute the live loads so that the individual primary members act together as a unit. Secondary members include diaphragms, cross-frames, struts, and lateral bracing.

Superstructures come in a variety of types and configurations and comprised of concrete, steel, timber, a combination of materials, or another material entirely. Retrofits or repairs may see use of a differing structural material than the existing. This may be due to availability, cost, geometric or installation restrictions. For instance, a fractured timber stringer may be retrofitted with a C-channel bolted to its side or a rolled beam may replace the stringer altogether.

It is necessary for the inspector to understand the components of the superstructure and their function within the load path. The bridge inspector must understand the critical areas of each type of superstructure as well as the defects associated with the structural material the component is comprised of.

This chapter is broken down by construction material (concrete, steel, timber, etc.) and superstructure type (girder, truss, arch, etc.). Each material section lists the elements in numerical order (as they are labeled in the AASHTO Manual for Bridge Element Inspection) and describes the element in detail. Each element segment of the chapter details the element level inspection and safety inspection to further guide the inspector.

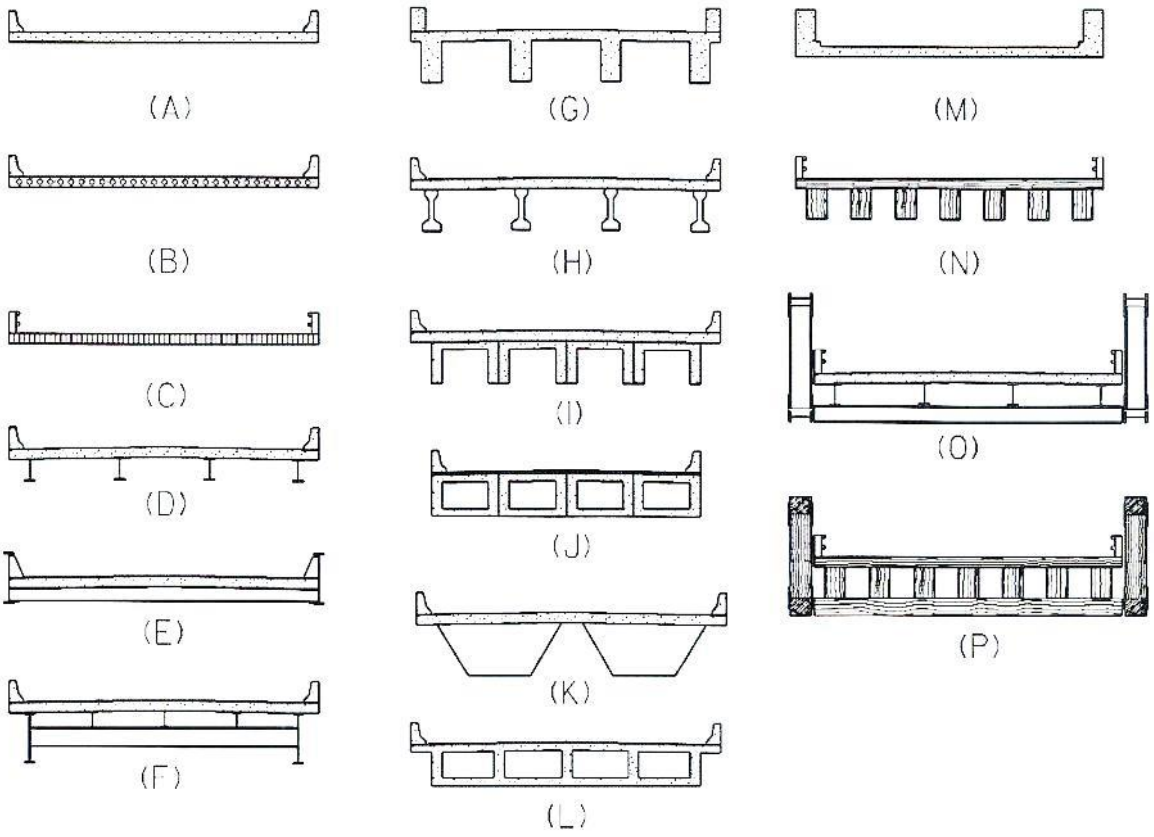

Figure 2.4.1-1: Common Superstructure Types – (A) Reinforced Concrete Flat Slab; (B) Reinforced Concrete Voided Slab; (C) Timber Slab; (D) Steel Multi-beam; (E) Steel Through Girder; (F) Steel Girder/Floor Beam/Stringer; (G) Reinforced Concrete Tee Beam; (H) Prestressed Concrete I Beam; (I) Prestressed Concrete Channel; (J) Prestressed Concrete Box Beam; (K) Steel Box Girder; (L) Post-tensioned Concrete Box Girder; (M) Reinforced Concrete Through Girder; (N) Timber Multi-beam; (O) Steel Truss; (P) Timber Truss.

Primary superstructure members must carry repetitive live loads, as well as repeatedly applied impact loads. Depending on the type of superstructure, the members may need to deliver these loads to the substructure by way of bending, tension, compression or a combination of these. To handle this type of demand, it is critical that the members be sound, as failure of one, in the worst case, could be catastrophic. This chapter provides guidelines for the bridge inspector on which parts of the superstructure are critical to inspect and what defects may cause future problems.

2.4.2 Reinforced Concrete Structures

Concrete has been used to construct bridges in the United States since 1889. With the exception of arches, conventional reinforced concrete was initially limited to short single span use. The development of prestressed concrete in the middle part of the 1900s, with the subsequent development of post-tensioned concrete boxes, allowed concrete to gain acceptance for use on medium and long span bridges. Concrete can be configured in many different ways, including:

1. **Cast-in-place slabs:** Cast-in-place slab bridges are the simplest type of concrete bridge. The slab acts as a single, wide beam spanning from substructure unit to substructure unit. There are no individual beams with this type of bridge, and the slab also acts as the deck. Slabs are used for simple spans of about 45 feet or less. Continuous slab bridges can be built with slightly longer span lengths. To attain greater negative bending strength on continuous bridges, the slab may be thickened (haunched) over the piers. The main reinforcing steel is placed parallel to traffic and located towards the bottom of the slab in positive bending regions, and towards the top of the slab in negative bending regions. On older and more complex structures, continuous cast-in-place slabs may contain voids to lighten the dead load of the bridge. Refer to Chapter 3 for more information on slab structures.

Figure 2.4.2-1: Single Span Flat Slab Bridge.

2. **Tee beams:** Tee beam bridges were commonly constructed in the early half of the 20th century. They are cast-in-place structures, with the deck cast monolithic with the beams. The “tee” shape is created by the rectangular beam stem below the deck, with the deck forming the top flange. In Wisconsin, the fascia beams on many tee beam bridges are upturned, doubling as parapets. Tee beam bridges are most commonly used for simple spans, although they may be made continuous by adding a haunch the beam stems over the piers. Individual spans may reach 50 feet in length, with the beams spaced from about 3 to 8 feet. Common beam depths range from 18 to 24 inches. The main reinforcing steel is placed longitudinally towards the bottom of the beam in positive bending regions and longitudinally within the deck in

negative bending regions. Transverse, vertical stirrups placed along the beams serve as shear reinforcing.

Figure 2.4.3-2: Reinforced Concrete Tee Beam Bridge.

3. **Through girders:** Through girder bridges were constructed in the early half of the 20th century. They are cast-in-place structures, with the deck cast monolithic with the girders. Two girders normally are used, and these very deep girders serve as the primary superstructure, as well as bridge parapets. WisDOT's policy is to code these members using element 110 - Reinforced Concrete Open Girder and their condition should be reflected in the condition rating for Superstructure only. The deck spans between the girders, connected to the lower portion of the girders. Through girder bridges are used for simple spans of 30 to 60 feet. Because the deck must span between the girders, through girder bridge widths rarely exceed 24 feet. The girders themselves are fairly large, being 18 to 30 inches wide and 4 to 6 feet deep. The main reinforcing steel is placed longitudinally towards the bottom of the girders, while the main deck reinforcing steel is placed transversely towards the deck bottom. Transverse, vertical stirrups placed along the girders serve as shear reinforcing.

Figure 2.4.3-3: Reinforced Concrete Through Girder Bridge.

4. **Channel beams:** Channel beam bridges use precast channel beams as the primary load-carrying members. The channels are placed on the substructure units so that they form an upside down “U”, with the vertical legs forming the beams and the horizontal top slab forming the deck. The channels are placed tight side by side and transversely connected so that the individual beams act as a unit under live loads. Grouted shear keys also help the beams to act together. Channel beam bridges are used for simple spans up to about 50 feet. Widths of the individual beams usually range from 3 to 4 feet. The main reinforcing steel is placed longitudinally towards the bottom of the channel legs, while the main deck reinforcing steel is placed transversely in the top slab. Vertical stirrups may be placed along the channel legs and serve as shear reinforcing.

Figure 2.4.3-4: Deteriorated Prestressed Channel Beams.

5. **Open spandrel arches:** Open spandrel arch bridges use either cast-in-place arch ribs or a single arch ring as the primary load-carrying members. The arches resist a combination of axial compression and bending moments. The deck and floor system are placed above the arches, and spandrel columns and caps (bents) deliver these loads to the arch. The space between the deck and arch (the spandrel) is left open, hence the name “open spandrel arch”. Since the arch acts primarily as a compression member, longitudinal steel is uniformly distributed around its perimeter, contained by transverse ties. The spandrel bent columns are reinforced in a similar manner. Spandrel bent caps act as fixed end beams, so reinforcing steel is placed near the bottom between the columns and near the top above the columns. Transverse, vertical stirrups placed along the cap serve as shear reinforcing. The deck and floor system loading the spandrel arches are designed and reinforced similar to other reinforced concrete beams.

Figure 2.4.3-5: Open Spandrel Reinforced Concrete Arch.

6. **Closed spandrel arches:** Closed spandrel ("Earth Filled") arch bridges use a single cast-in-place arch ring or barrel, as the primary load-carrying member, with the arch resisting a combination of axial compression and bending moments. The spandrel area is closed by solid walls built above the barrel edges, hence the name "closed spandrel arch". The deck/roadway is always placed above the arches, and the spandrel area may be filled or vaulted. In filled spandrels, the roadway pavement bears on fill material that occupies the spandrel area. This fill is contained by solid spandrel walls built above the barrel edges. Main reinforcing steel for solid spandrel walls retaining fill is placed at the back or fill side of the wall and cannot be inspected. In vaulted spandrels, the structural deck and floor system load the arch by way of transverse spandrel walls or spandrel bents, while the spandrel walls are nonstructural. The spandrel bents, deck, and floor system are reinforced similar to open spandrel arches. Arch barrels are reinforced with longitudinal steel uniformly distributed around its perimeter, contained by transverse ties. The top side of the barrel cannot be inspected, unless access is provided in vaulted closed spandrel arch bridges.

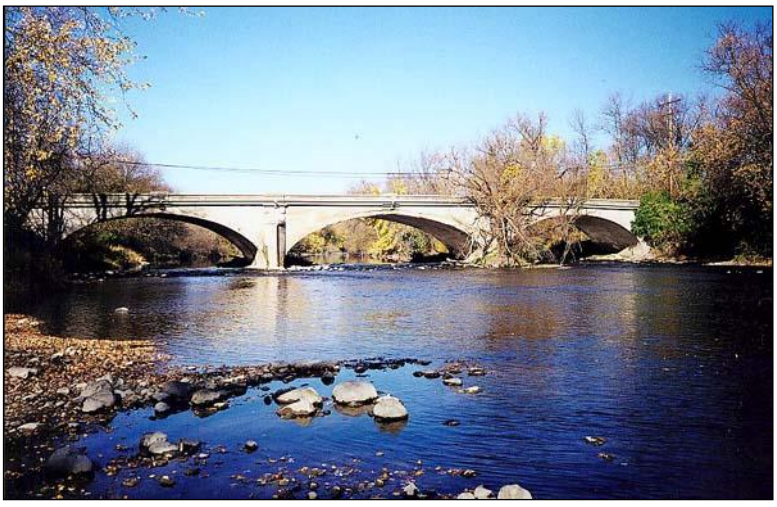

Figure 2.4.3-6: Closed Spandrel Concrete Arches.

7. **Rigid frames:** Rigid frame bridges are structures in which the vertical or inclined supporting “legs” are cast monolithically with the girders to form a rigid frame. These bridges are usually single span structures constructed to form an inverted channel, usually of a slab design. Multiple span bridges may also be constructed by forming a rectangular shape, a “K” shape or a triangular delta shape. Though the legs are used as bridge piers, the vertical or inclined legs are actually part of the superstructure because of their rigid connection to the horizontal slab or girders. This rigid intersection of the leg and horizontal member is referred to as the knee and allows both members to resist bending moments. Main reinforcing steel in the horizontal members is placed longitudinally near the bottom of the slab or girder between the abutments and legs. At the knees, it is placed longitudinally near the top on continuous bridges and around the outside or the corner on single span bridges. Main reinforcing steel is placed vertically on both frame leg faces on continuous bridges and only on the traffic face of single span bridges. Transverse, vertical stirrups placed along the horizontal member of beam frames serve as shear reinforcing, while transverse ties are placed along the legs. Spans of 50 to 200 feet are attainable using rigid frames.

Figure 2.4.3- 7: Single Span Concrete Rigid Frame

2.4.2.1 Reinforced Concrete Slab (Element 38)

These elements represent the simplest concrete superstructures and have been in use for many years. Concrete slab elements are currently the most economical method to span short distances. Slab elements are most commonly constructed using cast-in-place methods, although plant fabricated precast/prestressed or post-tensioned hollow core planks have also been used to create slab bridges. Precast slabs are different than precast box beams in that precast slabs contain two or more voids through them, while box beams have only one.

Refer to Part 2 – Chapter 3 Decks and Slabs for description and element level inspection methods for reinforced and prestressed concrete slabs.

Elements in this section are mostly field cast, though some may be non-prestressed plant cast members. All are strengthened with conventional mild reinforcing steel.

2.4.2.2 Reinforced Concrete Closed Web/Box Girder (Element 105)
Reinforced Concrete Open Girder (Element 110)
Reinforced Concrete Stringer (Element 116)
Reinforced Concrete Floor Beam (Element 155)

These are primary bending elements of conventionally reinforced concrete bridges. Closed web/box girders and open girders are longitudinal main members spanning between substructure units. They may sometimes be referred to as beams rather than girders. Stringers are small longitudinal members that span between the floor beams. Floor beams, in turn, span transversely between the main longitudinal girders.

These bending elements are normally rectangular or “tee”-shaped members. On tee-shaped beams, the top flange also functions as the bridge deck. The cross-section of a reinforced concrete closed web/box girder will normally contain several cells, rather than forming a single box shape. These bridge types may be thought of as a series of “I”-shaped girders lined up side-by-side. As with tee beams, the top flanges of box girders function as the bridge deck.

Figure 2.4.2.2-1: Inside of a Reinforced Concrete Box Girder.

Since each of these elements are conventionally reinforced bending members, the inspector should expect to find transverse flexural cracks on the top or bottom surfaces in the high moment areas. The main reinforcing steel for concrete bending members is placed longitudinally near the tension surface. Diagonal shear cracks on the sides of these elements may also be found at the abutments and piers. To provide shear strength, vertically oriented reinforcing steel ties are placed along the length of these elements.

Element Level Inspection

These elements define all reinforced concrete beam elements regardless of the wearing surface or protective systems used.

On the inspection report form, reinforced concrete girders, stringers, and floor beams are recorded in units of lineal feet. The correct method for calculating the girder/beam length is multiplying the number of beams in each span times the span length of each span. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

For box girders or other beams where traffic drives directly on the top flange, regardless of wearing surface, the top flange, above the fillet, shall be assessed as element 16 Reinforced Concrete Top Flange. The remaining portion of the beam shall be evaluated under the appropriate superstructure element. Refer to Chapter 3 of Part 2 for more information on deck and top flange elements and their associated material defects.

Element Level Inspection of reinforced concrete closed web/box girders, open girders, stringers, and floor beams should include the following items:

- Inspecting the member for cracks. The inspector should look for transverse flexural cracks on the underside of the beam between supports and on top of the deck over the piers on continuously supported bridges. Cracks wider than hairline in the flexural region of beams may indicate a serious structural overload.

- Checking for deteriorated concrete in the flexural zones that is causing debonding of the reinforcing steel. This is especially critical near the ends of the reinforcing steel bars, since a certain length of the bar must be embedded within sound concrete to fully develop its strength. The deterioration causing the debonding may be delaminations, spalls or longitudinal cracks.

- Examining the support areas for shear cracks. Shear cracks will be diagonal, extending up from the bearing towards mid-span. Wide shear cracks suggest the loss of aggregate interlock, meaning the member could by hanging from the reinforcing stirrups. Maximum crack widths should be measured and noted on the bridge inspection report.

- Checking the entire member for signs of corroding reinforcing steel, as indicated by rust stains or exposed reinforcement. Since section loss associated with reinforcing steel corrosion can reduce a member's strength, measure this loss if possible and record it on the inspection report.

- Looking for leaching, and noting if it is stained with rust since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Investigating the bearing areas for spalled concrete due to friction from thermal movement or crushed concrete due to bearing pressure overloads.
- Checking the member under drains or leaking expansion joints for cracks, delaminations, spalls, and exposed reinforcing steel.
- Checking previously repaired areas for soundness by hammer tapping.
- Looking for collision damage, typically found above or adjacent to traffic lanes or navigation channels.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Delamination/Spalls/Patched Areas (1080)
- Exposed Rebar (1090)
- Cracking (RC and Other) (1130)
- Abrasion/Wear (1190)
- Precast Concrete Connections (8906)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the underside of a slab is struck by vehicular traffic and exhibits section loss due to the loss of material. The defect would be reported under the appropriate defect for section loss with the note indicating the section loss was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow

- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.2.2-2: Reinforced Concrete Tee Beams – Condition State 1.

Figure 2.4.2.2-3: Reinforced Concrete Tee Beam – Condition State 2.

Figure 2.4.2.2-4: Corner of Reinforced Concrete Box Girder – Condition State 3.

Figure 2.4.2.2-5: Reinforced Concrete Tee Beam – Condition State 4.

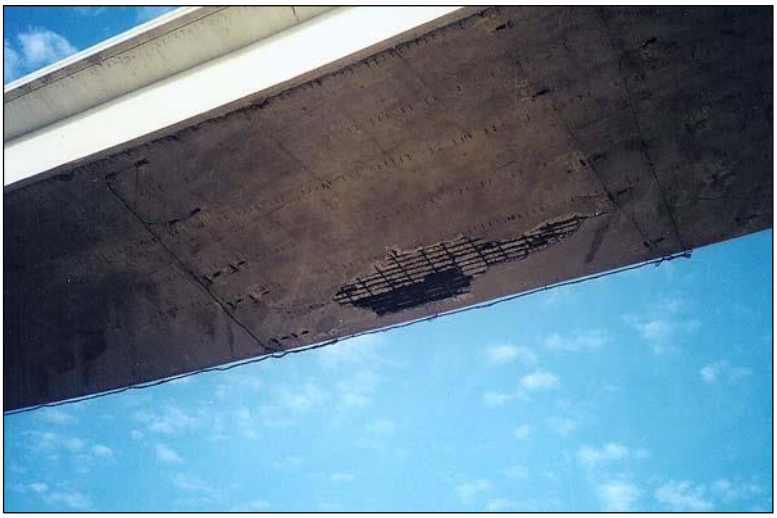

Figure 2.4.2.2-6: Reinforced Concrete Box Girder Bottom Flange Through Thickness Section Loss - Condition State 4.

Figure 2.4.2.2-7: Reinforced Concrete Box Girder Bottom Flange Through Thickness Section Loss - Condition State 4.

2.4.2.3 Reinforced Concrete Arch (Element 144)

Arches are primary elements that receive both compressive and bending moments. By far the most common arch cross-sectional shape is rectangular. Since an arch carries a high degree of compressive load, there should be little if any net tension on any of the surfaces due to bending moments. The main reinforcing steel is distributed uniformly around the perimeter.

Open spandrel arches will have two or more individual arch ribs loaded by vertical spandrel columns. Closed spandrel arches have a single, solid barrel forming the primary load-carrying member. If the closed spandrel is filled with backfill material to support the roadway, then the spandrel walls are also considered to be primary members. This is because they are functioning as bending members to retain the outward pressure caused by the fill. Failure of a spandrel wall would cause the fill to spill out, resulting in roadway settlement. For Element Level Inspection purposes, structural spandrel bent caps, columns, and walls shall be considered part of the arch element. Another important point to note is that filled, closed spandrel arches will always have Assessment 9325 – Roadway over Structure. This assessment is discussed in Part 2 – Chapter 6.

Figure 2.4.2.3-1: Reinforced Concrete Arch Bridge.

Element Level Inspection

This element is for only mild steel reinforced concrete (typical) arches regardless of protective system.

On the inspection report form, reinforced concrete arches are recorded in units of lineal feet. The correct method for calculating the arch length is the sum of all of the lengths of each arch panel measured longitudinally along the travel way. For filled arches, the arch quantity shall be measured from spring line to spring line. The components below the spring line are considered substructures. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of

the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

For filled spandrel reinforced concrete arches where the fascia extends above the roadway over structure, the area below the ground line to the arch shall be evaluated as the arch element. The area above the ground line shall be evaluated using the appropriate bridge railing element.

Element Level Inspection of reinforced concrete arches should include the following items:

- Examining the bearing areas for signs of concrete crushing, since the highest compressive forces experienced by an arch are found at the spring line. Crushing results in a loss of arch cross-sectional area, increasing the axial stresses.
- Looking for longitudinal cracking along the arch axis. Longitudinal cracks indicate a structural overload.
- Checking the entire arch and spandrel wall for delaminations, spalls, and exposed reinforcing steel. These defects reduce these members' cross-sectional area, resulting in higher stresses.
- Checking the entire arch for transverse cracks. These are the result of excessive bending moments or arch support settlements.
- Looking for leaching and rust stains along the entire arch and spandrel wall. These defects can grow into larger problems such as delaminations and spalls.
- Checking areas exposed to drainage and roadway runoff. The runoff may cause scaling, spalling, and concrete contamination.
- Checking to make sure weep holes in closed spandrel arch structures are functioning.
- Checking to make sure surface drains in closed spandrel arch structures are functioning properly, and not allowing water to penetrate the fill.
- Examining previous repair areas for soundness by hammer tapping.

Spandrel Components

All other components that transfer loading to the arch ribs, except the floor system (girders, floor beams and stringers) shall be evaluated under Reinforced Concrete Arch (Element 144). These components include spandrel bent caps, spandrel columns, spandrel walls, etc.

Spandrel bent caps support the decking floor system and transfer the loads to vertical spandrel columns or walls. These components are primary load-carrying members that load the arch ribs. They may also be used on vaulted closed spandrel arches.

Spandrel bent caps are primary load-carrying bending members. They may be bending members on spandrel walls, if their ends cantilever over the ends of the wall, but they are most often used as architectural features in this situation. They often must carry large girder

reaction loads and many times work in conjunction with the spandrel columns to form a frame for resisting lateral loads.

Element Level Inspection of spandrel caps should include the following items:

- Examination for vertical flexural cracks, either on the underside between columns or top side above columns or shafts. Cracks wider than hairline in a flexural region may indicate a serious structural overload.

- Checking for deteriorated concrete in the flexural zones that is causing debonding of the reinforcing steel. This is especially critical near the ends of the reinforcing steel bars, since a certain length of the bar must be embedded within sound concrete to fully develop its strength. The deterioration may be delaminations, spalls, and longitudinal cracks.

- Looking for shear cracks over and near the supports. Shear cracks will be diagonal, extending up from the column towards mid-span. Wide shear cracks suggest the loss of aggregate interlock, meaning the member could be hanging from the reinforcing stirrups. Maximum crack widths should be measured and noted on the bridge inspection report.

- Checking the entire member for signs of corroding reinforcing steel, as indicated by rust stains or exposed reinforcement. Since section loss associated with reinforcing steel corrosion can reduce a member's strength, measure this loss if possible and record it on the inspection report.

- Looking for leaching and noting if it is stained with rust, since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Checking the member under drains or leaking expansion joints for cracks, delaminations, spalls, and exposed reinforcing steel.

- Checking previously repaired areas for soundness by hammer tapping.

- Examining the top surface (bearing seat) of caps for cracking and spalling. The pedestals and grout pads under the bearings should also be checked for cracking, spalls, and deterioration that reduce the bearing area. Deterioration in these areas may be caused by the lack of reinforcing steel in older bridges, frozen expansion bearings that transmit lateral forces to the cap not intended for in the original design, and salt-laden water leaking through expansion joints.

- Looking for the presence of debris or standing water on the bearing seat. Debris suggests a failed/leaky expansion joint. Standing water indicates that the bearing seat is dished. Salt-laden standing water will eventually migrate to the reinforcing steel, causing corrosion, delaminations and spalls.

Spandrel columns are primarily compression members, but they must also resist lateral bending moments due to wind loads, eccentric loading at their tops, overloads, and

differential arch deflections. Note that these components are evaluated under the arch element regardless if they are comprised of prestressed concrete or not.

Element Level Inspection of reinforced concrete spandrel columns found on arches should include the following items:

- Checking the arch/spandrel column interface for transverse flexural cracks. These cracks may extend up several feet above the arch rib. They are an indication of excessive column bending due to overloads or differential arch deflection.

- Checking the spandrel bent cap/spandrel column interface for horizontal or diagonal flexural cracks. These cracks will originate at the inside corner of the cap/column junction and are another sign of excessive bent bending due to overloads or differential arch deflection.

- Checking the mid-height of the column for flexural cracks, as this is another sign of structural overloads or differential arch deflection.

- Examining the entire column for longitudinal cracks and crushed concrete. This would be the result of a serious structural overload.

- Checking the entire column for delaminations, spalls, and exposed reinforcing steel. These defects reduce these members' cross-sectional area, resulting in higher stresses. Record this information on the inspection report.

- Looking for leaching, and noting if it is stained with rust since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Checking previously repaired areas for soundness by hammer tapping.

On the inspection report form, the arch components other than the ribs shall be evaluated under the arch element as the component's projected length along the arch length. For example, a spandrel column 40 feet tall and 1 foot wide (along the length of the bridge) exhibits 2 inch deep spalling throughout its height. A quantity of 1 foot would be placed in Condition State 3 of Defect 1080 – Delamination/Spalls/Patched Areas under the arch element. It is the inspector's task to examine each component and reasonably assign the appropriate Condition State to the arch element. This will quantify the component's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.4.2.3-2: Open Spandrel Reinforced Concrete Arch – Cracking Condition State 2.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Delamination/ Spall/ Patched Area (1080)
- Exposed Rebar (1090)
- Cracking (RC and Other) (1130)
- Abrasion/Wear (1190)
- Precast Concrete Connections (8906)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the underside of a slab is struck by vehicular traffic and exhibits section loss due to the loss of material. The defect would be reported under the appropriate defect for section loss with the note indicating the section loss was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the

severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.3 Prestressed Concrete Structures

Prestressed concrete superstructures have been gaining popularity since the 1950s, with the subsequent development of post-tensioned concrete boxes, allowing concrete to gain acceptance for use on medium and long span bridges. Prestressed concrete superstructures can be configured in many different ways, including:

1. **Precast prestressed voided slab:** Precast voided slabs are bridges similar to cast-in-place slabs in that each slab acts as a single, wide beam spanning from substructure unit to substructure unit. As with cast-in-place slabs, the precast slab also acts as the deck. Precast voided slabs, however, are manufactured in a plant and pretensioned. These precast slabs contain circular or elliptical voids for reducing material and weight. Each slab or plank is usually 3 or 4 feet wide, with depths ranging from about 15 to 26 inches. Each slab is placed tight side by side and transversely clamped together so that the individual planks act as a unit under live loads. Grouted shear keys also help the beams to act together. Precast voided slabs are used for simple spans of about 70 feet. The main steel prestressing strands are placed parallel to traffic, and located towards the bottom of the slab. Precast voided slabs will generally have two or three voids. While contrary in name, these superstructure elements would be evaluated as closed web/box girders along with a top flange element.

2. **Prestressed box beams:** Precast box beams are bridges similar to precast voided slabs. However, precast box beams contain only a single void. In early applications, the top flange of the box beam also acted as the deck. Current practice is to place an asphalt overlay or separate concrete overlay or deck on top of the beams. Concrete decks often act compositely with the box beams. Each beam is usually 3 or 4 feet wide, having a depth ranging from about 12 to 60 inches. Each beam may be placed side by side and transversely clamped together so that the individual beams act as a unit under live loads. Grouted shear keys also help the beams to act together. They may also be spaced 2 to 6 feet apart to form a bridge similar in appearance to a tee beam bridge. Prestressed box beams are used for simple spans from about 20 to 130 feet in length. The main steel prestressing strands are placed parallel to traffic and located in the bottom flange of the box. Conventional reinforcing steel ties are placed transversely along the beam for shear reinforcement.

3. **Prestressed I-beams and Bulb-Tees:** Prestressed I-beams and bulb-tees are commonly precast members without voids. They make for efficient use of material by concentrating the concrete away from the beam's neutral axis where it is needed most for stiffness and strength. Concrete decks are often designed to act compositely with the I-beams. They are used for simple spans up to about 160 feet in length. They may also be made continuous over piers for live loads and secondary dead loads. This is done by placing conventional reinforcing steel longitudinally in the deck over the piers to resist negative bending. The main steel prestressing strands are placed parallel to traffic and located in the bottom flange of the beam. Conventional reinforcing steel ties are placed transversely along the beam for shear reinforcement. Secondary members of prestressed I-beam bridges include concrete diaphragms at the abutments and piers and either steel or concrete diaphragms within the spans.

Figure 2.4.3-1: Prestressed I-beams.

4. **Prestressed Double Tees:** Double tee beams are commonly used in building applications such as parking garages but have since been adopted into bridge structures. As the name suggests, the component resembles two T's that are side by side. These superstructures are typically used in shorter simple spans but may be used in continuous applications. Segments are held together through the use of lateral connectors. Also a wearing surface is typically placed over the tops of the beams to create a uniform driving surface and to seal the longitudinal joints between beams. Prestressed double tee superstructures may resemble cast-in-place tee beams however the longitudinal joint seen underneath and the thinner stems of the beams indicate prestressed concrete. Double tee beams are typically constructed with 12-34 inch stems (legs) with flange width (out-out of each segment) ranging from 8 to 10 feet. The bridges typically span 25-55 feet. Primary prestressing stands are placed near the bottom of the legs. Conventional steel reinforcing is placed longitudinally as temperature and shrinkage steel. In continuous applications, ducts may be draped through the stems to allow for post-tensioning. Shear reinforcing U-shaped stirrups are placed vertically in the stems and extend up into the flange. Primary flange reinforcing is placed transversely and follows the placement of a typical concrete deck.

Figure 2.4.3-2: Double Tee Girder Bridge.

5. **Box girders:** Box girder bridges are used for very long structures and curved spans. The sections are very large, and a single box can be used to carry an entire roadway. Insides of the boxes are usually large enough for inspectors to enter. Traditional box girders are cast-in-place and may be conventionally reinforced or post-tensioned. Cast-in-place box girders will often contain several internal vertical webs and are referred to as multi-cell box girders. Concrete box girders are used for spans in excess of 150 feet. The main reinforcement of post-tensioned box girders is a combination of conventional steel reinforcement and post-tensioning tendons. This reinforcement is placed in the bottom flange between the substructure units to resist positive bending and in the top flange above the piers to resist negative bending. The post-tension tendons are normally placed within galvanized steel ducts. Conventional reinforcing steel ties are placed transversely along the beam for shear and torsion reinforcement.

Figure 2.4.3-3: Conventionally Reinforced Box Girder Bridge.

6. **Segmental box girders:** Segmental box girder bridges are similar to traditional box girders. However, the segments of segmental box girders are manufactured at a precast plant, shipped to the site, and erected individually. They commonly have a trapezoidal shape, with the top flange cantilevering over the inclined webs, and normally contain only one cell. All are post-tensioned.

Figure 2.4.3-4: Segmental Box Girder Bridge.

2.4.3.1 Prestressed Concrete Closed Web/Box Girder (Element 104)
Prestressed Concrete Open Girder (Element 109)
Prestressed Concrete Stringer (Element 115)
Prestressed Concrete Floor Beam (Element 154)

These elements include factory-manufactured prestressed open or box beams, field cast/post-tensioned box girders, post-tensioned segmental box girders, and post-tensioned segmental arches.

These are primary bending elements that are typically plant-fabricated precast/prestressed open: I-beams, bulb tees, inverted channels, closed boxes, and segmental post-tensioned closed boxes. On closed box shapes and inverted channel sections, the top flange may also function as the bridge deck. Stringers account for all prestressed members that support the deck in a floor system. Floor beams account for all prestressed floor beams that typically support stringers in a floor system.

Figure 2.4.3.1-1: Prestressed Concrete I-Beam Bridge.

Each of these elements are stressed in axial compression, therefore, the inspector should not expect to find any transverse flexural cracks. Transverse flexural cracks indicate a structural overload to the member, and these cracks should be measured and recorded.

For multi-span bridges, open prestressed girders are usually designed for live load and secondary dead load continuity over the piers. This is accomplished by placing longitudinal conventional reinforcing steel bars over the piers within the deck. A cast-in-place diaphragm is placed over the piers to fill the gap between the girder ends and to laterally lock them into position. As live or secondary dead loads are placed on the deck, negative bending is resisted by compression in the girder's bottom flange and diaphragm and by tension within the deck longitudinal continuity steel.

Element Level Inspection

On the inspection report form, prestressed concrete girders, stringers, and floor beams are recorded in units of lineal feet. The correct method for calculating the girder/beam length is multiplying the number of beams in each span times the span length of each span. For box girders and double tees, all legs or webs monolithic with the top flange are considered 1 girder. That is a 50 foot long double tee beam girder would be measured 50 feet, not be quantified as 100 feet (50 feet for each leg). Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

For box girders or other beams where traffic drives directly on the top flange, regardless of wearing surface, the top flange, above the fillet, shall be assessed as Element 15 – Prestressed Concrete Top Flange. The remaining portion of the beam shall be evaluated under the appropriate superstructure element. Refer to Chapter 3 of Part 2 for more information on deck and top flange elements and their associated material defects.

Element Level Inspection of prestressed closed web/box, open girders and stringers should include the following items:

- Looking for transverse flexural cracks on the girder underside in the positive moment regions. Their presence indicates a serious structural overload or loss of prestressing/post-tensioning force. The inspector should measure the crack widths and lengths and document their location.

- Examining the beam ends for evidence of cracked or spalled concrete, sometimes accompanied with corroded reinforcing strands. This may occur due to leaking expansion joints that corrodes the reinforcing or prestressing steel. It may also be the result of a lack of non-prestressed reinforcement in the zone of prestressing force transfer.

- Sighting down the length of the beams to check for sagging. Sagging is a sign that the beam is losing its prestressing force and is unable to carry the loads for which it was designed.

- Looking for diagonal shear cracks on the girder sides near the abutments and piers.

- Looking for vertical cracks on the girder sides near the abutments and piers. Vertical cracks in these areas suggest the bearing assemblies are restricting girder movement.

- Examining the beam underside for parallel longitudinal cracks. These usually occur along the prestressing strands and may occur due to inadequate concrete cover. Rust stains that accompany the cracks suggest that the prestressing strands are corroding and debonding.

- Checking for leakage/efflorescence between the longitudinal joint of prestressed box beams placed next to each other. This condition, along with reflective longitudinal cracking on the deck surface, suggests that the grouted shear keys between the members have failed. Once these shear keys have failed, live loads cannot be shared between adjacent beams.

- Checking for excessive or differential deflections between box or channel beams placed next to each other. This is a sign that the beam is losing its prestressing force and is unable to carry the loads for which it was designed. It also indicates failed grouted shear keys.

- Verifying that the drain holes are open on prestressed box beams.

- Looking for delaminations, spalls, and exposed reinforcing steel.

- Checking for any superelevation irregularities on curved box girder bridges. This is a sign that torsional distress has occurred.

- Documenting box girder top flange transverse flexural cracks and any leaching and rust staining that may accompany them. This will usually be performed during an in-depth inspection, as the underside of a box girder's top flange can only be seen from inside the cells.

- Signs of impact damage include scrapes on member undersides, chips, cracks, spalls, and possibly a broken out section of a member. Particularly strong collisions will expose several reinforcing bars or prestressing strands. Underside scrapes are an indication of vehicle contact with the superstructure, and are typically found above or adjacent to traffic lanes or navigation channels.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Delamination/ Spall/ Patched Area (1080)
- Exposed Prestressing (1100)
- Cracking (PSC) (1110)
- Abrasion/Wear (1190)
- Precast Concrete Connections (8906)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the underside of a slab is struck by vehicular traffic and exhibits section loss due to the loss of material. The defect would be

reported under the appropriate defect for section loss with the note indicating the section loss was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.3.1-2: Prestressed I-Beam – Condition State 1.

Figure 2.4.3.1-3: Prestressed I-Beams – Condition State 1.

Figure 2.4.3.1-4: Prestressed Channel Beams – Cracking Condition State 3.

Figure 2.4.3.1-5: End of Prestressed I-Beam with Spalls and Exposed Strands and Rebar – Delamination/Spall/Patched Area Condition State 3.

Figure 2.4.3.1-6: Prestressed Channel Beams with Exposed Strands, Longitudinal Cracks, Leaching, and Rust Staining – Cracking Condition State 4.

Figure 2.4.3.1-7: Delaminated/Spalled Prestressed Beam End with Exposed Strands – Exposed Prestressing Condition State 4.

2.4.3.2 Prestressed Concrete Arch (Element 143)

Prestressed arches are rare forms of the concrete arch. Arches are primary load-carrying elements that carry axial compressive stresses, as well as bending, tension, and compressive stresses. Normally, the axial loads are great enough on an arch that there are no net tensile stresses due to bending. When bending stresses are large enough to produce net tension, post-tensioning is used to pre-compress the arch cross-section. This keeps the entire cross-section in compression, eliminating any net tensile stress.

Figure 2.4.3.2-1: Open Spandrel Prestressed Arch Bridge with Struts.

Since prestressed arches are stressed in axial compression, the inspector should not expect to find any transverse flexural cracks. Transverse flexural cracks indicate a structural overload to the member, and these cracks should be measured and recorded.

Element Level Inspection

This element is for prestressed or post-tensioned arches regardless of protective system.

On the inspection report form, prestressed concrete arches are recorded in units of lineal feet. The correct method for calculating the arch length is the sum of all of the lengths of each arch panel measured longitudinally along the travel way. For filled arches, the arch quantity shall be measured from spring line to spring line. The components below the spring line are considered substructures. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

For filled spandrel reinforced concrete arches where the fascia extends above the roadway over structure, the area below the ground line to the arch shall be evaluated as the arch element. The area above the ground line shall be evaluated using the appropriate bridge railing element.

Element Level Inspection of prestressed arches should include the following items:

- Looking for transverse flexure cracks along the arch. Their presence indicates a serious structural overload, loss of prestressing/post-tensioning force or arch support settlements. The inspector should measure the crack widths and lengths, and document their location.

- Examining the bearing areas for signs of concrete crushing, since the highest compressive forces experienced by an arch are found at the spring line. Crushing results in a loss of arch cross-sectional area, increasing the axial stresses.

- Looking for longitudinal cracking along the arch axis. Longitudinal cracks indicate a structural overload.

- Checking the entire arch for delaminations, spalls, and exposed reinforcing steel. These defects reduce the members' cross-sectional area, resulting in higher stresses.

- Looking for leaching and rust stains along the entire arch. These defects can grow into larger problems such as delaminations and spalls.

- Checking areas exposed to drainage and roadway runoff. The runoff may cause scaling, spalling, and concrete contamination.

- Examining previous repair areas for soundness by tapping with a hammer.

Spandrel Components

All other components that transfer loading to the arch ribs, except the floor system (girders, floor beams and stringers) shall be incorporated in the Prestressed Concrete Arch (Element 143) Condition States. These include spandrel bent caps, spandrel columns, spandrel walls, etc.

Spandrel bent caps support the decking floor system and transfer the loads to vertical spandrel columns or walls. These components are primary load-carrying members that load the arch ribs. They may also be used on vaulted closed spandrel arches.

Spandrel bent caps are primary load-carrying bending members. They may be bending members on spandrel walls if their ends cantilever over the ends of the wall, but they are most often used as architectural features in this situation. They often must carry large girder reaction loads and many times work in conjunction with the spandrel columns to form a frame for resisting lateral loads.

Element Level Inspection of spandrel caps should include the following items:

- Examination for vertical flexural cracks, either on the underside between columns or top side above columns or shafts. Cracks wider than hairline in a flexural region indicate a serious structural overload.

- Checking for deteriorated concrete in the flexural zones that is causing debonding of the reinforcing steel. This is especially critical near the ends of the reinforcing steel bars, since a certain length of the bar must be embedded within sound concrete to fully develop its strength. The deterioration may be delaminations, spalls, and longitudinal cracks.

- Looking for shear cracks over and near the supports. Shear cracks will be diagonal, extending up from the column towards mid-span. Wide shear cracks suggest the loss of aggregate interlock, meaning the member could be hanging from the reinforcing stirrups. Maximum crack widths should be measured and noted on the bridge inspection report.

- Checking the entire member for signs of corroding reinforcing steel, as indicated by rust stains or exposed reinforcement. Since section loss associated with reinforcing steel corrosion can reduce a member's strength, measure this loss if possible and record it on the inspection report.

- Looking for leaching and noting if it is stained with rust, since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Checking the member under drains or leaking expansion joints for cracks, delaminations, spalls, and exposed reinforcing steel.

- Checking previously repaired areas for soundness by hammer tapping.

- Examining the top surface (bearing seat) of pier caps for cracking and spalling. The pedestals and grout pads under the bearings should also be checked for cracking, spalls, and deterioration that reduce the bearing area. Deterioration in these areas may be caused by the lack of reinforcing steel in older bridges, frozen expansion bearings that transmit lateral forces to the pier not intended for in the original design, and salt-laden water leaking through expansion joints.

- Looking for the presence of debris or standing water on the bearing seat. Debris suggests a failed/leaky expansion joint. Standing water indicates that the bearing seat is dished. Salt-laden standing water will eventually migrate to the reinforcing steel, causing corrosion, delaminations, and spalls.

Spandrel columns are primarily compression members, but they must also resist lateral bending moments due to wind loads, eccentric loading at their tops, overloads, and differential arch deflections. Note that these components are evaluated under the arch element regardless if they are comprised of prestressed concrete or not.

Element Level Inspection of reinforced concrete spandrel columns found on arches should include the following items:

- Checking the arch/spandrel column interface for transverse flexural cracks. These cracks may extend up several feet above the arch rib. They are an indication of excessive column bending due to overloads or differential arch deflection.

- Checking the spandrel bent cap/spandrel column interface for horizontal or diagonal flexural cracks. These cracks will originate at the inside corner of the cap/column junction and are another sign of excessive bent bending due to overloads or differential arch deflection.

- Checking the mid-height of the column for flexural cracks, as this is another sign of structural overloads or differential arch deflection.

- Examining the entire column for longitudinal cracks and crushed concrete. This would be the result of a serious structural overload.

- Checking the entire column for delaminations, spalls, and exposed reinforcing steel. These defects reduce these members' cross-sectional area, resulting in higher stresses.

- Looking for leaching, and noting if it is stained with rust since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Checking previously repaired areas for soundness by hammer tapping.

On the inspection report form, the arch components other than the ribs shall be assessed under the arch element as the component's projected length along the arch length. It is the inspector's task to examine each component and reasonably assign the appropriate Condition State to the arch element. This will quantify the component's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Delamination/ Spall/ Patched Area (1080)
- Exposed Prestressing (1100)
- Cracking (PSC) (1110)
- Abrasion/Wear (1190)
- Precast Concrete Connections (8906)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the underside of a slab is struck by

vehicular traffic and exhibits section loss due to the loss of material. The defect would be reported under the appropriate defect for section loss with the note indicating the section loss was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.4 Steel Structures

Since the late 1800s, steel has been one of the most commonly used materials to construct bridge superstructures. The steel can be configured in many different ways, and it is used to create virtually any bridge type, including:

1. **Rolled multi-beams:** Rolled multi-beams are bridges constructed using three or more hot-rolled steel beams as the primary members. Beam depths are usually no more than 36 inches, which limits spans to about 90 feet maximum. Transverse diaphragm secondary members, usually C-shaped channel sections, may be bolted or riveted to the beam webs. To increase a beam's bending capacity; cover plates may be riveted, bolted or welded to the underside of the bottom flange. Unfortunately, the ends of welded cover plates create a significant stress riser and a highly fatigue prone detail.

Figure 2.4.4-1: Rolled Steel Multi-beam Bridge.

2. **Fabricated multi-girders:** Fabricated multi-girders are bridges constructed using three or more built-up steel girders as the primary members. The girders are fabricated by either riveting/bolting together steel plates and angles or by welding steel plates together. Greater economy is typically achieved by varying flange thickness/width or the number of plates in a flange to accommodate the bending moment magnitude. Web depths may also be deepened (haunched) over the piers to achieve the same effect. Girder depths are usually greater than 36 inches, allowing for spans up to about 500 feet. Due to their greater web depths, transverse cross-frames using angles or T-shapes are usually used as secondary elements. In addition, vertical and/or longitudinal stiffeners may be welded or riveted to the web to prevent web buckling. Older structures may use lateral bracing placed at the same level of the bottom or top girder flanges to connect adjacent girders. Larger spans may be built with a floor system consisting of stringers and floor beams as additional primary members. Floor beams are transverse beams that frame into the girder webs. The stringers bear on or frame into floor beams. Stringers are usually rolled beams placed between and running parallel to the girders. Along with the girders, the stringers directly support the deck. Stringers and floor beams are not nonredundant steel tension members (NSTMs).

Figure 2.4.4-2: Steel Multi-Girder Bridge.

3. **Two-girder systems:** Two-girder systems are bridges constructed using only two built-up steel girders as primary members. These bridges have floor systems that use floor beams and sometimes stringers. Features of two-girder systems are also common to fabricated multi-girder systems. Two-girder systems do not have load path redundancy and therefore are classified as NSTM bridges. Floor beams in this configuration may be considered NSTM, depending on the spacing between them.

Figure 2.4.4-3: Two Girder System Bridge.

4. **Through girders:** Through girder bridges are similar to two-girder bridges but the deck is placed between the girders rather than on top of them. Many older short- to medium-span highway and railroad bridges use this configuration. These types of bridges are two-girder systems and are therefore classified as NSTM bridges.

Figure 2.4.4-4: Steel Through Girder Bridge.

5. **Box girders:** Box girder bridges have girders fabricated from plates welded into a rectangular or trapezoidal closed shape. Because closed shapes are stiffer in torsion than open "I" shaped girders, box girders are commonly used for curved spans. Closed shapes also help to better protect the steel from corrosion since only half the plate area is exposed to the elements. The vertical plates form the webs, the bottom plate forms the bottom flange, and the top plate (if present) or concrete deck forms the top flange. Both the web and flange plates are normally strengthened with transverse and longitudinal stiffeners to prevent buckling. Box girders can usually be entered through access hatches for inspection of their interiors. Bridges using one box girder do not have load path redundancy and are therefore classified as NSTM bridges. Most two box girder bridges are also classified as NSTM. However, some two girder bridges have been designed with substantial bracing between the boxes so that one of them can support the entire bridge should the other fail. Box girder bridges used in simple span applications can span up to 75 feet. When used in continuous applications, spans of over 100 feet can be attained.

Figure 2.4.4-5: Steel Box Girder Bridge.

6. **Trusses:** Truss bridges are structures with two or more parallel trusses being the main load-carrying members. The deck may be placed on top of the trusses (deck truss) or between the trusses (through truss when there is overhead lateral bracing or a pony truss when there is no overhead lateral bracing). Through or pony trusses are most often constructed using two trusses, and therefore are considered NSTM structures. Deck trusses are constructed with two or more trusses. Only two truss deck trusses are NSTM structures. Truss chord, diagonal, and vertical members may be fabricated from eyebars, rolled shapes, or built-up members. Connections are made with rivets, bolts, welds, and pin connections. All truss bridges have floor systems similar to two-girder systems. To keep the two trusses in line longitudinally, secondary lateral bracing members diagonally connect the bottom chords. For deck and through trusses, the top chords are connected the same way. Sway bracing keeps the two trusses in line laterally. On deck and through trusses, sway bracing transversely frames between the truss verticals. On through trusses, sway bracing may limit vertical clearances. On pony trusses, sway bracing is usually placed as a transverse diagonal on the outside of the trusses. It connects the top chord to transverse “outrigger” floor beam extensions and functions to prevent buckling of the top chord.

Figure 2.4.4-6: Steel Truss Bridge.

7. **Deck arches:** Deck arch bridges are structures with the deck placed on top of two or more riveted, bolted, or welded arches. The arches are the main load-carrying members, and their ends bear on foundations at grade. Their bearing ends are usually pinned and the end reactions have a vertical component due to the dead and live gravity loads, and a horizontal component due to the arch’s outward thrust. A pin may also be present at the arch crown, forming a three-hinged arch. The area between the deck and arch is known as the spandrel. Deck arches use vertical compression members, called spandrel columns, to deliver the deck loads to the arch

ribs. The spandrel columns may be rolled or built-up shapes. Each arch rib may be fabricated into an "I" or box shape (solid rib arch) or into a truss shape (braced rib arch). When diagonal braces connect the spandrel columns above the rib, the bridge is classified as a spandrel braced arch. The floor system will contain floor beams, spandrel girders, and sometimes stringers. Secondary members include the upper and lower lateral bracing which brace the floor system and arch ribs, respectively. Transverse sway bracing keeps the ribs and spandrels in line laterally. Although many deck arch bridges have only two arch ribs, the bridges are not considered NSTM since arches resist a combination of compression loads and bending moments, not tension.

Figure 2.4.4-7: Steel Deck Arch Bridge.

8. **Through arches:** Through arch bridges are structures with the deck placed below the crown of and between two riveted, bolted, or welded arches. As with deck arches, the arches are the main load-carrying members and are usually pinned, with the ends bearing on foundations at grade. A pin may also be present at the arch crown to form a three-hinged arch. Through arches use vertical tension members to suspend the deck under the arch ribs. The tension members may be steel cables, wire rope, or solid steel hangers. Each arch rib may be fabricated into a box shape (solid rib arch) or more commonly into a truss shape (braced rib arch). The floor system will contain floor beams, girders, and sometimes stringers. Secondary members include lateral bracing for the arch ribs and floor system, and transverse sway bracing to keep the ribs in line laterally. As with deck arches, most through arches are not NSTM bridges. The exception is the tied arch.

9. **Tied arches:** Tied arch bridges are special types of through arches. The ends of tied arches bear on piers, and are tied together with a tie girder. The tie girder is in tension and necessary to resist the very large horizontal thrusts of the arch rib. It functions in a similar manner to the string on an archer's bow. Because tied arch bridges have only two arches and two tie girders, tied arch bridges are considered NSTM since there are only two tie girders and they are in tension. A failure of one tie

girder will directly lead to a failure of its associated arch. The tie girder may also behave as a bending member, in conjunction with the ribs, to deliver dead and live gravity loads to the pier. Primary and secondary members of tied arches are similar to those of through arches, although the arch ribs are typically solid box shaped elements. Floor beams frame directly into the webs of the tie girders, and cables or hangers (solid or hollow) directly support the tie girder. The hangers may be oriented vertically or diagonally.

Figure 2.4.4-8: Steel Tied Arch Bridge.

10. **Rigid frames:** Rigid frame bridges are structures in which the structure's inclined supporting "legs" are integrated with the girders to form a rigid frame. Rigid frames are usually constructed using welded plate girders and legs to form a "K" shape or, in some instances, an inverted triangular delta shape. Though the legs are used as bridge piers, the legs are actually part of the superstructure because of their rigid connection to the girders. This rigid intersection of the leg and girder is referred to as the knee and allows both the girders and legs to resist bending moments. Large moments and shear forces are resisted by the knee, resulting in a complex arrangement of stiffeners in this area. The legs are pinned at grade, and the girder ends supported by conventional abutments. Rigid frame bridges may use two or more frames to support the deck. The girders, legs, and bearings are all primary members on multi-rigid frame. Secondary members are the diaphragms, cross-frames, longitudinal stiffeners, transverse stiffeners, and radial stiffeners. Spans 50 to 200 feet are attainable using rigid frames.

Figure 2.4.4-9: Steel Rigid Frame Bridge.

11. **Cable-stayed bridges:** Cable-stayed bridges are typically long span structures that use one or two planes of inclined stay cables as their main means of support. The stay's opposite ends are attached to pylons, which deliver the cable forces to the foundation. Spans from 700 -1400 feet are attainable using cable-stayed bridges.

Figure 2.4.4-10: Cable Stayed Pedestrian Bridge.

12. **Suspension bridges:** Suspension bridges are typically long span structures that support the deck from vertical suspender cables attached to two or more catenary main suspension cables. The main suspension cables are draped over towers, and their ends fixed to gravity anchors.

Figure 2.4.4-11: Steel Pedestrian Suspension Bridge.

Steel superstructures are classified as being either NSTM or non-NSTM. Refer to Section 1.4.2.4 for a discussion of NSTM bridges and inspection procedures.

2.4.4.1 Steel Closed Web/Box Girder (Element 102)
Steel Open Girder/Beam (Element 107)
Steel Stringer (Element 113)
Steel Floor Beam (Element 152)
Steel Cross Girder – Closed Web/Box Girder (8102)
Steel Cross Girder – Open Girder/Beam (8107)

These steel elements are either hot-rolled structural sections (wide flanged section, channels, angles, plates) or are built-up by riveting, bolting or welding together two or more hot-rolled structural sections.

These elements are primary load-carrying bending members. Hot-rolled beams of multi-beam bridges may be recorded as either an open girder or stringer, depending upon the inspector's interpretation of these terms. In the case where a deck is supported by a floor system, stringers shall be recorded as those beams that tie into the floor beam and girders shall be recorded as those beams that floor beams tie into.

Steel Cross Girders are oriented perpendicular to the direction of the main girders ensuring the main girders act together and share loads. The element must be located above the bearings to be a cross girder. If the element is located below the bearings, it is considered a pier cap. A cross girder is not a diaphragm or a floor beam.

Figure 2.4.4.1-1: Steel Cross Girder – Closed Web/Box Girder

Element Level Inspection

These elements refer to those beam elements regardless of protective system. The Condition States of these elements are independent of the protective coating on the element. Refer to Part 2 Chapter 6 Steel Protective Coatings for inspection methods and defects of paint and other protective coatings.

On the inspection report form, steel girders/beams are recorded in units of lineal feet. The correct method for calculating the beam length is the sum of all of the lengths of each beam section and multiplying by the span length for each span. For steel box girders, one box is considered one beam section. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

The evaluation of steel beams is three-dimensional in nature. The inspector shall evaluate all faces and determine the predominant defect for each unit of measure along the element. This includes evaluating all faces, interior and exterior, of box girders per linear foot and recording the controlling defect. If multiple defects are found within the same unit of measure, the inspector shall record the predominant defect on the inspection report and describe other defects within the notes under the element. The predominant defect is determined first by

Condition State. If Condition States are equal, the hierarchy then falls to the lowest associated defect number.

Maintenance inspection of box girders, open girders, stringers, and floor beams should include examination of the rust texture and color of the steel surface to judge if the corrosion is active or if there is the desired oxide coating. Problems are most likely to be found under leaky expansion joints, areas subjected to constant traffic spray, and areas that accumulate debris or bird waste. The inspector should look for signs of active corrosion on the steel surfaces. These signs, in order of decreasing severity, include:

1. Deep pitting of the surface;
2. Laminar rust or visible laminations on steel edges;
3. Surfaces that can be rubbed off by hand or with a wire brush;
4. Flakes 1/2 inch in diameter;
5. Flakes 1/8 inch in diameter;
6. Surfaces with a granular and flaky texture; and
7. Surfaces with a coarse texture.

For all of these situations, the remaining metal thickness should be measured with calipers or an ultrasonic thickness gauge.

Safety Inspection

During the Element Level Inspection of primary bending members, it is extremely important to remember that the entire purpose of a bridge inspection is to ensure public safety. A structural inspection must also be carried out. The main purpose of these primary members is to transmit loads to the substructure, and a structural failure of one could mean a local bridge failure, requiring that part or all of the bridge be shut down. Failure of a NSTM open girder would cause a probable collapse of the entire bridge.

Flexural Areas: Girders, beams, and stringers experience bending throughout their lengths. The largest bending moments occur around midspan on simply supported members. On continuous members, the largest positive bending moments are located around midspan, and the largest negative bending moment locations are directly over the interior supports. Since stringers are supported by floor beams, they will behave in a manner similar to continuous girders. There is no bending experienced by either simply supported or continuous members at expansion joints

Maintenance inspection in the flexural areas of steel box girders, open girders, stringers, and floor beams should include the following items:

- Examining the flexure zones and tension flanges for corrosion and loss of cross-sectional area, which is the most common steel defect. About 10 percent flange section loss or more will begin to raise the stress level an appreciable amount.

- Removing of spot areas of debris accumulation to check for corrosion. Bird waste that is often found on the flanges is acidic and traps moisture and road salts, accelerating corrosion.

- Checking rivet/bolt heads on built-up components, as corrosion on the heads may indicate corrosion along the entire fastener length, reducing structural integrity.

- Looking for pack rust, noted by individual plate bending between fasteners. Pack rust may be present between the plies of riveted/bolted connections such as field splices or secondary member connections.

- Looking for overload damage in the form of compression flange buckling and tension flange elongation or fracture in the high moment flexural regions.

- Looking for torsion related damage on curved box girders at the diaphragms/cross-frames, webs, and flanges as evidenced by plate or member distortions.

- Examining suspect fasteners for looseness by twisting by hand or tapping the heads with a hammer.

- Checking the girders for distortion or scraping caused by traffic impacts. The impact damage is usually most prominent on the fascia girders.

- Viewing down the member's length to check vertical and horizontal alignments, as well as for any canting (lateral bending or twisting). This type of damage may be due to overloads, traffic impact or support settlement.

Shear Zones: The zones of highest shear stresses are at the abutments and both sides of the piers. Most beams and girders have vertical bearing stiffeners at their supports.

Maintenance inspection in the shear areas of steel box girders, open girders, stringers, and floor beams should include the following items:

- Looking for web crippling where bearing stiffeners are not used. Web crippling is a permanent wrinkling or buckling of the web due to overloads.

- Checking for excessive web section loss due to corrosion, especially under expansion joints and integral abutments, where the steel may remain damp for extended periods. Web section loss increases shear stresses and makes the web less stiff and more susceptible to crippling.

- Checking the bearing stiffeners for excessive corrosion and any associated buckling due to overloads.

Safety Inspection - Fatigue

Primary bending members are susceptible to fatigue damage. Fatigue cracks usually show up as rust stains or rusty breaks in the paint, propagating perpendicular to the direction of stress. An excellent resource discussing fatigue related damage in bridges is the *Manual for Inspecting Bridges for Fatigue Damage Conditions*. This resource contains background

information, inspection techniques, and many photographs/diagrams illustrating suspect fatigue prone details and damage.

The following text attempts to summarize locations for fatigue prone details and what the inspector should examine on hot-rolled beams, fabricated girders, box girders, floor system components, and riveted/ bolted members. Nonetheless, it is strongly recommended that the inspector become familiar with the *Manual for Inspecting Bridges for Fatigue Damage Conditions.* Any fatigue related damage found should be recorded under defect 1010 *Cracking*.

Hot-Rolled Beams: Fatigue inspection of hot-rolled steel beams should include the following items:

- Checking for in-plane bending fatigue cracks which are most likely to develop at the ends of cover plates welded to the tension flange. This detail must be inspected carefully, since it has one of the lowest fatigue strengths of any welded detail, and a tension flange failure could have catastrophic effects.

- Looking at the diaphragm connection plates that are welded to the tension flange, as these act as stress risers. Transverse cracks may occur in the flange.

- Checking for out-of-plane bending cracks that may be found at diaphragm connections of skewed bridges. Since the diaphragms may not line up end-to-end, differential beam deflections can cause the diaphragm to push/bend the web out of plane when the connection plate is not connected to the flange. A similar condition exists at the fascia beams of non-skewed bridges. The damage typically shows up as a “U” shaped crack on the web, traveling around the diaphragm connection plate’s clipped corner at the flange. The connection plate is usually fillet welded to the web on both sides, and the crack occurs either within the weld or in the web material at the weld toe. On continuous spans, the connection plate is often not welded to the top tension flange over the piers. In this situation, fatigue cracks may develop in the shape of an upside down “U” at the connection plate’s upper clipped corner. Horizontal cracks may also occur at the toe of the flange/web fillet due to the high lateral restraint of the top flange provided by deck embedment. Though rare, out-of-plane bending may crack welds joining the connection plate to the flange.

Fabricated Girders: In-plane bending at fatigue prone details on fabricated girders is similar to those found on hot-rolled beams. In addition, fatigue inspection in the flexural areas of fabricated steel girders should include the following items:

- Investigating longitudinal stiffener butt welds located within tension or stress reversal zones. These carry the same longitudinal stresses as the girder web and may produce fatigue cracks due to low quality welds containing internal flaws or having rough surfaces.

- Investigating longitudinal stiffeners terminating in tension or stress reversal zones. These are fatigue prone details due to the sudden change in girder cross-sectional geometry.

- Investigating groove welds used to join the ends of web plates or different size flange plates in long spans. These welds may be ground flush or with the reinforcement left in place. In either case, though fatigue cracks are not normally expected to be found, poor welding or inspection may have left internal flaws within the weld metal, especially on older bridges.

- Examining lateral bracing member gusset plates groove-welded to tension flange edges or lapped and fillet welded to the tension flange. This is an outdated detail found on older bridges. These attachments should be inspected closely, as the gusset plates are details that are highly fatigue prone, similar to the ends of welded flange cover plates, and could have the potential for brittle fracture.

- Examining intersecting welds. High residual tensile stresses, internal flaws, and low fatigue strengths are created when welds intersect at attachments. Common intersecting weld locations are at longitudinal stiffener/transverse stiffener/web junctions and at horizontal gusset plate/vertical connection plate/web junctions.

- Examining intersecting welds found in haunched girders fabricated by groove welding the haunch plate to the web of a rolled shape. To fabricate this detail, the bottom flange of the rolled shape is cut off. This produces a re-entrant corner into which the haunch plate is welded. Since it is difficult to produce a quality weld at the short vertical length and at the groove weld intersection, fatigue cracks may develop at these locations. Similar details are found at insert plates used for web section loss repairs. In these repairs, square plates are fitted inside a square hole cut into the web. The plate edges are welded to the web, creating a patch. These patches contain intersecting welds and questionable groove weld terminations.

- Looking for welded repairs which increase the static strength of a member but greatly reduce the fatigue strength. These include patch plates fillet welded over heavily corroded areas (producing sudden geometric changes) and poor quality plug welds used to fill misdrilled bolt holes (plug weld cooling also creates high residual tensile stresses in the base material).

- Looking for out-of-plane bending cracks found in fabricated girders that are similar to those found on beam/diaphragm connections. These cracks occur at diaphragms, but may also occur at similarly detailed floor beam connections.

- Looking for out-of-plane bending cracks at horizontal web-gaps, which are the most common source of fatigue cracking in fabricated girders. Web gaps are created when the lateral bracing's horizontal gusset plate is notched to fit around a vertical connection plate and then welded to the girder web. Out-of-plane bending in this case is produced by forces originating from the lateral braces that push and twist the web. The connection plate and gusset greatly stiffen the web, and high bending stresses are generated in the relatively flexible gap between these two plates. The inspector should investigate the web gap on both sides of the connection plate, paying close attention to the toe of the weld along the connection plate and at the end of the horizontal gusset plate at the notch. Any crack will be oriented vertically at either of these two locations. Though not as common as at web-gaps, vertical cracks can develop at the ends of the gusset plates. Cracks that are close to, or propagated into the flange should be immediately reported to the Inspection Program Manager.

- Looking for out-of-plane bending cracks when superstructure girder flanges are welded directly to a cross girder web. Because intersecting groove or seal welds are used to create girder flange continuity through the cross girder web, a fatigue prone detail is created due to intersecting welds and weld flaws. In addition, high in-plane cross girder stresses combined with out-of-plane forces caused by the girder often lead to fatigue cracking.

Figure 2.4.4.1-2: Fabricated Girders and Rolled Cross-Frame Members.

Figure 2.4.4.1-3: Girder Failure Due to Brittle Cracking – Condition State 4.

Figure 2.4.4.1-4: Girder Failure Due to Brittle Cracking – Condition State 4.

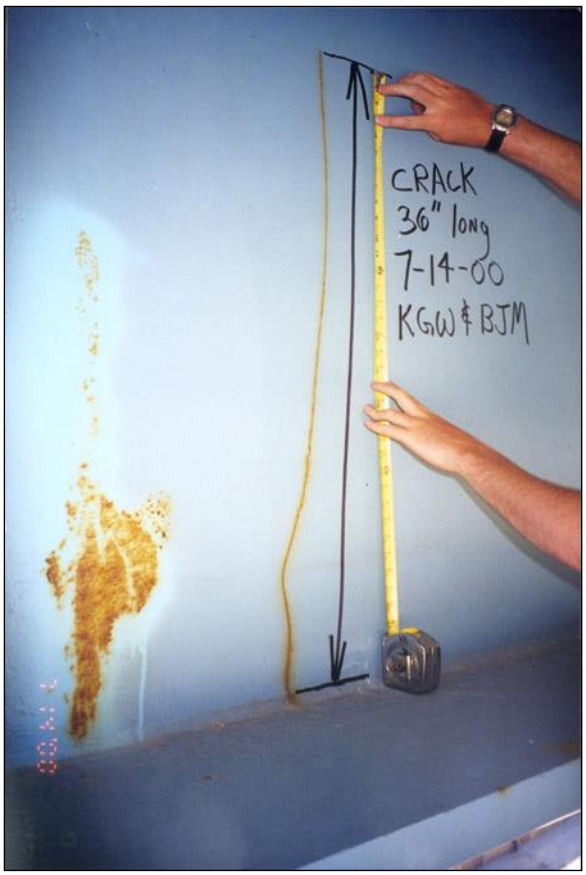

Figure 2.4.4.1-5: Brittle Crack 3 Feet Long in a Girder Web 10 Feet Deep, Opposite of Lower Lateral Shelf Plate - Condition State 4.

Figure 2.4.4.1-6: Same Crack as shown in Figure 4.75, Inside Face of Girder at Lower Lateral Shelf Plate - Condition State 4.

Figure 2.4.4.1-7: Fatigue Crack at a Welded Cover Plate End (Rust Line within the Ground Area).

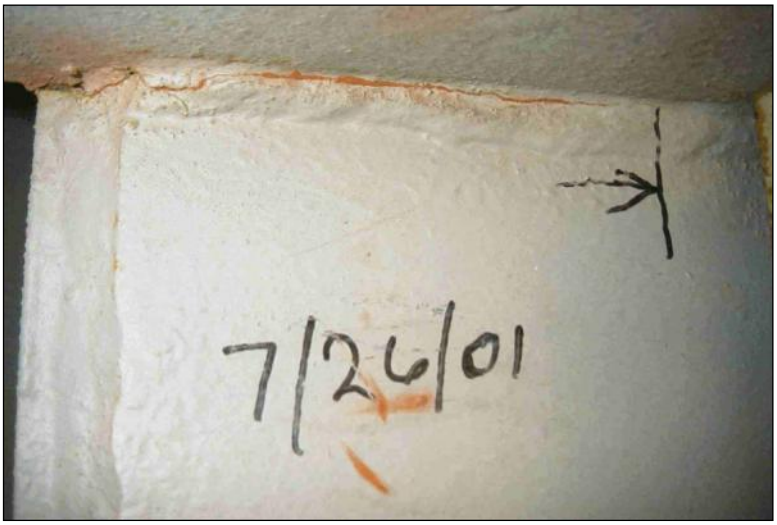

Figure 2.4.4.1-8: Rust Line at Weld Toe Indicating a Fatigue Crack.

Figure 2.4.4.1-9: Welded Girder to Cross Girder Connection Detail.

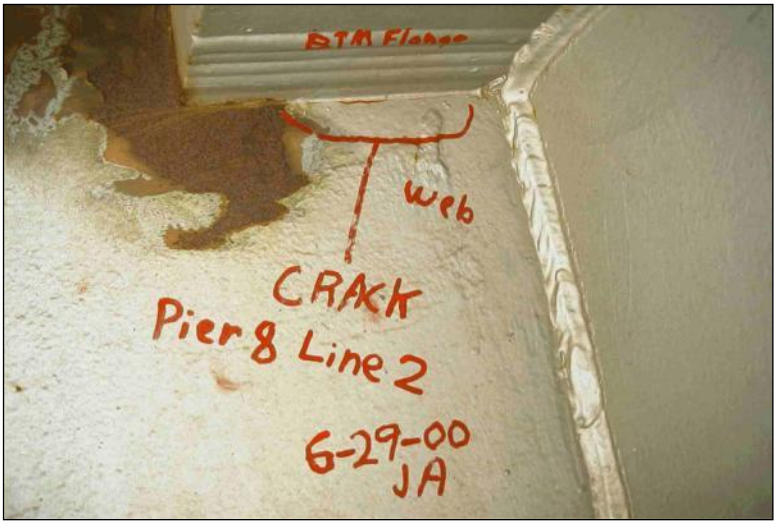

Figure 2.4.4.1-10: Fatigue Crack at the Girder Bottom Flange/Cross Girder Web Connection.

Figure 2.4.4.1-11: Fatigue Crack Through Girder Web Arrested with a Drilled Hole, Top of Connection Plate - Condition State 2.

Figure 2.4.4.1-12: Fatigue Crack at Flange/Web Intersection Arrested with Drilled Holes - Condition State 2.

Figure 2.4.4.1-13: Fatigue Crack at Flange/Web Intersection After Cleaning – Condition State 2.

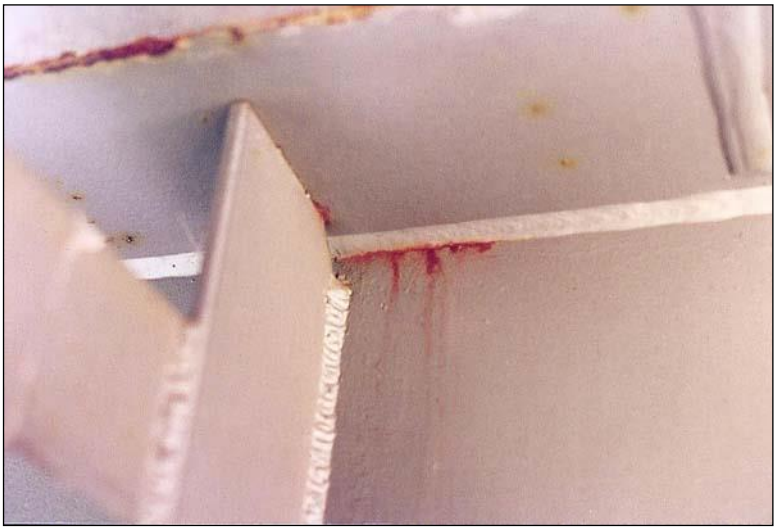

Figure 2.4.4.1-14: Fatigue Crack Indicated by "Fretting Corrosion" Rust Stains Along a Longitudinal Fillet Weld - Condition State 3.

Figure 2.4.4.1-15: Double Fatigue Cracks in Girder Web, Top of Connection Plate – Condition State 3.

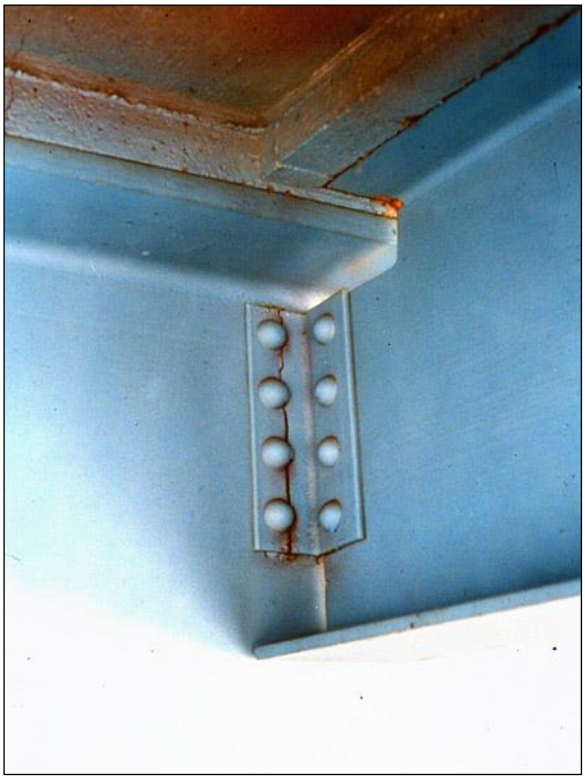

Figure 2.4.4.1-16: Fatigue Crack Through the Rivet Line of a Connection Angle - Condition State 3.

Figure 2.4.4.1-17: Unarrested Fatigue Crack - Condition State 3.

Box Girders: In-plane bending fatigue prone details on box girders are similar to those found on fabricated open girders. In addition, fatigue inspection in the flexural areas of steel box girders should include the following items:

- Investigating back-up bars that are butt-welded together end-to-end, or tack welded into place, and located within tension or stress reversal zones. These bars carry the same longitudinal stresses as the girder itself and may produce fatigue cracks due to low quality welds joining the bars or due to discontinuity of the bars.

- Investigating any web or flange longitudinal stiffeners that are welded together end-to-end and located within tension or stress reversal zones. Pay particular attention to all questionable details along the tension flanges.

- Checking for out-of-plane bending cracks. These cracks found in box girders are similar to those found on fabricated open girder connections. However, the forces in box girder diaphragms/cross-frames may carry higher loads than those of open girders, and out-of-plane fatigue cracking is more likely.

- Checking all welded attachments inside the box including the transverse stiffeners, cross-frames, diaphragms, lateral bracing (including web-gaps), and stay-in-place deck panels.

- Checking the ends of all partial depth diaphragm connections sometimes found inside of arch tie girders. These diaphragms are placed in line with the floor beams, and bending moments at the floor beam connection push or bend the tie girder web sideways between the flanges and diaphragm. Damage typically shows up as a U-shaped crack around the diaphragm connection plate end or as a horizontal crack near the flange/web weld.

Figure 2.4.4.1-18: Steel Box Girders - Condition State 1.

Floor System Components: In-plane bending fatigue prone details may be present on stringers and floor beams. To facilitate erection and connection fit-up, stringers and floor beams may have cut flanges, copes, or blocked flange plates at their ends. High, localized stresses can result due to the abrupt change in cross-sectional geometry. In addition, cut flanges and copes may have sharp re-entrant corners rather than smooth transitions, creating stress risers. These details may also have been torched, leaving rough edges or notches.

Fatigue inspection of floor system components should include the following items:

- Closely looking at re-entrant corner details near the tension flange. Cracks will typically start at these locations, propagating into the web.

- Checking for stiffeners that may be erroneously welded to a tension flange, creating a stress riser. Bearing stiffeners are common on floor beams and stringers. Carefully check the welds and flange on these floor system components for cracks.

- Looking for out-of-plane bending cracks. These cracks may occur in the webs of floor beams and in the webs of their associated cantilever brackets. Cracking may take place when the stringers rest on the floor beam's top flange and the girder top flange is not embedded in the deck. A tie plate provides continuity between the floor beam and bracket top flanges but is not welded to the girder top flange. Deflection of the girder relative to the floor beam bends the floor beam/bracket web sideways, producing fatigue cracks near the top flange. This same type of cracking may occur in the floor beam connection to the girders of arch bridges and to truss lower chords..

Figure 2.4.4.1-19: Fabricated Steel Girders and Floor Beams, and Rolled Stringers.

Riveted and Bolted Members: Though welded structures are most often associated with fatigue concerns, mechanically fabricated members are also susceptible to fatigue damage. Vulnerable locations are essentially the same for welded and riveted members, that is, at

connections. Damage is most likely to be found at the ends of members with simple supports provided by connection angles. Because these types of connections carry a small amount of bending, moments are applied to the connection angles, and prying forces are applied to the rivets and bolts passing through the angle's outstanding legs (legs perpendicular to the member being connected).

Fatigue inspection of riveted and bolted members should include the following items:

- Investigating the fasteners. Fasteners closest to the ends of the connection angles are subjected to the highest prying force and cyclic loading may cause the fastener shank to fail in fatigue. Signs of fatigue include fretting corrosion/rust powder under the fastener head or nut, gaps between the connected parts and the fastener head or nut, a dull sound when the head is tapped with a hammer or a missing fastener.

- Investigating the connection angles. Fatigue cracks can be found in the connection angles either in the angle's outstanding legs or in the parallel legs (legs parallel to the member being connected). Cracks will usually originate near the angle's fillet at one end of the angle. The same type of fastener and angle failures may also occur at diaphragm/cross-frame connections.

- Looking for out-of-plane bending damage in the girder web adjacent to floor beam connections. This usually occurs at web-gaps created when transverse or longitudinal stiffeners use a separate row of fasteners than those used to join the connection angles to the web.

- Checking for cracks that may occur in the webs of floor beams and in the webs of their associated cantilever brackets. This is a detail similar to that found on welded floor systems. Cracking may take place when the stringers rest on the floor beam's top flange and the girder top flange is not embedded in the deck. A tie plate provides continuity between the floor beam and bracket top flanges, but is not fastened to the girder top flange. Relative deflection of the girder to the floor beam bends the floor beam/bracket web sideways, producing fatigue cracks near the top flange. This same type of cracking is seen in the floor beam connection to truss lower chords.

Figure 2.4.4.1-20: Sheared Off Diaphragm Bolts Due to Fatigue.

Miscellaneous: Fatigue inspection of miscellaneous components should include the following item:

- Looking for stress risers on tension flanges such as tack welds, gouges, and indiscriminately placed attachment welds. These may occur on any welded or riveted/bolted member listed above. Flaws such as these should be marked, recorded, and ground smooth. Until the areas are repaired, the member should be closely monitored to spot crack development.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the bottom flange of a beam is struck by vehicular traffic and exhibits deformation out-of-pane. The defect would be reported under Distortion (1900) with the note indicating the deformation was caused by traffic impact.

Overload damage (buckled compression member or member element, yielded tension member or member component, crippled web at a support) and heat damage are not addressed as defects. The omissions should not be interpreted as meaning overload and heat damage are minor defects. On the contrary, overload damage and heat damage are significant defects that can be as severe as fatigue cracks. The inspector should document this type of damage in writing under the *Structural Notes* dialog box near the end of the Field Bridge Inspection Report form and notify the Program Manager immediately.

If excessive debris is present on the flanges, cleaning should be recommended under *Maintenance Actions*.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the

severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.4.1-21: Weathering Steel Girders with Scaling on the Bottom Flanges – Condition State 3**.**

Figure 2.4.4.1-22: Steel Girders – Corrosion Condition State 2.

Figure 2.4.4.1-23: Girder – Corrosion Condition State 2.

Figure 2.4.4.1-24: Steel Girder – Corrosion Condition State 2.

Figure 2.4.4.1-25: Steel Girder – Corrosion Condition State 2.

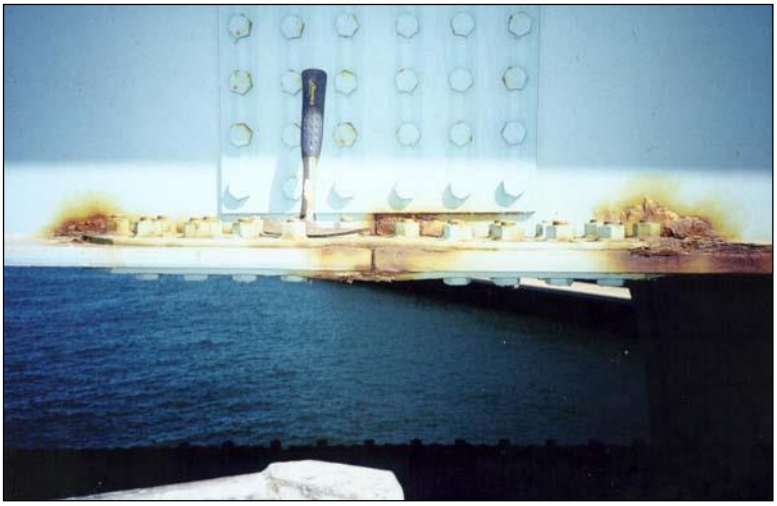

Figure 2.4.4.1-26: Pack Rust at Bolted Girder Bottom Flange Field Splice – Connection Condition State 2.

Figure 2.4.4.1-27: Impact Damage to a Girder Bottom Flange – Distortion Condition State 2.

Figure 2.4.4.1-28: Impact Damage to Bottom Flange of a Two-Girder Bridge – Distortion Condition State 2.

Figure 2.4.4.1-29: Pit Caused by Section Loss on Girder Bottom Flange – Corrosion Condition State 3.

Figure 2.4.4.1-30: Steel Girders – Corrosion Condition State 3.

Figure 2.4.4.1-31: Steel Girders – Corrosion Condition State 3.

Figure 2.4.4.1-32: Section Loss on Girder Web Painted Over – Corrosion Condition State 3.

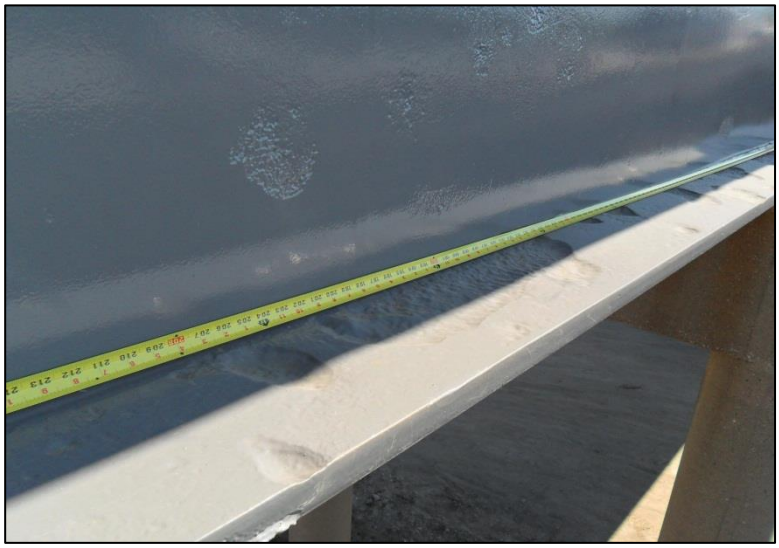

Figure 2.4.4.1-33: Severe Pitting Along Bottom Flange Painted Over – Corrosion Condition State 4.

Figure 2.4.4.1-34: Laminate Section Loss of Web – Corrosion Condition State 4.

Figure 2.4.4.1-35: Steel Rolled Beams – Corrosion Condition State 4.

Figure 2.4.4.1-36: Steel Beam – Corrosion Condition State 4.

Figure 2.4.4.1-37: Through Thickness Section Loss of a Rolled Beam Web - Condition State 4.

Figure 2.4.4.1-38: Heavy Pack Rust (Painted Over) at a Multiple Plate Bearing Stiffener – Connection Condition State 4.

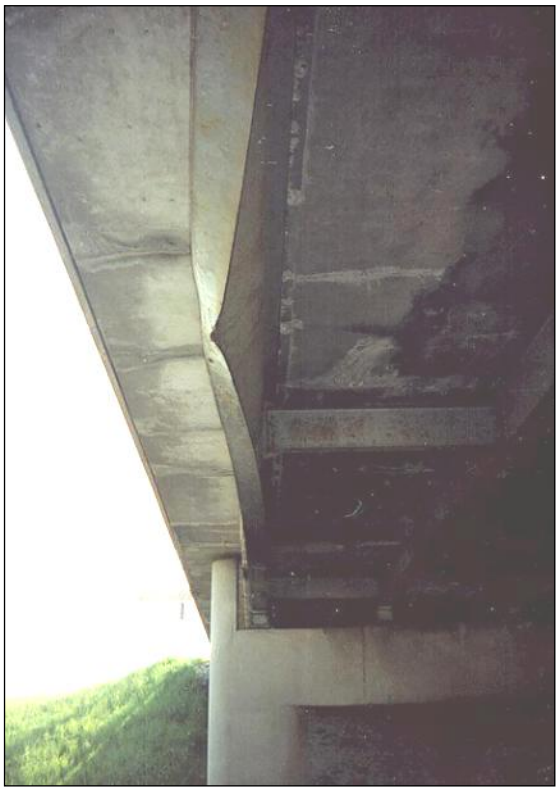

Figure 2.4.4.1-39: Impact Damage to Exterior Girder – Distortion Condition State 4.

2.4.4.2 Steel Cable – Primary (Element 147)
Steel Cable – Secondary (Element 148)

These elements are for all cable groups regardless of protective system. These steel elements are tension only members used in suspension, cable-stayed, post-tensioned concrete and tied arches.

Cables may be used as vertical suspenders, angled cable stays, catenaries or post-tensioning strands. Unlike solid rods, many individual wires are helically spun together, placed parallel to each other, or spun into rope to build up the size of the cable. End anchorages are usually made by brooming or spreading apart the cable wires inside a steel fitting. The conical-shaped steel fitting is then filled with a socketing medium, such as molten zinc, to lock the wires in place. Another anchoring method for smaller diameter cables or individual strands is the placement of jaws (wedges) around the strand. These jaws grip and anchor the strand as the strand is pulled and seated into a conical-shaped anchor block.

The most common method used to coat cables/rods for corrosion protection is galvanizing the individual cable wires or rods. Painting is another method. However, paint's effectiveness for corrosion protection of cables is questionable due to its inherent brittleness coupled with the cable's flexibility. Flexible proprietary coatings are also used. These "caulk-like" products never completely harden, thereby allowing a cable to deflect or vibrate without cracking or chipping the coating. These coatings may also be used to fill the voids between the cable wires during the manufacturing process. On large suspension bridge main cables, a zinc dust paste (in earlier days lead paste) is typically spread onto the outside perimeter of the cable and then tightly wrapped with a galvanized steel continuous band. On some cable-stayed bridges and post-tensioned concrete boxes, the cables/rods may be wrapped with a plastic tape. Cables may be covered with a polyethylene tube. Grout, grease, or mastic material is often used to fill the voids between the element and the wrapping tape or tube. These coating systems, however, do not allow for the direct inspection of the elements and have a tendency to trap moisture.

Primary cables include all main suspension (catenary cable) or cable stay cables not embedded in concrete.

Secondary cables include suspender cables (under the catenary cable) or other groups of cables that carry superstructure (deck or girder) loads to the main load carrying cable or arch.

These cables do not include moveable bridge cables. The assessment Moveable Bridge Cables (9021) shall be used to capture issues with equalizer or lifting cables on moveable bridges. Refer to Chapter 7, Part 2 for additional information on Assessments.

Secondary cables are secondary elements. Observed failures should be noted within the Inspection Report form. However, it will not significantly impact the structural integrity of the superstructure. Remedial actions will need to be taken to repair the failure, as the secondary members are responsible for transferring loads. A missing element may, in time, result in unwarranted stresses within the primary structural members.

Element Level Inspection

Corrosion is the primary concern to any steel cable and has a more profound structural effect on cables than it does on solid elements. Because cables have a much greater surface area than a solid rod with the same cross-sectional area, surface corrosion on the wires will reduce a cable's cross-sectional area more than on a solid element. For example, a cable containing 153 – 0.162 inch diameter wires has about the same area as a solid 2-inch diameter bar (3.14 square inches). If corrosion removes 1/32 inch of thickness from all steel surfaces, the remaining cable area would be 1.19 square inches (a 62 percent reduction in area), and the remaining solid rod area would be 2.95 square inches (a 6 percent reduction in area). It is clear that corrosion can have a serious effect on the load-carrying capacity of cable elements.

Since cables are made up of many individual wires, only about 10 percent of the total wire surface area can be seen during a routine inspection. Voids between the interior wires can trap moisture. The inspector must therefore assume that the same corrosion severity that can be seen on the cable's surface exists on the interior wires as well.

On the inspection report form, cable elements are recorded in units of "each". This is because cable removal/replacement is normally required when severe deterioration exists (except for suspension bridge main cables). For each cable, it is the inspector's task to assign the most appropriate defect Condition State to the entire element. This will quantify the element's surface condition and help generate quantity/cost estimates for future remedial work. If multiple defects are found within the same unit of measure, the inspector shall record the predominant defect on the inspection report and describe other defects within the notes under the element. The predominant defect is determined first by Condition State. If Condition States are equal, the hierarchy then falls to the lowest associated defect number.

Maintenance inspection of cables should include the following items:

- Looking for broken wires. Broken wires may be caused by pack rust due to bending near the anchorages, excessive section loss due to corrosion, or abrasion against the cable guide due to wind induced vibrations.

- Indicating on the report if the cables are vibrating excessively due to wind, and noting the cable's amplitude, wind speed, and wind direction.

- Checking for cable loosening or slippage at the end fittings. Signs of this condition may be wire abrasion and/or corrosion having an orange to light brown color with a dusty appearance.

- Inspecting the lower end fittings for excessive cable corrosion where water would tend to accumulate, as well as for any cracks in the casting.

- Noting any slipping or unraveling of the main cable banding on suspension bridges.

- Reporting severe corrosion that has formed pack rust between individual wires or between the wires and end fittings.

Large suspension and cable-stayed bridges are individually unique and will normally have their own operation and maintenance manuals to guide the inspector during routine

evaluations. There are many components which make up these structures. Due to their scarcity in Wisconsin, the inspection of these components will not be discussed here. The reader is referred to Federal Highway Administration (FHWA) publication *Safety Inspection of In-Service Bridges Participant Notebook* (two-week training course) for additional guidance.

Overload damage (yielded cable or cable pulled out of its anchorage) and heat damage are significant defects that can be as severe as fatigue cracks. The inspector should document this type of damage in writing under the *Structural Notes* dialog box located near the end of the Field Bridge Inspection Report form, and notify the Program Manager immediately.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.4.2-1: Cable Coated with a Flexible Resin System – Condition State 1.

Figure 2.4.4.2-2: Galvanized Steel Cable – Condition State 1.

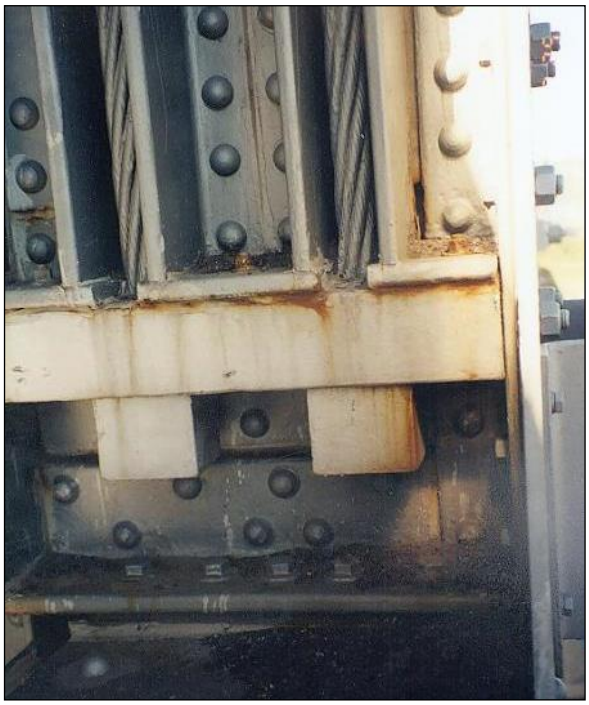

Figure 2.4.4.2-3: Cable Anchorage for a Tied Arch Bridge – Condition State 1.

Figure 2.4.4.2-4: Galvanized Steel Cable Exhibiting Freckled Rust – Corrosion Condition State 2.

Figure 2.4.4.2-5: Cable Anchorage for a Cable Stayed Bridge – Connection Condition State 2.

2.4.4.3 Steel Tension Rods/Post-Tensioned Cables (Element 8165)

Tension rods are most often used to strengthen concrete elements as a retro-fit. They are commonly installed on the exterior of the element. Post-tensioned cables are most often used to prestress concrete box girder bridges. This keeps the girder cross-section in compression under all loads, thereby eliminating cracking. This results in a girder much stiffer and stronger than a cracked/conventionally reinforced concrete girder. These elements are also used to transversely post-tension timber slab bridges, or other plank-type superstructure panels. Post-tensioning transverse to the roadway creates friction, allowing the many individual panels or boards to act as a unit.

Normally, only the ends of the rods are visible for inspection, although early examples of post-tensioned concrete box girders left the entire rod exposed within the cells.

This element does not include the suspender cables or rods on tied-arch structures. Those members shall be evaluated under the Steel Cable – Secondary (Element 148).

Two methods are commonly used to anchor the ends of the rods or cables. For rods, their ends are threaded. After jacking the rod into tension, a bearing nut is screwed onto the threaded end tight to the anchorage plate, and the jack released. Cables will often use a wedge system for anchorage. Cables pass through a special anchorage plate with conical-shaped holes. After jacking the cable into tension, cone-shaped wedges are placed around the cable, and the jack released. Relaxation of the cable draws the wedges into the conical shaped anchorage holes, locking the cable in place.

Element Level Inspection

On the inspection report form, tension rod/post-tensioned cable elements are recorded in units of “each”. For each rod or cable, it is the inspector’s task to assign the most appropriate defect Condition State to the entire element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work. If multiple defects are found within the same unit of measure, the inspector shall record the predominant defect on the inspection report and describe other defects within the notes under the element. The predominant defect is determined first by Condition State. If Condition States are equal, the hierarchy then falls to the lowest associated defect number.

Element Level Inspection of tension rods/post-tensioned cables should include the following items:

- Looking at the anchorages for lack of bearing or slip of the cable through the wedge. Sudden losses of force may allow the rod/cable to snap and shoot out of the anchorage.

- Pulling/shaking rod ends to check for looseness. Looseness indicates a complete loss of prestressing force and element effectiveness.

- Lightly tapping the tensioned length of the rod/cable (if accessible) with a rubber mallet. Similar to the strings of a guitar, tensioned elements should ring, while un-tensioned elements will produce a dull thud. As an alternative, the rods/cables may

be shaken. Tensioned elements should be taught, while un-tensioned elements move easily.

- Looking for corrosion and its extent. Severe corrosion will produce section loss and an increase in tensile stresses.

- Looking for broken rods/cables. Broken elements may be caused by excessive corrosion that had caused overstresses. Corrosion may also cause section loss on the threads at the ends of a rod. This can cause the anchorage nuts or couplers to slip.

- Inspecting the anchorage nuts for cracks or other damage.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange

- Condition State 4 Severe Red

Figure 2.4.4.3-1: Exposed Post Tensioning Rods Inside of a Concrete Box Girder - Condition State 1.

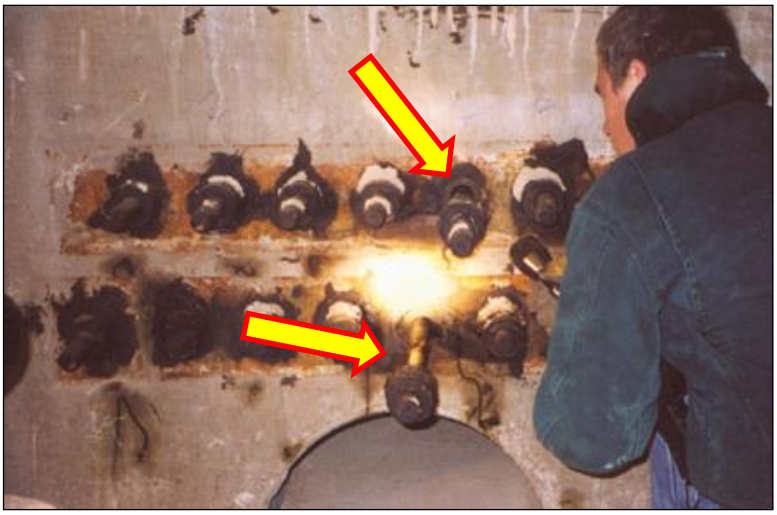

Figure 2.4.4.3-2: Post-Tensioning Rods (2) Protruding from the Anchorage, Indicating a Loss of Prestressing Force – Connection Condition State 3.

Figure 2.4.4.3-3: Protruding Post-Tensioning Rod – Connection Condition State 3.

Figure 2.4.4.3-4: End of Post-Tensioning Rod with Cracked Nut – Connection Condition State 3.

2.4.4.4 Steel Truss (Element 120)

This element encompasses all steel truss elements. This includes all tension and compression primary members throughout the truss.

The components included under this element are primary load-carrying members, and bracing members in certain circumstances, and are theoretically loaded in either pure tension or compression. Members carrying tensile loads are NSTMs. Though each of these members may contain internal redundancy (multiple eyebars or built-up riveted shapes) this fact should not be neglected and each element of the member inspected as a NSTM.

Figure 2.4.4.4-1: Truss Bottom Chord.

On simply supported trusses, the bottom chords are NSTMs On continuous trusses, both the bottom chord and top chord are NSTM, depending on their location relative to the pier. Bottom chords are in tension and therefore NSTM between the substructure elements. Top chords are in tension and therefore NSTM above the piers.

Element Level Inspection

On the inspection report form, trusses are recorded in units of lineal feet. A steel truss bridge will have at least two trusses. The correct method to calculate the quantity is to sum of all lengths of each panel as measured longitudinally along the travel way. This element includes all primary components in plane with the truss including, the bottom chord, top chord, verticals and diagonals, as well as bracing member above the roadway on through trusses. The truss components may exhibit more than one Condition State along its length. Distress observed on vertical, diagonal or bracing members shall be reported as the length projected along the length of the truss. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total

quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Through trusses contain overhead lateral and vertical sway bracing. For through trusses *or through arches,* the upper bracing (lateral, vertical, portal, and sway) above the deck will be evaluated and coded under Assessment 9170 – Truss or Tied/Arch Portal Bracing. All lateral bracing below the deck will be evaluated under assessment 9169 Lateral Bracing. Refer to Chapter 7, Part 2 for additional information on Assessments. The upper bracing supports the compression members of the truss stabilizing thus making them important members. Its failure could result in the buckling of the compression chord of the truss.

Deck trusses have no bracing located above the roadway. All bracing on deck trusses shall be evaluated as assessment 9169 Lateral Bracing. Refer to Chapter 7, Part 2 for additional information on Assessments.

Truss bottom chord members are located below the deck, exposing them to water, deicing chemicals, roadway debris, and occasionally drift impact from streams below during high water events. The top chord, vertical, and diagonal members are exposed to a less severe environment (except for deck trusses) but are still susceptible to traffic impact and water and deicing agents.

Safety Inspection

During the Element Level Inspection of truss members, it is important for the inspector to remember that the entire purpose of a bridge inspection is to ensure public safety. A structural inspection must also be carried out, regardless of the coating condition. The main purpose of a truss is to transmit loads to the substructure. A structural failure of one of a truss' NSTMs could mean a total bridge collapse. The following will serve as a guide for the inspector of what to look for and where to look for it while examining one of these members. This will also help the inspector judge a member's ability to carry the design loads and identify current or future structural problems.

Tension Members: It is required that tension members be identified prior to performing a NSTM Inspection. It is also advantageous to identify tension members for Routine Inspection. On simply supported trusses, the bottom chords will always be in tension, similar to the bottom flange of simply supported beams/girders. On continuous trusses, the top chord will be in tension over the piers, and the bottom chord is in tension between supports. Again, this is analogous to the top and bottom flanges of continuously supported beams/girders. On simply supported trusses, diagonals that point upward and away from midspan are tension members, as well as any counters which form an "X" pattern at or near midspan. There is no easy method to determine which diagonals are in tension of continuously supported trusses and vertical members of any truss. This determination is made during the NSTM inspection preparation phase and should not be done in the field.

Maintenance inspection of truss tension members should include the following items:

- Examining all tension components for corrosion and loss of cross-sectional area, which is the most common steel defect.

- Removing spot areas of debris accumulation to check for corrosion. Bird waste that is often found on the horizontal surfaces is acidic and traps moisture and road salts, accelerating corrosion.

- Checking for corrosion, not only on the primary member components, but also on members used to build up the primary members such as lacing bars and batten plates.

- Closely examining eyebar heads for corrosion, lack of movement, and cracking at the forging location.

- Checking rivet/bolt heads on built-up components. Corrosion on the heads may indicate corrosion along the entire fastener length, reducing structural integrity.

- Looking for pack rust, noted by individual plate bending between fasteners. Pack rust may be present between the plies of riveted/bolted connections such as lacing bars, batten plates, gusset plates, or field splices.

- Checking to make sure all eyebars of a multiple eyebar member are parallel to one another. This suggests that the loads are evenly distributed. Unintended bending and compressive stresses may be introduced into a tension member from substructure settlement or heavily rusted/frozen pinned joints. Signs of this are bowed or buckled tension members. The inspector should look for overloads on other members when this situation is encountered, since loads previously carried by the tension member must be redistributed somewhere else within the bridge.

- Checking all pins for excessive wear.

- Checking to see if pin spacers are keeping the eyebars or loop rods properly aligned and symmetric about the truss plane.

- Examining the condition of threaded members such as truss rods at turnbuckles.

- Checking suspect fasteners for looseness by twisting by hand or tapping the heads with a hammer.

- Looking for member distortion or scraping from traffic impacts.

Compression Member: On simply supported trusses, the top chords will always be in compression, similar to the top flange of simply supported beams/girders. On continuous trusses, the bottom chord will be in compression over the piers, and the top chord is in compression between supports. Again, this is analogous to the bottom and top flanges of continuously supported beams/girders. On simply supported trusses, diagonals which point downward and away from midspan are compression members. There is no easy method to determine which diagonals of continuously supported trusses and vertical members of any truss are in compression. This determination is made during the NSTM Inspection preparation phase and should not be done in the field.

Maintenance inspection of truss compression members should include the following items:

- Checking compression members for corrosion damage.
- Removing spot areas of debris accumulation to check for corrosion. Bird waste that is often found on the horizontal surfaces is acidic and traps moisture and road salts, accelerating corrosion.
- Looking for pack rust, noted by individual plate bending between fasteners. Pack rust may be present between the plies of riveted/bolted connections such as lacing bars, batten plates, gusset plates, or field splices.
- Looking for compression overload damage in the form of local member element buckling or plate distortion. Global buckling will take the form of a bowed member or a member bowed into an “S” shape if a point support is provided between its ends.
- Checking suspect fasteners for looseness by twisting by hand or tapping the heads with a hammer.
- Looking for member distortion or scraping from traffic impacts.

Safety Inspection - Fatigue

Truss tension members are susceptible to fatigue damage. Fatigue cracks usually show up as rust stains or rusty breaks in the paint, propagating perpendicular to the direction of stress.

The following text attempts to summarize locations for fatigue prone details and what the inspector should examine on truss tension members. Nonetheless, it is strongly recommended that the inspector to become familiar with the *Manual for Inspecting Bridges for Fatigue Damage Conditions*, as this reference will have more complete information on the topic. Any fatigue related damage found should be recorded under Condition State 4 and the Program Manager notified immediately.

Eyebars and Loop Rods: These are pin-connected members. Many old eyebars were fabricated by forging cast eyebar heads to a rolled bar which formed the eyebar body. Loop rods are a very old style of tension member. The loops were formed by heating and bending the rod ends, then forging the free end back onto the body.

Figure 2.4.4.4-2: Multiple Eyebars Connected with a Pin.

Fatigue inspection of eyebars and loop rods should include the following items:

- Checking for fatigue cracking at the forge zones, as well as at the eyebar head to body transition when the bar edge is flame cut. When the pins are heavily corroded and appear to have locked up eyebar or loop rod movement, transverse cracks may appear in the member body away from the forge zone or in the eyebar head. This is due to unintended bending moments are being introduced into the member.

- Examining plates or attachments that connect two or more eyebars by welding. Cracks found in the weld material can readily propagate into the eyebar base metal.

Riveted Built-up Members: The primary cause of fatigue cracks in these members occurs when their ends are pin-connected. Heavily-corroded pins have friction and, as a result, large, unintended bending stresses may be introduced to the member.

Fatigue inspection of riveted built-up members should include the following items:

- Closely inspecting all surfaces for fatigue cracks on tension members and on the hangers of cantilevered trusses. Fatigue cracks are most likely to form at areas of stress concentrations such as rivet holes.

Miscellaneous: For any welded or riveted/bolted tension members listed above, fatigue inspection should include the following items:

- Looking for stress risers in the form of welded repairs or reinforcement plate attachments. Welded reinforcement plate attachments have one of the lowest fatigue strengths of any detail.

- Closely examining areas of heavy corrosion and section loss that can also act as notches and stress risers.

- Checking defects such as tack welds, gouges, and indiscriminately placed attachment welds. Welds made to old steel that is not very weldable have a tendency to cause cracking in the base metal. These defects should be ground smooth or closely monitored to spot crack development.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

All defects on gusset plates connecting bracing below or above the deck on through trusses would be recorded under the assessment notes for 9169 Lateral Bracing if below or 9170 Truss or Tied/Arch Portal Bracing if above. Defects found on gusset plates within the plane of the truss would be captured under the appropriate defect under the Steel Gusset Plate element 162.

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance debris impact observed along the bottom chord of the truss would be recorded under defect Distortion 1900. The inspector would note in the defect notes that the distortion was caused by debris impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

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- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange

- Condition State 4 Severe Red

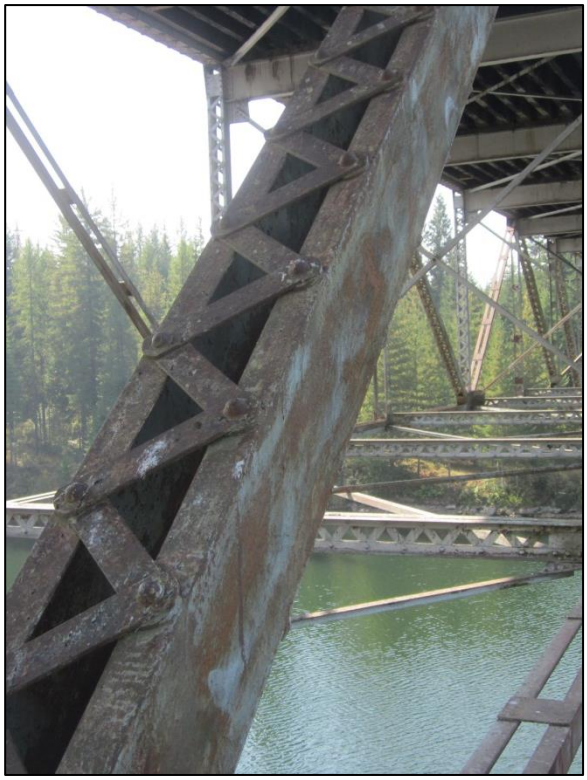

Figure 2.4.4.4-3: Diagonal Truss Member Exhibiting Rust with No Loss of Section – Corrosion Condition State 2.

Figure 2.4.4.4-4: Batten Plate on Truss Bottom Chord – Connection Condition State 4.

Figure 2.4.4.4-5: Truss Bottom Chord (Note Impact Damage) – Distortion Condition State 2.

Figure 2.4.4.4-6: Impact Damage to a Truss Sway Strut – Distortion - Condition State 2.

2.4.4.5 Steel Gusset Plate (Element 162)

Gusset plates are used to connect several superstructure members together. They are commonly used on trusses and arches and may connect primary or secondary members together. They are constructed of steel plate and may be arranged in pairs or as a single plate. The superstructure members are fastened to the gusset plate by way of welds, bolts, rivets, or a combination. Plate shapes are usually atypical. A gusset plate typically connects three to five members together at a point.

Gusset plates are considered NSTMs when they connect one or more NSTM together. See Section 2.4.4.8 NSTM Steel Superstructure Inspection for more information on NSTMs.

Element Level Inspection

On the inspection report form, gusset plates are recorded in units of each. The correct method for calculating the total quantity is the sum of the number of primary load path gusset plate assemblies. Only those steel gusset plate(s) connections that are on the main truss/arch panel(s). For multiple plate gusset connections at a single panel point, the quantity shall be one gusset plate regardless of the number of individual plates at the single connection point. Gusset plates connecting lateral or sway bracing below or above the deck are not included in this quantity and shall be recorded under the assessment 9169 Lateral Bracing if below or 9170 Truss or Tied/Arch Portal Bracing if above. Any defects found on these gusset plates shall be noted under the assessment notes and used in evaluating the overall assessment (EA).

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. For a gusset plate, only one defect will be applied to each gusset assembly. Therefore it is the inspector's responsibility to determine the predominant defect for the entire assembly. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

If multiple defects are found within the same unit of measure, the inspector shall record the predominant defect on the inspection report and describe other defects within the notes under the element. The predominant defect is determined first by Condition State. If Condition States are equal, the hierarchy then falls to the lowest associated defect number.

Gusset plates may be comprised of several components, including built-up sections, filler plates or angles for fastening to primary members. Asymmetric plate shapes are also common to accommodate different sized members and orientations. For this reason gusset plates undergo very complex reactions and as such, require very careful and detail oriented inspections.

Element level inspection of gusset plates should include the following items:

- Examining the connections to the truss to check for loose, corroded, or missing fasteners, cracked welds, and fatigue cracks. The inspector should take note of any tack welds that may be present on the gusset plate as these are fatigue prone details. Partially cracked tack welds pose a large threat for crack propagation into the base metal. Fully cracked tack welds with no evidence of base metal cracking are not

problematic. Fatigue cracking in gusset plates is commonly found at rivet holes. Rivet holes are punched rather than drilled resulting in overstressing of the surrounding plate material.

- Checking suspect fasteners for looseness by twisting by hand or tapping the heads with a hammer. Loose fasteners or connections may be the result of an overload that caused excessive tension forces in the member.

- Checking corroded areas for excessive section loss that may be increasing member stress. Particular attention should be given to areas near the roadway where connections to horizontal surfaces can trap water and deicing agents.

- Removing spot areas of debris accumulation to check for corrosion. This is especially true on truss connections where the bottom chord is plated allowing debris to build up and come into contact with the lower chord gusset plates. Bird waste that is often found on these elements is acidic and traps moisture and road salts, accelerating corrosion.

- Checking repairs and retrofits. Welded retrofits are extremely problematic and are prone to fatigue cracking at the welds. Inspect all plate layers for pack rust and distortion.

- Inspecting of the gusset plate and its connecting member for distortion, such as a bow, sweep, or kink. This distortion may occur in the plate between the end row of fasteners of one member and an adjacent row of fasteners in another member. This is called the free edge of the gusset plate. This would suggest a structural overload, or flexural bending and further investigation of the primary load-carrying members should be carried out. Plate distortion may also be induced by pack rust between members or plates.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

All defects on gusset plates not in the plane of the truss and not connecting the portal members to the truss end posts shall be recorded under defect Connection 1020 on the Steel Truss element 120. Defects found on gusset plates in the plane of the truss or connecting portal members to the truss end posts shall have the defects captured under the appropriate defect on the Steel Gusset Plate element 162. Gusset plates connecting bracing below the roadway on through trusses or all deck trusses would be recorded under the assessment notes for 9169 Lateral Bracing.

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance debris impact observed on a gusset plate of the bottom chord of the truss would be recorded under defect Distortion 1900 under the Steel Gusset Plate element. The inspector would note in the defect notes that the distortion was caused by debris impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.4.5-1: Vertical to Bottom Chord Gusset Plate Deformed Out-of-Plane – Distortion Condition State 2.

2.4.4.6 Steel Arch (Element 141)

This element refers to all steel arches regardless of protective system. There are several components that make up an arch bridge, such as the arch ribs, spandrel bent columns and caps, cable hangers, built-up hangers, diagonal and vertical braces, and tie girders. This section concerns itself with all of the primary structural arch components.

Arches are primary load-carrying elements resisting axial compressive loads and bending moments. Though most steel arches have two main members, they are not tension members and are therefore not considered NSTM. Arch members are classified as solid ribbed, braced ribbed (trussed arch), spandrel braced or tied. The *Bridge Inspector's Reference Manual* has several photographs in Chapter 10 illustrating some of these arch styles.

Solid ribbed steel arches are fabricated into I-girders or box shapes. Braced rib arches have two curves (usually fabricated boxes) defining the arch shape, braced with truss webbing between the curves. These are usually used for longer spans or where better control of live load deflections is required. Spandrel braced arches are similar to solid ribbed arches, but have diagonal bracing between the spandrel bents above the arch. Tied arches have their ends connected with a tension tie girder as a means to removing the arch's horizontal thrust from its bearings. These tension ties are NSTMs of the arch element. The ties and arches are usually fabricated box members.

Deck arches receive deck, floor system, and traffic loads by way of steel spandrel bents. Spandrel bents consist of two or more steel columns connected at their tops by a transverse steel girder. The bent's girder usually acts as a floor beam, and its reaction loads are delivered to the bent columns. The columns bear directly on top of the arch ribs.

Figure 2.4.4.6-1: Steel Arch and Spandrel Bent Columns.

Through arches and tied arches suspend the deck, floor system, and traffic loads underneath by way of hangers. The hangers may be either cables or built-up steel tension members. On

half through arches and continuous tied arches, the bridge deck is also supported on spandrel bents near the arch bearings.

Element Level Inspection

On the inspection report form, arches are recorded in units of lineal feet. A steel arch bridge will have at least two arches. The correct method to calculate the quantity is to sum of all lengths of each panel as measured longitudinally along the travel way. This element includes all primary components in the plane of the arch including all spandrel components (columns, caps and bracing). The arch components may exhibit more than one Condition State along its length. Distress observed on vertical, diagonal or bracing members shall be reported as the length projected along the length of the arch. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

The arch ribs, arch diagonals, spandrel columns, spandrel longitudinal bracing, built-up hangers, and tie girders are all evaluated under the arch element. Steel arches include both solid and braced ribbed arches. On braced rib arches, the top and bottom chords and the vertical and diagonal longitudinal bracing between them should be considered part of the arch element.

Through arches contain overhead lateral and vertical sway bracing. For through trusses *or through arches,* the upper bracing (lateral, vertical, portal, and sway) will be evaluated and coded under Assessment 9170 – Truss or Tied/Arch Portal Bracing. All lateral bracing below the roadway and not between the arch ribs will be evaluated under assessment 9169 Lateral Bracing. Refer to Chapter 7, Part 2 for additional information on Assessments. The upper bracing supports the arch ribs, which are compression members, preventing the ribs from moving out of plane thus making them important members. Its failure could result in the buckling of the arch rib.

Deck arches have no bracing located above the roadway. All bracing on deck arches shall be evaluated under the arch element. Similar to the bracing between arch ribs, all bracing on a deck arch can be viewed as bracing the compression members (arch ribs). Therefore all the bracing on deck arches is considered primary.

Safety Inspection

During the Element Level Inspection of arch, longitudinal bracing, column and shaft members, it is important not to forget that the entire purpose of bridge inspection is to ensure public safety. A structural inspection must also be carried out. The following will serve as a guide for the inspector of what to look for and where to look for it while examining one of these members. This will also help the inspector judge a member's ability to carry the design loads and identify current or future structural problems.

Arches: Arch ribs carry compressive loads and bending moments. Compressive forces are fairly constant throughout the arch. Bending moments will be variable and depend on the location of arch hinges. Moments are zero at the hinges. Arches may have three hinges (one at the crown and two at the bases), two hinges (at the bases), or no hinges (fixed).

Maintenance inspection of steel arches should include the following items:

- Looking for local compression overload damage in the form of local member component buckling, plate waviness or crippling.
- Looking for global buckling which will take the form of longitudinal rib misalignment.
- Checking suspect fasteners for looseness by twisting by hand or tapping the heads with a hammer.
- Checking corroded areas for excessive section loss that may be increasing member stress. Particular attention should be given to details that trap water.
- Inspection of the longitudinal bracing members of braced rib arches. These members should be inspected in a manner similar to truss members (see Section 2.4.4.4 Steel Truss (Element 120). They are designed to take compressive loads, tensile loads or both.
- Examining the rib splice plates for loose fasteners and excessive corrosion.
- Investigating the hinge pins for corrosion and excessive wear.

Spandrel Components

Spandrel components on arch bridges may carry a combination of compressive loads and bending moments (spandrel columns and caps), tensile loads (hangers or longitudinal bracing members) or compressive loads (longitudinal bracing).

Maintenance inspection of spandrel components should include the following items:

- Looking for local compression overload damage in the form of local member component buckling, plate waviness or crippling.
- Looking for global buckling which will take the form of a bow or sweep in the member.
- Examining the member ends for cracks and loose fasteners.
- Checking suspect fasteners for looseness by twisting by hand or tapping the heads with a hammer.
- Checking corroded areas for excessive section loss that may be increasing member stress. Particular attention should be given to details that trap water.

Safety Inspection - Fatigue

Tension members on arches and shafts include hangers, arch braces, and spandrel braces. Fatigue cracks usually show up as rust stains or rusty breaks in the paint, propagating perpendicular to the direction of stress.

Fatigue inspection of arch and spandrel components should include the following items:

- Closely inspecting all surfaces for fatigue cracks on tension members and on the hangers of through and tied arches. Fatigue cracks are most likely to form at areas of stress concentrations such as rivet holes or discontinuous welds.

- Looking for stress risers in the form of welded repairs or reinforcement plate attachments. Welded reinforcement plate attachments have one of the lowest fatigue strengths of any detail.

- Closely examining areas of heavy corrosion and section loss that can also act as notches and stress risers.

- Checking defects such as tack welds, gouges, and indiscriminately placed attachment welds. Welds made to old steel that is not very weldable have a tendency to cause cracking in the base metal. These defects should be ground smooth or closely monitored to spot crack development.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

All defects on gusset plates connecting bracing below or above the deck on steel arches would be recorded under the assessment notes for 9169 Lateral Bracing if below or 9170 Truss or Tied/Arch Portal Bracing if above. Defects found on gusset plates within the plane of the arch would be captured under the appropriate defect under the Steel Gusset Plate element 162.

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, traffic impact damage is observed on the arch rib of a through arch near the roadway. This damage would be recorded in the Steel Arch element under defect Distortion 1900. The inspector would note in the defect notes that the distortion was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the

severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.4.6-2: Pack Rust at Riveted Steel Arch Flange Plates – Connection Condition State 3.

2.4.4.7 Steel Pin or Pin and Hanger Assembly (Element 161)

These elements are primary load-carrying members. On trusses and two-girder system bridges, pin and hanger assemblies are NSTMs. The pins for this element refer to those used to connect truss members or girders, and not the pins sometimes used for bearings. Individual bridge pin elements are located at member end connections of many trusses or on multi-span girders where the girder frames into a transverse girder. Pin and hanger assemblies, consisting of two pins and two hangers, are found on multi-span girder bridges where it is necessary to locate an expansion hinge away from a pier. The assemblies are placed at the tip of a girder's cantilever span and are used to suspend an adjacent span. Cantilevered trusses also use pin and hanger assemblies, but for Element Level Inspection purposes, the pins are treated as individual pin elements, and the hanger is considered a truss vertical member. Truss hangers should therefore be inspected according to Section 2.4.4.1, as an axial or bending steel member.

Pins are intended to be frictionless connections that allow for member rotation but are not designed to carry any torsion. They are fabricated in a variety of sizes. The smallest are solid and use cotters to prevent the pin from walking out of the connection under vibratory loads. Medium diameter pins are also solid, but their ends are threaded so that nuts can be used to prevent walkout. Sometimes, holes are drilled through the center axis of medium sized pins. The largest pins have holes drilled through their center axis, through which passes a threaded rod. The rods also pass through pin end cap plates. Nuts are threaded onto the rod to retain the cap plates and therefore the pin. On trusses, pins are normally employed to connect the ends of eyebars or loop rods, although large pins can connect the ends of modern built-up members. On girders, single pins pass through web plates framed into transverse cross girders to form a non-expansion hinge.

Hangers are designed to act as links and are consequently intended to be tension only members. At least two are used per connection, one on each side of the girder web. Hangers may be shaped as simple flat plates, or as eyebars.

Figure 2.4.4.7-1: Pin and Hanger Assembly.

Figure 2.4.4.7-2: Single Pin and Plate Assembly.

Element Level Inspection

These elements encompass all steel pins and pin and hanger assembles regardless of protective system.

On the inspection report form, pin/pin and hanger assemblies are recorded in units of “each”. This is because complete pin and hanger removal/replacement is normally required for severe deterioration. For each pin or assembly, it is the inspector’s task to assign the most appropriate defect Condition State to the entire element, which includes the hangers (if present). This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work. If multiple defects are found within the same unit of measure, the inspector shall record the predominant defect on the inspection report and describe other defects within the notes under the element. The predominant defect is determined first by Condition State. If Condition States are equal, the hierarchy then falls to the lowest associated defect number.

Safety Inspection

During the Element Level Inspection of pin/pin and hanger assemblies, a structural inspection must also be carried out. A structural failure of either a pin or hanger on a NSTM bridge would likely mean a total collapse. The following will serve as a guide for the inspector of what to look for and where to look for it while examining one of these members. This will also help the inspector judge a member’s ability to carry the design loads and identify current or future structural problems.

Pins: Safety inspection of single pins or those in pin and hanger assemblies should include the following items:

- Examining all pins for signs of the desired member rotation about the pin, such as powdery orange or red rust (fretting rust) near surfaces that rub, cracked paint between the pin and member or physical movement as traffic crosses the bridge.

- Measuring the amount of pin wear on truss, arch, or girder hanger expansion hinge assemblies. Since access may be difficult due to closely-spaced members or cap plates, creative measurements must be made. Two measurements must be taken at each pin to obtain adequate information of pin or member wear. The inspector should measure the distance from the centerline of the pin to the end of the hanger and measure from the center of the pin to the inside flange surface of the girder through which the pin passes. These readings will give measurements for wear at the pin/hanger interface and pin/web interface, respectively.

- Measuring the amount of pin wear on non-expansion hinges. The inspector should measure from the center of the pin to the inside surface of the girder's top and bottom flanges. These readings will give measurements for wear at the bottom of pin/web interface and top of pin/web interface, respectively.

- Making the above-mentioned measurements from the centerline of the threaded rod on pins using cap plates.

- Comparing the above-mentioned measurements to the distances shown on the original design drawings, accounting for the pin hole tolerance (usually 1/32 inch). Wear of 1/8 inch or greater should be brought to the attention of the Program Manager. If the original design drawings are not available, the inspector should record the measurement for comparison to measurements taken on future inspections. If possible, a wire or stiff steel rule should be used to probe between the plies of plates to measure the distance from the pin surface to the surfaces mentioned above.

- Checking for ratcheting. On new structures, rotations are accommodated by the girder web sliding on the pin surface. Fretting corrosion between the web hole and pin surface will advance, eventually "locking up" the web/pin movement. After this occurs, rotations take place by the hanger sliding on the pin surface. This is known as ratcheting, and is evidenced by a broken paint film, wear marks, and corrosion between the pin nut and hanger plate.

- Looking for pack rust in between the girder web and hanger. Pins connecting plate hangers or tightly packed eyebars are difficult to access. As a result, these pins often do not receive proper cleaning or painting during maintenance operations, and excessive corrosion rather than excessive wear may be the consequence. Excessive corrosion may lock up the joint, introducing unintended bending stresses into the pin and hanger or superstructure member.

- Tapping the pin or threaded rod nut with a hammer to check for excessive looseness. If the pins are excessively loose, notify the Program Manager immediately. A bridge inspector should never unscrew a pin nut or remove a cap plate to get a better look at

the pin. Disassembly is not part of a Routine inspection. Doing so could be catastrophic if pack rust between the girder web and hanger has placed the assembly on the verge of failure. Disassembly is only undertaken as part of an In-Depth Inspection program and only after proper auxiliary joint support is in place.

- Checking the cap plates for flatness.
- Checking to make sure adjacent girder flanges and webs are in alignment.

Figure 2.4.4.7-3: Pin Shear Failure.

Hangers: Safety inspection of hangers in pin and hanger assemblies should include the following items:

- Measuring the distance between the hanger and girder web at several locations. Variation of 1/8 inch or more could mean hanger twist or lateral movement.
- Looking for fretting corrosion between the hanger and girder web, which will be evident by a dusty-looking reddish rust around the plates' interface. Fretting corrosion is caused by two tightly fitting plates rubbing against each other.

Safety Inspection – Hanger Cracks

Hanger plates are very susceptible to damage when corrosion "freezes" the pins and does not allow for free rotation. When this occurs, the assembly ceases to behave as a hinge, and begins to carry bending moments. These bending moments introduce tensile stresses into the hangers in addition to the tensile stress for which the hangers were designed. Out-of-plane bending stresses may also be generated from girder misalignment or pack rust. As a result, overstress cracks may develop in the hanger plate.

Inspection of hangers in pin and hanger assemblies should include the following items:

- Careful checking all edges and surfaces of all hangers, especially the ends beyond the pin centerlines, and the forged areas of any eyebars. Forged areas will usually be near the eyebar head and body junction.

- Checking both sides of the hanger for fatigue cracks, if possible. A flashlight and inspection mirror may help.

- Using nondestructive evaluation methods (dye penetrant, magnetic particle, ultrasonic) to look for cracks. Nondestructive evaluation should be performed as part of an In-Depth Inspection.

- Immediately reporting any cracks to the Inspection Program Manager. The nature of pin and hanger assemblies is that a failure of one may cause a domino effect failure on multi-girder bridges.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange

- Condition State 4 Severe Red

Figure 2.4.4.7-4: Steel Girders, Pin and Plate – Condition State 1.

Figure 2.4.4.7-5: Girder, Pin, and Hanger – Corrosion Condition State 2.

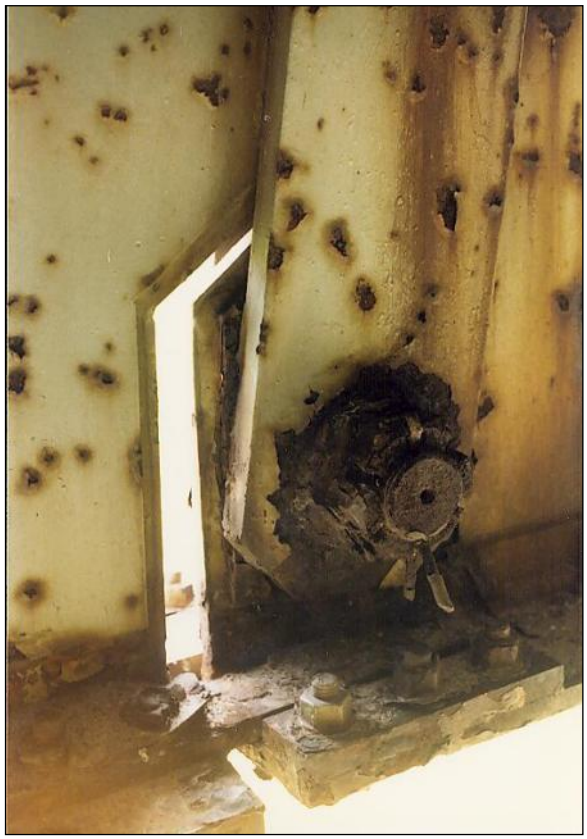

Figure 2.4.4.7-6: Pin and Hanger – Corrosion Condition State 4.

2.4.4.8 NSTM Steel Superstructure Inspection

Members are classified as being either nonredundant steel tension member (NSTM) or non-NSTM. Per CFR 23 Subpart C 650.305, a nonredundant steel tension member 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.” Superstructures that have one or two primary load-carrying members spanning between supports that are in tension or have a tension component are not load path redundant. This means that failure of one of these primary members will likely cause a collapse of the entire structure. Superstructures that have three or more primary load-carrying members spanning between supports are non-NSTM. These members are load path redundant, which means that if one of the primary members fails, the loads it carries can be redistributed to the remaining primary members, thereby preventing a total collapse.

Figure 2.4.4.8-1: NSTM Two-Girder Bridge.

Figure 2.4.4.8-2: NSTM Through Girder Bridge.

NSTMs require special attention during an inspection. There must not be any other member or system of members that will serve the functions of the member in question should it fail. Fatigue failures are the main cause of concern of steel NSTMs since fatigue failures can be brittle and give no warning as to imminent collapse.

The detailed procedures for carrying out a inspection are beyond the scope of this Manual. However, it is highly recommended for the inspector to become familiar with the FHWA Memorandum *Inspection of Nonredundant Steel Tension Members*, as this reference provides complete information on this topic. It contains information related to NSTM inspection organization, definitions of NSTM bridges, inspection procedures, reports, and recommendations. The following text describes the minimum NSTM Inspection requirement for the State of Wisconsin.

Inspection Preparation

The inspection team should develop the Inspection Procedures, Plan and NSTM Plan. Inspection Preparation is an important part of the NSTM Bridge Inspection. Important steps that will help in accomplishing a successful inspection are; Planning, Scheduling, Equipment, Personnel Requirements, and Field Inspection Procedures.

Historical Plan and Review

It is critical to gather all of the historical information that is available. Suggested information that may be available:

- Original design plans
- “As-built” plans
- Original shop drawings
- Construction history
- Maintenance history
- Rehabilitation history
- Bridge inspection reports and photographs

This information should be reviewed by the Inspection Program Manager and Inspection Team Leader prior to performing the NSTM Inspection, in order to determine the NSTM. It is strongly recommended that original plans and documents remain in the office of the maintaining agency and that only copies are taken to the field.

Identification of NSTMs

NSTMs shall be identified by the Program Manager working with the Inspection Team Leader. This identification shall be shown on the plans or a sketch of the bridge.

NSTMs, member components, and all other tension members must be identified on the Inspection form or an attached plan.

To qualify as a NSTM, the member or components of the member must be in tension and there must NOT be any other member or system of members that will serve the functions of the member in question should it fail. The alternate systems or members represent redundancy.

Tension components of a bridge member consist of components of tension members and those portions of a flexural member that are subject to tension stress. Any attachment having a length in the direction of the tension stress greater than 4 inches, and welded to the tension area of a NSTM shall be considered part of the tension component and, therefore, shall be considered “NSTM”.

NSTMs have all or part of their cross-section in tension. Most cracks in steel members occur in the tension zones, generally at a flaw or defect in the base material. Frequently, the crack is a result of fatigue occurring near a weld, a material flaw, and/or changes in member cross-section. See Appendix for a review of typical fatigue prone details.

After the crack occurs, failure of the member could be sudden and lead to collapse of the bridge. For this reason, steel bridges with the following structural characteristics or components should be reviewed for a NSTM Inspection.

- Two-truss through bridges
- Low two-truss bridges (pony trusses)
- Deck two-truss bridges
- Thru-girder bridges
- Two-girder bridges
- Tied arch bridges
- Movable bridges
- Steel pier caps and cross-girders
- Pin and hanger system on two- or three-girder systems

See Appendix for examples of NSTM bridges, components, bending definitions, typical crack locations, and typical pin and hanger parts.

Inspection Plan

The inspection plan is the final step in preparation for the field inspection. From the information gathered, a plan needs to be organized for the field inspection. A pre-inspection visit to the site may be required to finalize the inspection plan.

Visual inspection is intended to be the primary examination method of Wisconsin's NSTM Inspection policy. This policy requires that each NSTM member or member component be inspected "hands-on", a maximum distance of one arm's length for the entire length of the member and/or member component.

An inspection plan may include some or all of the following and must be prepared prior to the field inspection:

- A brief historical fact statement
- Essential plans that would help with field inspection
- Identification of NSTMs on inspection form or attached plan
- Access equipment and personnel needed to perform the field inspection
- Inspection tools and safety equipment needed to perform the field inspection
- Traffic control requirements
- Estimate of inspection time
- Coordination with and notification of owner and other agencies

To meet the minimum requirements of this policy, all of the required information shall be noted in appropriate locations of the NSTM bridge inspection report form. On larger or more complex structures, it may be necessary to create chapters for each of the required areas of the inspection plan, which can be attached to the NSTM bridge inspection report form.

Field Inspection Procedures, Reports, and Condition Ratings

Field inspection procedures are the implementation of an inspection plan. Good preparation will increase the quality of the field inspection and ensure that all needed tools, safety devices, and operational procedures are available to effectively and efficiently complete the task. It is critical that the Inspection Team Leader guide the field inspection process to assure that each inspection is given the desired level of quality and assures public safety.

Historical Review

The construction history, along with any rehabilitation and maintenance history should be reviewed at the bridge site prior to performing an inspection. This will be helpful in possibly defining deficiencies that may be found during the inspection. It also is advantageous to know the age of possible deficiencies to determine their criticality when making final recommendations.

NSTM Plan Review

Bridge orientation should be evaluated to determine the location of NSTMs. Location of panel points in relation to orientation of the bridge should be determined.

The NSTMs that have been identified on the inspection plan should be reviewed prior to performing the inspection. The inspector shall analyze the types of deficiencies that may be expected from the bridge details, risk factors, and potential issues in formulating their Inspection procedures and plan. Also, the inspector shall determine if any of the repairs or rehabilitation since original construction may have influenced the deterioration of a particular member or connection. Possible locations for potential cracking should be identified and highlighted on the inspection plan.

Traffic Control

Traffic control requirements should be reviewed prior to performing an inspection to assure safety of the inspection team and the traveling public. The Inspection Team should review the traffic control requirements with persons performing the traffic control, if they are not part of the Inspection Team. The traffic control requirements shall be documented on the NSTM bridge inspection report form. Any unsafe traffic control conditions should be corrected before performing an inspection.

Access Equipment and Procedures

Access equipment must be evaluated to determine if it can provide the required visual "hands-on" inspection of all NSTMs.

Typical methods of access available include but are not limited to:

- Deck-parked under-bridge inspection units (Snooper, Reach-All, etc.)
- Ground-parked aerial lifts (manlift, etc.)
- Scaffolding and staging
- Boats
- Ladders
- Climbing/Rope access

Access equipment, safety features, and procedures need to be evaluated prior to inspection. The safety sheet provided with the equipment, including emergency evacuation procedures, should be reviewed.

The Wisconsin Department of Transportation (WisDOT) can offer specialized equipment and services that may be required on certain bridges for access. One of the most useful pieces of equipment available is an under bridge inspection unit. Only trained WisDOT personnel are authorized to operate these units.

Personal Safety

All personal safety equipment needed for an inspection shall be identified and checked for condition. Such equipment may include, but is not limited to, high visibility clothing, body

harnesses, hard hats, safety shoes, eye protection, ear protection, respiratory protection, and protection from hazardous paint or other materials.

Safety of the inspector is essential in providing a quality NSTM inspection. Providing for all the proposed safety requirements is paramount to maintain the confidence of the inspector and to ensure a quality NSTM inspection.

Inspection Tools

A review of the tools listed on the inspection plan should be done, along with a review of the conditions on the bridge, to determine what tools may be required to perform a thorough visual, “hands-on” inspection.

There may be a considerable amount of debris or corrosion in the areas that require inspection. It is critical that all debris and loose corrosion be removed to perform the inspection.

Tools that may be helpful during a NSTM Inspection include:

- Hand-held scraper
- Chipping hammer
- Wire brush
- Drafting brush
- Pocket knife
- Rulers
- Flashlight (or halogen light if power is available)
- Ten-power magnifying glass or crack comparator
- Camera
- Calipers (or D-meter for thickness measurement)

For difficult areas, small power grinders, wire brushes or end grinders may be helpful.

Field Inspection

All NSTMs components identified on the inspection plan must be inspected “hands-on”. The members and/or components shall be cleaned so that all extraneous material is removed to provide for a thorough evaluation.

The condition of each member and/or component must be determined, including any deficiencies such as section loss, cracks, unspecified welds, field welds, tack welds, sharp

bends, kinks or other unspecified damage. For information on how to record the condition of each NSTM, see Part 1.

Specialized NDE that may be required for certain details, or to further analyze defects. The following nondestructive testing services are readily available:

- Ultrasound for analysis of pins, welds and cracks
- Liquid (dye) penetrant for surface flaw analysis
- Magnetic particle for surface or slightly subsurface flaw analysis
- Ultrasound thickness gauge for section loss analysis
- Other NDE services may be available as well.

Deficiencies that require emergency repairs or action shall be reported immediately to the Inspection Program Manager and bridge owner. If the Inspection Team Leader has doubts about the load-carrying capacity of a bridge when such deficiencies are found, they shall take action to close the bridge and make immediate contact with the Inspection Program Manager. Team Leaders do have the authority to close a bridge when, in their judgment, it is unsafe for use. Local law enforcement or state patrol may be called to assist in bridge closure. Only the Inspection Program Manager shall have the authority to reopen a bridge.

2.4.5 Timber Structures

Timber is probably the earliest material ever used to construct a bridge superstructure. The earliest timber superstructure form was likely a tree felled across a stream or ravine. Modern timber can be configured in many different ways, including:

1. **Slabs:** Timber slab bridges are constructed using either glue-laminated or nail-laminated sawn lumber placed longitudinally between supports. The slab acts as a single wide beam spanning from substructure unit to substructure unit. There are no individual beams with this type of bridge, so the slab also acts as the deck. Slabs are used for simple spans of about 35 feet or less and for continuous spans of slightly greater lengths. Glue-laminated slab depths range from 6-3/4 inches to 14-1/4 inches thick, using individual strips of dimensional lumber 3/4 to 2 inches thick to form 42-inch to 54-inch wide panels. Nail-laminated slab depths range from 8 inches to 16 inches deep, using 2-inch to 4-inch dimensional lumber. For maximum strength and stiffness, the lumber width is oriented vertically in the completed structure. Timber slabs may have transverse distributor/spreader beams (Element 8166) attached to their undersides as a method to distribute live loads across the entire bridge width. Steel transverse post-tensioning rods (Element 8165) may also be used for this purpose, as well as to keep the planks in alignment on glue-laminated slabs. See Part 2 Chapter 3 Decks and Slabs for element level inspection of timber slabs.

2. **Solid sawn multi-beams:** Solid sawn multi-beam bridges are constructed using three or more beams as the primary members. Span lengths are limited by the longest available length of solid lumber, so they are usually used for bridge spans from 15 to 30 feet. Typical beam dimensions are 4 to 8 inches wide and 12 to 18 inches deep. Beam spacing is usually on the order of 2 feet on center. Solid wood blocking or bridging is normally placed between the beams to keep the beam in proper alignment. Due to the limited availability of large timbers of this size and the ready availability of high quality glue-laminated beams, solid sawn multi-beam bridges are rarely built nowadays.

Figure 2.4.5-1: Solid Sawn Multi-Beam Bridge.

Figure 2.4.5-2: Solid Sawn Multi-Beam Bridge.

Figure 2.4.5-3: Solid Sawn Multi-beam Bridge.

3. **Glue-laminated multi-beams:** Glue-laminated multi-beam bridges are similar to sawn multi-beam bridges, with the exception that the beams are pre-manufactured members. This is performed by bonding several strips of wood together with a waterproof structural adhesive to form a built-up beam. By using ¾-inch to 2-inch thick strips of wood for the laminations, natural wood defects may be placed in a non-critical location or may be eliminated completely from the final product. The result is a fairly uniform beam with strength properties greater than solid wood of similar dimensions. Standard 3-inch to 14¼- inch wide beams are common, and depths are limited only by transportation and pressure treating considerations. Clear spans up to 150 feet have been attained, though spans less than 80 feet are more common.

4. **Trusses/covered bridges:** Truss bridges are structures with two parallel trusses as the main load-carrying members. Covered bridges are truss bridges with a wood covering to prevent decay of the superstructure. Spans up to 250 feet are attainable. The deck is typically placed between the trusses (through truss when there is overhead lateral bracing or a pony truss when there is no overhead lateral bracing), although it may sometimes be placed on top of the trusses (deck truss). Modern connections are made with steel bolts and gusset plates. Older trusses use wooden peg connections. Truss bridges normally have wood floor systems using floor beams, and usually stringers. To keep the two trusses in line longitudinally, secondary lateral bracing members diagonally connect the bottom chords, and for deck and through trusses, the top chords as well. Lateral bracing may be made of wood, wrought iron or steel. Sway bracing keeps the two trusses in line laterally. On deck and through trusses, sway bracing transversely frames between the truss verticals. On pony trusses, sway bracing is usually placed as a transverse diagonal on the outsides of the trusses. It connects the top chord to transverse “outrigger” floor beam extensions and functions to prevent buckling of the top chord. Sway bracing is normally made out of wood.

5. **Arches:** Modern wood arch bridges are constructed of curved glue-laminated main members. Wood arches use two hinges for spans up to about 80 feet. Spans up to about 300 feet are feasible, and these arches use three hinges. Wood arches are most commonly used as pedestrian bridges, although they have been built for highway use.

2.4.5.1 Timber Open Girder (Element 111)
Timber Stringer (Element 117)
Floor Beam (Element 156), Timber Spreader Beam (Element 8166)

These elements are the primary bending elements of sawn or glulam multi-beam bridges. Timber beams of multi-beam bridges may be recorded as either an open girder or stringer, depending upon the inspector’s interpretation of these terms. Floor beams, most commonly found on wood truss or arch bridges, span transversely between these main load-carrying members. Transverse distributor beams, commonly located under timber slabs, should be considered as timber spreader beams. In the case where a deck is supported by a floor system, timbers that tie into the floor beam shall be recorded as stringers, while timbers that floor beams tie into shall be recorded as girders.

Figure 2.4.5.1-1: Multiple Timber Beams.

Element Level Inspection

On the inspection report form, timber girders/beams and floor beams are recorded in units of lineal feet. The correct method for calculating the beam length is the sum of all of the lengths of each beam section and multiplying by the span length for each span. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of timber girders, stringers, and floor beams should include the following items:

- Checking for member crushing at the abutments and piers. These are the most suspect areas because they tend to collect and retain the most moisture and debris, creating ideal environments for fungal growth and insect attack.

- Looking for shear related damage at and near the supports. Overloads result in high shear stresses that cause horizontal splits to form along the length of the beam, approximately mid-height. Splits will allow fungi and insects access to the untreated interior of a beam.

- Examining the high flexural regions of the beam for signs of overload damage such as crushing near the top surface and transverse cracking near the bottom surface.

- Examining floor beam connections to trusses or arches for splits or checks. This type of damage can seriously weaken the connection.

- Looking for decay at the top of the beam where deck planks are attached. This area is ideal for trapping and retaining moisture, resulting in beam decay.

- Examining the entire member for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.

- Looking for any delaminations of individual wood strips in glulam beams. Because debonding that extends through the beam width changes the original deep, stiff member into two smaller flexible members, this type of deterioration can be especially serious.

- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the beam is tapped with a hammer.

- Look for fire damage, especially near the abutments where fires can be built close to the beams.

- Checking fasteners (nails, bolts, lag screws, deck clips) for corrosion or slipping. Check suspect fasteners for looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Sighting along the length of the beam under traffic loads to look for excessive vertical or lateral deflections. Excessive deflections indicate that the member cannot carry its original design load or that other bridge members are damaged and additional load has shifted to the member in question. The measured or estimated amount of deflection should be recorded.

- Hammer tapping random and suspect areas to evaluate the wood's soundness.

- Performing probe tests in areas suspected to be experiencing decay. See Section 2.3.4.1, Timber Slab (Element 54) for a description of this procedure.

- Drilling or boring suspect planks to estimate the extent of decay.

- Looking for collision damage and reporting this condition under the appropriate Condition State. Signs of impact damage include scrapes on member undersides, chips, cracks, and possibly a permanently displaced or broken member.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shakes/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)

- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance traffic impact damage is observed on the underside of a timber girder. This damage would be recorded in the Timber girder element under defect Decay/Section Loss. The inspector would note in the defect notes that the section loss was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.5.1-2: Timber Beams – Decay/Section Loss Condition State 1.

Figure 2.4.5.1-3: Timber Beams – Checks/Shakes/Cracks/Splits/Delaminations Condition State 1.

Figure 2.4.5.1-4: Timber Beam – Decay/Section Loss Condition State 4.

2.4.5.2 Timber Truss (Element 135)

These elements are primary load-carrying members. Truss members are theoretically loaded in either pure tension or compression. Truss components may be constructed of either sawn lumber of glulam members. Covered bridges are trusses with covers to protect the wood from moisture due to rain, sleet, and snow. Deck dead and live loads are delivered to both trusses and arches by way of floor beams spanning transversely between these main load-carrying members.

Element Level Inspection

This element encompasses all timber truss elements. This includes all tension and compression primary members throughout the truss. Observed distress in truss vertical or diagonal members shall be reported as the length projected along the length of the truss.

On the inspection report form, trusses are recorded in units of lineal feet. A timber truss bridge will have at least two trusses. The correct method to calculate the quantity is to sum of all lengths of each panel as measured longitudinally along the travel way. This element includes all primary components in plane with the truss including, the bottom chord, top chord, verticals and diagonals. For through trusses *or through arches,* the upper bracing (lateral, vertical, portal, and sway) will be evaluated and coded under Assessment 9170 – Truss or Tied/Arch Portal Bracing. The truss components may exhibit more than one Condition State along its length. Distress observed on vertical, diagonal or bracing members shall be reported as the length projected along the length of the truss. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of timber trusses should include the following items:

- Checking the truss bottom chord members for crushing at the abutments. These are the most suspect areas because they tend to collect and retain the most moisture and debris, creating ideal environments for fungal growth and insect attack.

- Examining the entire element for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.

- Looking for any delaminations of individual wood strips in glulam members. Debonding occurring in the vicinity of connectors can be serious if the member is carrying tensile loads.

- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the member is tapped with a hammer.

- Look for fire damage, especially near the abutments and arch bearings where fires can be built close to the primary load-carrying members.

- Checking fasteners (nails, bolts, lag screws, deck clips) for corrosion or slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Sighting along the length of a truss under traffic loads to look for excessive vertical or lateral deflections. Excessive deflections indicate that the member cannot carry its original design load or that other bridge members are damaged and additional load has shifted to the member in question. The measured or estimated amount of deflection should be recorded.

- Performing probe tests in areas suspected to be experiencing decay. See Section 2.3.5.1, Timber Slab (Element 54) for a description of this procedure.

- Drilling or boring suspect members to estimate the extent of decay.

- Looking for collision damage and reporting this condition under the appropriate Condition State. Signs of impact damage include scrapes on member undersides, chips, cracks, and possibly a permanently displaced or broken member.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shakes/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance traffic impact damage is observed on the underside of a timber girder. This damage would be recorded in the Timber girder element under defect Decay/Section Loss. The inspector would note in the defect notes that the section loss was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element

defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.5.3 Timber Arch (Element 146)

These elements are primary load-carrying members. Arches are loaded in combined compression and bending. Arch components may be constructed of either sawn lumber of glulam members. There are several components that make up an arch bridge, such as the arch ribs, spandrel bent columns and caps, cable hangers, built-up hangers, diagonal and vertical braces, and tie girders. This section concerns itself with all of the primary structural arch components.

Element Level Inspection

On the inspection report form, arches are recorded in units of lineal feet. A timber arch bridge will have at least two arches. The correct method to calculate the quantity is to sum of all lengths of each panel as measured longitudinally along the travel way. This element includes all primary components in the plane of the arch including all spandrel components (columns, caps and bracing). The arch components may exhibit more than one Condition State along its length. Distress observed on vertical, diagonal or bracing members shall be reported as the length projected along the length of the arch. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

The above elements include the arch ribs, arch diagonals, spandrel columns, spandrel longitudinal bracing, built-up hangers, and tie girders. Steel arches include both solid and braced ribbed arches. On braced rib arches, the top and bottom chords and the vertical and diagonal longitudinal bracing between them should be considered part of the arch element.

Through arches contain overhead lateral and vertical sway bracing. All lateral and vertical bracing above the roadway on through arches shall be evaluated under the Assessment 9170 – Truss or Tied/Arch Portal Bracing. All lateral bracing below the roadway and not between the arch ribs will be evaluated under assessment 9169 Lateral Bracing. Refer to Chapter 7, Part 2 for additional information on Assessments. The upper bracing supports the arch ribs, which are compression members, preventing the ribs from moving out of plane thus making them important members. Its failure could result in the buckling of the arch rib.

Deck arches have no bracing located above the roadway. All bracing on deck arches shall be evaluated under the arch element. Similar to the bracing between arch ribs, all bracing on a deck arch can be viewed as bracing the compression members (arch ribs). Therefore all the bracing on deck arches is considered primary.

Element Level Inspection of timber arch ribs should include the following items:

- Checking the arch members for crushing at the abutments. These are the most suspect areas because they tend to collect and retain the most moisture and debris, creating ideal environments for fungal growth and insect attack.

- Examining the entire element for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.

- Looking for any delaminations of individual wood strips in glulam members. Debonding occurring in the vicinity of connectors can be serious if the member is carrying tensile loads.

- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the member is tapped with a hammer.

- Look for fire damage, especially near the abutments and arch bearings where fires can be built close to the primary load-carrying members.

- Checking fasteners (nails, bolts, lag screws, deck clips) for corrosion or slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Sighting along the length of an arch under traffic loads to look for excessive vertical or lateral deflections. Excessive deflections indicate that the member cannot carry its original design load or that other bridge members are damaged and additional load has shifted to the member in question. The measured or estimated amount of deflection should be recorded.

- Performing probe tests in areas suspected to be experiencing decay. See Section 2.4.4.1, Timber Slab (Element 54) for a description of this procedure.

- Drilling or boring suspect members to estimate the extent of decay.

- Looking for collision damage and reporting this condition under the appropriate Condition State. Signs of impact damage include scrapes on member undersides, chips, cracks, and possibly a permanently displaced or broken member.

Spandrel Components

Timber vertical spandrel columns of open spandrel arches are primary load-carrying members that support the spandrel bent cap and load the arch ribs.

Spandrel columns are primarily compression members, but they must also resist lateral bending moments due to wind loads, eccentric loading at their tops, overloads, and differential arch deflections.

Element Level Inspection of timber spandrel columns found on arches should include the following items:

- Checking the arch/spandrel column interface for bearing failures. Bearing failures on timber members loaded parallel to the grain will “broom out”.

- Checking the mid-height of the spandrel column for flexural cracks, which are a sign of structural overloads or differential arch deflection.

- Examining the entire element for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.

- Looking for any splitting of sawn timber members. Excessively long or wide splits may be a sign of a structural overload.

- Looking for any delaminations of individual wood strips in glulam members.

- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the member is tapped with a hammer.

- Looking for fire damage, especially near the arch bearings where fires can be built close to the primary load-carrying members.

- Checking fasteners (bolts, lag screws) for corrosion or slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Sighting along the length of the bridge under traffic loading to look for out-of-plumbness. The column should also be sighted along its length to check for bowing. Excessive deflections indicate that the member has been overstressed or that the bridge is experiencing differential settlements. The measured or estimated amount of deflection should be recorded.

- Performing probe tests in areas suspected to be experiencing decay. See Section 2.4.4.1, Timber Slab (Element 54) for a description of this procedure.

- Drilling or boring suspect members to estimate the extent of decay.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shakes/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance traffic impact damage is observed on the

arch rib of a through arch near the roadway. This damage would be recorded in the Timber Arch element under defect Decay/Section Loss (1140). The inspector would note in the defect notes that the distortion was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.6 Masonry Structures

The only masonry bridge superstructure form is the arch. Masonry arches have been used for building and bridge construction since ancient times (notably a 3,400-year-old Babylonian arch), and current use of some of these structures is a testament to their durability. This fact is even more remarkable when one considers that unlike modern concrete arches, stone masonry arches are not strengthened with reinforcing steel.

2.4.6.1 Masonry Arch (Element 145)

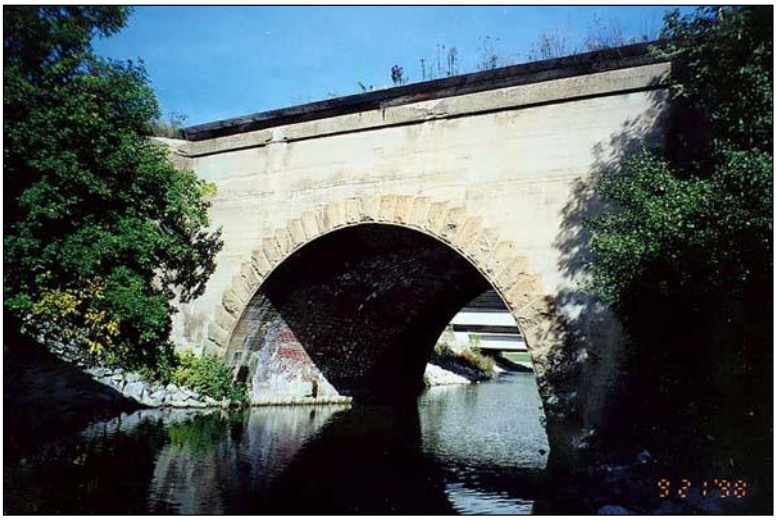

Figure 2.4.6.1-1: Masonry Arch.

In Wisconsin, most stone masonry arches span over small rivers or streams. Arches are primary elements that receive both compressive and bending moments. Since an arch carries a high degree of compressive load, there should be little, if any, net tension along its cross-section. Because of this, there should be no cracking at any of the masonry mortar joints due to bending moments.

Masonry arches are closed spandrel structures that have a single, solid barrel forming the primary load-carrying member. Fill material is always placed on top of the arch to support the roadway, and spandrel walls are used to retain this fill. Spandrel wall failure would cause the fill to spill out, resulting in roadway settlement. For Element Level Inspection purposes, spandrel walls shall be considered primary members and part of the arch element. An important point to note is that masonry arch structures will always have element 8325 “Roadway Over Structure”. This element is discussed in Part 2, Chapter 6.

Element Level Inspection

On the inspection report form, arches are recorded in units of lineal feet. The correct method to calculate the quantity is to measure along the centerline of the roadway above, not along the length of the barrel. The arch is measured from spring line to spring line. The spandrel walls that fall under the projected vertical limits of the spring line are to be assessed under the masonry arch element. Any additional length beyond the spring lines shall be assessed under the appropriate substructure element. Distress observed within the barrel of the

masonry arch shall be reported as the length projected along the travel way. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of masonry arches should include the following items:

- Examining the bearing areas for signs of crushed masonry, since the highest compressive forces experienced by an arch are found at the spring line. Missing or crushed masonry units result in a loss of arch cross-sectional area, increasing the axial stresses.

- Looking for crushed or missing masonry units and mortar. This would suggest a possible overload. Missing or crushed mortar results in a loss of arch cross-sectional area, increasing the axial stresses.

- Checking the arch and spandrel wall surfaces for bulges. This defect suggests unstable soil and the roadway above will also likely show signs of settlement. A bulge or flatness in the arch indicates that it is not functioning properly. Significant areas of bulging should be reported immediately to the Inspection Program Manager.

- Looking for cracked, broken, or deteriorated masonry units and mortar. This would suggest weathering due to freeze/thaw effects.

- Checking the entire arch for transverse mortar cracks. These are the result of excessive bending moments or arch support settlements.

- Looking for leaching along the entire arch and the spandrel walls. This indicates water is flowing through the mortar joints and leaching out the cement minerals. Long-term leaching will weaken the mortar.

- Checking areas exposed to drainage and roadway runoff. The runoff may cause scaling of the masonry units.

- Checking to make sure weep holes in the arch are functioning.

- Checking to make sure surface drains are functioning properly and are not allowing water to penetrate the fill.

- Examining previous repair areas for soundness.

- Examining connections of soil reinforcing bars. These are threaded rods that are installed through the face of the barrel wall. Fasteners should be tapped by a hammer and checked by hand to verify they are firmly installed.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Mortar Breakdown (Masonry) (1610)
- Splits/Spall/Patched Area (1620)
- Masonry or Panel Displacement (1640)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance traffic impact damage is observed on the fascia of a masonry arch near the roadway. This damage would be recorded in the Masonry Arch element under defect Split/Spall/Patched Area (1620). The inspector would note in the defect notes that the section loss was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements as necessary.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.7 Other Material Superstructures

This section is the catch-all for all other superstructure primary members comprised of construction materials that cannot be captured under materials previously covered. These materials may include plastic composites or fiber reinforced polymers.

2.4.7.1 Other Closed Web/Box Girder (Element 106)
Other Open Girder (Element 112)
Other Stringer (Element 118)
Other Floor beam (Element 157)

These are primary bending elements for bridges. Closed web/box girders and open girders are longitudinal main members spanning between substructure units. They may sometimes be referred to as beams rather than girders. Stringers are small longitudinal members that span between the floor beams. Floor beams, in turn, span transversely between the main longitudinal girders.

These bending elements are normally rectangular or “tee”-shaped members. On tee-shaped beams, the top flange also functions as the bridge deck. The cross-section of a closed web/box girder may contain several cells, rather than forming a single box shape (if the element comprises the full width of the superstructure Element 65 Other Material Slab should be used). These bridge types may be thought of as a series of “I”-shaped girders lined up side-by-side. As with tee beams, the top flanges of box girders function as the bridge deck.

Element Level Inspection

On the inspection report form, other girders, stringers, and floor beams are recorded in units of lineal feet. Each element may exhibit more than one Condition State along its length. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

Since each of these elements are bending members, the inspector should expect to find transverse flexural cracks on the top or bottom surfaces in the high moment areas. Diagonal shear cracks on the sides of these elements may also be found at the abutments and piers.

The interior and exterior surfaces of box girders must be inspected in order to properly assign Condition States to the member.

Element Level Inspection of other material closed web/box girders, open girders, stringers, and floor beams should include the following items:

- Checking the entire member for signs of corrosion, as indicated by rust stains or freckled surface rust.
- Inspecting the member for cracks. The inspector should look for transverse flexural cracks on the underside of the beam between supports and on top of the deck over

the piers on continuously supported bridges. Cracks wider than hairline in the flexural region of beams may indicate a serious structural overload.

- Examining the support areas for shear cracks. Shear cracks will be diagonal, extending up from the bearing towards mid-span. Maximum crack widths should be measured and noted on the bridge inspection report.

- Looking for delaminations, spalls, or patching of material. In FRP material looking for blistering (“surface bubbles”), discoloration, or any other type of material deterioration. These defects are an indication of distressed material and may compromise the structural integrity of the member.

- Looking for material deterioration in the form of discoloration, debonding, fraying, etc. For other materials, it may be in the inspector’s best interest to refer to the texture of the material as this may degrade over time.

- Investigating the bearing areas for spalling, crushing, or wrinkling (FRP) to friction from thermal movement or overloads.

- Checking the alignment of the members, verifying there is no bowing, swaying, or other undesirable orientation or movement of the superstructure members.

- Checking the member under drains or leaking expansion joints for cracks, delaminations, rust staining, or efflorescence. Prolonged exposure to moisture may deteriorate the material.

- Checking previously repaired areas for soundness by hammer tapping.

- Looking for collision damage. Fascia beams are most susceptible to vehicular hits. The inspector should take note of the bottom flange to look for any section loss or scraping.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Area (1080)
- Deterioration (1220)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the bottom flange of a beam is struck by vehicular traffic and exhibits deformation out-of-pane. The defect would be reported under Distortion (1900) with the note indicating the deformation was caused by traffic impact.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.7.2 Other Truss (Element 136)

This element should be used when a truss is comprised of a construction material not previously covered. This element is not commonly used.

This element includes all primary truss members, including verticals, diagonals, and bottom and top chords. This element should be used for through trusses and deck trusses. The element should be rated on member conditions regardless of protective coating systems.

Element Level Inspection

On the inspection report form, trusses are recorded in units of lineal feet. A truss bridge will have at least two trusses. The correct method to calculate the quantity is to sum of all lengths of each panel as measured longitudinally along the travel way. This element includes all primary components in plane with the truss including, the bottom chord, top chord, verticals and diagonals, as well as bracing member above the roadway on through trusses. The truss components may exhibit more than one Condition State along its length. Distress observed on vertical, diagonal or bracing members shall be reported as the length projected along the length of the truss. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Through trusses contain overhead lateral and vertical sway bracing. For through trusses *or through arches,* the upper bracing (lateral, vertical, portal, and sway) will be evaluated and coded under Assessment 9170 – Truss or Tied/Arch Portal Bracing. All lateral bracing below the roadway will be evaluated under assessment 9169 Lateral Bracing. Refer to Chapter 7, Part 2 for additional information on Assessments. The upper bracing supports the compression members of the truss stabilizing thus making them important members. Its failure could result in the buckling of the compression chord of the truss.

Deck trusses have no bracing located above the roadway. All bracing on deck trusses shall be evaluated as assessment 9169 Lateral Bracing. Refer to Chapter 7, Part 2 for additional information on Assessments.

Truss bottom chord members are located below the deck, exposing them to water, deicing chemicals, roadway debris, and occasionally drift impact from streams below during high water events. The top chord, vertical, and diagonal members are exposed to a less severe environment (except for deck trusses), but are still susceptible to traffic impact and water and deicing agents.

If multiple defects are found within the same unit of measure, the inspector shall record the predominant defect on the inspection report and describe other defects within the notes under the element. The predominant defect is determined first by Condition State. If Condition States of the overlapping defects are equal, the hierarchy then falls to the lowest associated defect number.

Element Level Inspection of other material trusses should include the following items:

- Checking the truss bottom chord members for crushing at the abutments. These are the most suspect areas because they tend to collect and retain the most moisture and debris.

- Looking for any delaminations, spalls, patched areas, or debonding (FRP). Debonding occurring in the vicinity of connectors can be serious if the member is carrying tensile loads.

- Look for fire damage, especially near the abutments and arch bearings where fires can be built close to the primary load-carrying members.

- Checking all members and fasteners for corrosion. The inspector should pay close attention to horizontal prominences with fasteners that can capture debris and moisture.

- Checking fasteners for slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Checking the alignment and orientation of members and taking note of any members undergoing distortion. Distortion may be an indication that the member is being overloaded or eccentrically loaded unintentionally.

- Sighting along the length of a truss under traffic loads to look for excessive vertical or lateral deflections. Excessive deflections indicate that the member cannot carry its original design load or that other bridge members are damaged and additional load has shifted to the member in question. The measured or estimated amount of deflection should be recorded.

- Checking for any type of material deterioration (discoloration, wearing, or fraying). These defects are an indication of material distress and may compromise structural integrity.

- Looking for collision damage and reporting this condition under the appropriate Condition State. Signs of impact damage include scrapes on member undersides, chips, cracks, and possibly a permanently displaced or broken member.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Area (1080)
- Deterioration (1220)

- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the end post of a truss is struck by vehicular traffic and exhibits deformation out-of-pane. The defect would be reported under Distortion (1900) with the note indicating the deformation was caused by traffic impact.

If excessive debris is present on the flanges, cleaning should be recommended under *Maintenance Actions*.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.7.3 Other Arch (Element 142)

This element should be used when an arch is comprised of a construction material not previously covered. The other arch element includes the arch ribs, arch diagonals, spandrel columns, spandrel longitudinal bracing, built-up hangers, and tie girders. Lateral bracing between ribs (not of braced ribs) should be assessed under the Lateral Bracing element (9169). This element is not commonly used.

The element should be rated on member conditions regardless of protective coating systems.

Element Level Inspection

On the inspection report form, arches are recorded in units of lineal feet. An arch bridge will have at least two arches. The correct method to calculate the quantity is to sum of all lengths of each panel as measured longitudinally along the travel way. This element includes all primary components in the plane of the arch including all spandrel components (columns, caps and bracing). The arch components may exhibit more than one Condition State along its length. Distress observed on vertical, diagonal or bracing members shall be reported as the length projected along the length of the arch. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Through arches contain overhead lateral and vertical sway bracing. For through trusses *or through arches,* the upper bracing (lateral, vertical, portal, and sway) will be evaluated and coded under Assessment 9170 – Truss or Tied/Arch Portal Bracing.. All lateral bracing below the roadway and not between the arch ribs will be evaluated under assessment 9169 Lateral Bracing. Refer to Chapter 7, Part 2 for additional information on Assessments. The upper bracing supports the arch ribs, which are compression members, preventing the ribs from moving out of plane thus making them important members. Its failure could result in the buckling of the arch rib.

Deck arches have no bracing located above the roadway. All bracing on deck arches shall be evaluated under the arch element. Similar to the bracing between arch ribs, all bracing on a deck arch can be viewed as bracing the compression members (arch ribs). Therefore all the bracing on deck arches is considered primary.

Element Level Inspection of other arch ribs should include the following items:

- Checking all primary members for transverse flexural cracking.
- Checking the arch members for crushing at the abutments. These are the most suspect areas because they tend to collect and retain the most moisture and debris.
- Looking for any delaminations, spalls, patched areas or debonding (FRP). Debonding occurring in the vicinity of connectors can be serious if the member is carrying tensile loads.

- Look for fire damage, especially near the abutments and arch bearings where fires can be built close to the primary load-carrying members.

- Checking all members and fasteners for corrosion. The inspector should pay close attention to horizontal prominences with fasteners that can capture debris and moisture.

- Checking fasteners for slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Checking the alignment and orientation of members and taking note of any members undergoing distortion. Distortion may be an indication that the member is being overloaded or eccentrically loaded unintentionally.

- Sighting along the length of an arch under traffic loads to look for excessive vertical or lateral deflections. Excessive deflections indicate that the member cannot carry its original design load or that other bridge members are damaged and additional load has shifted to the member in question. The measured or estimated amount of deflection should be recorded.

- Checking for any type of material deterioration (discoloration, wearing, or fraying). These defects are an indication of material distress and may compromise structural integrity.

- Looking for collision damage and reporting this condition under the appropriate Condition State. Signs of impact damage include scrapes on member undersides, chips, cracks, and possibly a permanently displaced or broken member.

Spandrel Components

Vertical spandrel columns of open spandrel arches are primary load-carrying members that support the spandrel bent cap and load the arch ribs.

Spandrel columns are primarily compression members, but they must also resist lateral bending moments due to wind loads, eccentric loading at their tops, overloads, and differential arch deflections.

Element Level Inspection of spandrel columns found on arches should include the following items:

- Checking the arch/spandrel column interface for bearing failures. Bearing failures on timber members loaded parallel to the grain will “broom out”.

- Checking the mid-height of the spandrel column for flexural cracks, which are a sign of structural overloads or differential arch deflection.

- Examining the entire element for signs of deterioration or discoloration.

- Looking for any delaminations, spalls, or debonding of the material.

- Looking for fire damage, especially near the arch bearings where fires can be built close to the primary load-carrying members.

- Checking fasteners (bolts, lag screws) for corrosion or slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Sighting along the length of the bridge under traffic loading to look for members aligned out-of-plumb. The column should also be sighted along its length to check for bowing. Excessive deflections indicate that the member has been overstressed or that the bridge is experiencing differential settlements. The measured or estimated amount of deflection should be recorded.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Area (1080)
- Deterioration (Other) (1220)
- Distortion (1900)

Condition State Commentary

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the rib of a through arch is struck by vehicular traffic and exhibits deformation out-of-pane. The defect would be reported under Distortion (1900) with the note indicating the deformation was caused by traffic impact.

If excessive debris is present on the flanges, cleaning should be recommended under *Maintenance Actions*.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each

defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.7.4 Other Primary Structural Members (Element 8170)

This element is for all other superstructure components that cannot be assigned under the previous superstructure categories described within this chapter (girder, stringer, floor beam, truss, arch, etc.). This element is also independent of material type. That is this element may be used to capture a steel, concrete, timber or other material member if necessary if the member does not fall into the .

Purlins, commonly found on steel deck systems of moveable bridges are flexural members that support a grid deck and bear on the stringers of a superstructure floor system. This member would appropriately be captured using the Other Primary Structural Members element.

Diaphragms of curved bridges shall be captured under this element. Diaphragms on curved superstructures are considered primary load carrying members. Due to the lateral and torsional forces the diaphragm resists keeping the girders in alignment make these members primary and must therefore be captured as an element.

Element Level Inspection

On the inspection report form, other primary structural members are recorded in units of lineal feet. The correct method for calculating the element length is the sum of all of the lengths of each appropriate member. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of other structural members should include the following items:

- Checking the entire member for signs of corrosion, as indicated by rust stains or freckled surface rust.
- Inspecting the member for cracks. The inspector should look for transverse flexural cracks on the underside of beam members between supports. Cracks wider than hairline in the flexural region of beams may indicate a serious structural overload.
- Examining the bearing areas for shear cracks. Shear cracks will be diagonal, extending up from the bearing towards mid-span. Maximum crack widths should be measured and noted on the bridge inspection report.
- Looking for delaminations, spalls or patching of material. In FRP material looking for blistering ("surface bubbles"), discoloration or any other type of material deterioration. These defects are an indication of distressed material and may compromise the structural integrity of the member.
- Examining the entire member for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.

- Investigating the bearing areas for spalling, crushing, or wrinkling (in FRP) to friction from thermal movement or overloads.
- Checking the alignment of the members, verifying there is no bowing, swaying or other undesirable orientation or movement of the superstructure members.
- Checking the member under drains or leaking expansion joints for cracks, delaminations, rust staining, or efflorescence.
- Checking for any type of material deterioration (discoloration, wearing, or fraying). These defects are an indication of material distress and may compromise structural integrity.
- Checking all members and fasteners for corrosion. The inspector should pay close attention to horizontal prominences with fasteners that can capture debris and moisture.
- Checking fasteners for slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.
- Checking previously repaired areas for soundness by hammer tapping.
- Looking for collision damage. Fascia beams are most susceptible to vehicular hits. The inspector should take note of the bottom flange to look for any section loss or scraping.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Area (1080)
- Exposed Rebar (1090)
- Exposed Prestressing (1100)
- Cracking (PSC) (1110)
- Cracking (RC and Other) (1130)
- Decay/Section Loss (1140)
- Check/Shake/Split/Delamination (1150)

- Abrasion/Wear (1180)
- Deterioration (1220)
- Distortion (1900)

Condition State Commentary

Cracks or fractures should be measured and clearly marked and dated when observed to establish a reference for future inspections. Defects that appear to be affecting the structural integrity of the structure or are posing a threat to public safety should be brought up with the Program Manager immediately.

Elements exhibiting damage should report the damage in the note of the report under the defect associated with the damage. For instance, the end post of a truss is struck by vehicular traffic and exhibits deformation out-of-pane. The defect would be reported under Distortion (1900) with the note indicating the deformation was caused by traffic impact.

If excessive debris is present on the flanges, cleaning should be recommended under *Maintenance Actions*.

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. Concrete Condition States are dependent on crack width and spall dimensions and depth.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.8 Bearing Elements

Bearings serve as the interface between the superstructure girders and the substructure. Bearings carry the dead loads and live loads from the girders down to the substructure and also accommodate bridge rotation, expansion, and contraction. Bridge movement can result from temperature changes, substructure movement, live and dead load deflections, wind loads, quick braking of a vehicle, etc. Movable (expansion) bearings accommodate superstructure longitudinal and rotational movements. Fixed bearings accommodate superstructure rotational movements only. Both types of bearings resist transverse and vertical bridge movements.

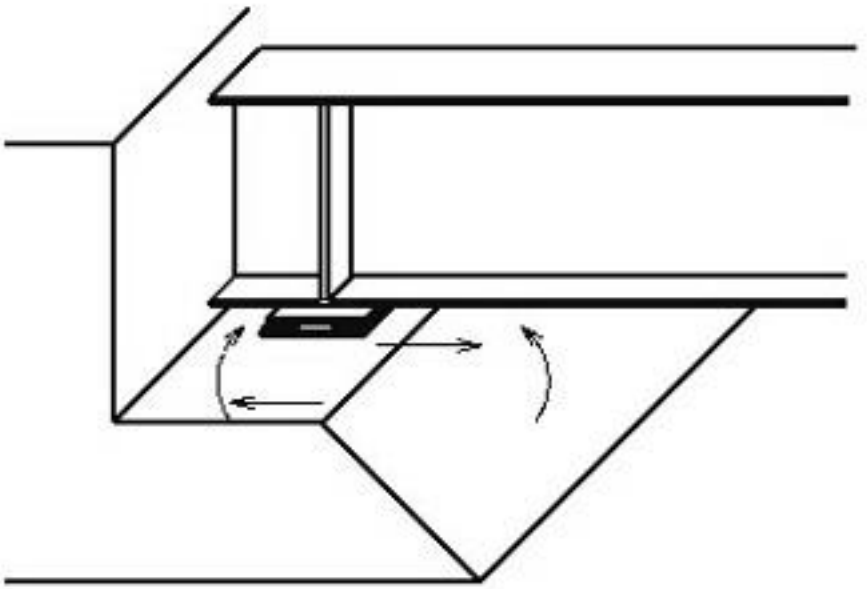

Figure 2.4.6-1: Range of Motion of an Expansion Bearing.

There are several bearing types, but there are four basic bearing elements.

1. **Sole plate:** this is a steel plate attached to the bottom flange of a girder or to a truss member, or may be embedded within a prestressed girder. It serves to transmit the girder reaction force from the girder to the bearing device and may also serve to stiffen the flanges of steel girders.

2. **Bearing device:** bearing devices transmit the girder reaction force from the sole plate to the masonry plate. They can be any of a number of materials and serve several different functions. Expansion bearing devices include rockers, rollers, sliding bronze or steel plates, sliding Teflon plates, thick deformable elastomeric pads, or pot bearings. Fixed bearing devices include bearing shoes, rocker plates, thin non-deformable elastomeric pads or simply a layer of compressible material such as cork or asphalt impregnated joint filler.

3. **Masonry plate:** a steel plate that distributes the forces from the bearing device to the substructure. They are called “masonry” plates because they are placed on top of the concrete masonry of the substructure units. There may be a lead sheet or other material placed below this plate to more evenly distribute the bearing stresses over the possible irregular surface of the concrete.

4. **Anchorage:** longitudinal forces, lateral forces, and sometimes uplift forces act on the bearings. If not restrained, these forces may cause parts of the bearing assembly (most often the masonry plate) to “walk out” from under the bearing device. Anchorages secure the elements of the bearing assembly to the substructure and also to contain bearing components not secured to the substructure from shifting. Most often, the anchorage system consists simply of anchor bolts and nuts to secure the masonry plate. Current American Association of State Transportation and Highway Officials (AASHTO) design standards require that anchor bolts be a minimum of 1 inch in diameter. They may be embedded within the substructure concrete or bolted to steel or timber substructures. Retainer angles or bars may also be used to prevent the bearing device or girder from walking out or moving laterally.

Not all bearings will have all four of the basic bearing elements. As a minimum, however, a compressive filler or thin pad will usually be found between the superstructure and substructure.

2.4.8.1 Elastomeric Bearing (Element 310)

Elastomeric bearings are made of neoprene rubber. Depending on the thickness and construction, they can be used for either fixed or expansion bearings. They may be the only component of a bearing assembly, but sometimes a sole plate is used. Elastomeric bearings are often bonded to the substructure by means of an adhesive and may be vulcanized to the sole plate. They are an attractive choice for a bearing system because they do not corrode and have a long life span if properly designed.

Fixed elastomeric bearings use thin (approximately ½ to 1 inch) rectangular pads made of plain neoprene. They have little lateral movement capability and may be found on short spans or prestressed concrete bridges. **Wisconsin does not code these thin elastomeric bearing pads as a specific element, but assesses their condition as part of the superstructure element.**

Expansion elastomeric bearings are thick rectangular pads that accommodate longitudinal and rotational superstructure movements through shear deformations. Expansion elastomeric bearings may be plain or laminated. Plain pads are used for shorter span bridges with small longitudinal movements and small reaction loads. Under large longitudinal movements, plain pads will deform excessively and their edges will tend to roll off the substructure, resulting in larger bearing pressures and wear. Large reaction loads will cause plain pads to excessively deform vertically, excessively bulge, and possibly split. To guard against these undesirable situations, many expansion elastomeric pads are laminated with steel plates. Laminated pads can be built up higher than 1 inch, which gives them a higher deformation threshold. The steel plates control the amount the elastomeric material is allowed to bulge, thereby controlling vertical deflections. Often, the plates will not be visible because they are encapsulated within the pad with a thin elastomer side covering.

Element Level Inspection

This element encompasses those bearings that are primarily composed of elastomers, with or without metal or fabric reinforcement.

On the inspection report form, bearings are recorded in units of each. The correct method for calculating the total quantity is the sum of the bearing assemblies throughout the structure. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. For a bearing, only one defect will be applied to each bearing assembly. Therefore it is the inspector's responsibility to determine the predominant defect for the entire assembly. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of elastomeric bearings should include the following items:

- Checking the sole plate and/or masonry plate, if any are present, for corrosion. Excessive corrosion may lead to a breakdown or failure of the plate's connection to the elastomeric material.

- Checking for excessive bulging on the sides of the pad. Bulging in excess of about 15 percent of the pad's thickness is a cause for concern. This could be due to excessive vertical loads combined with excessive girder rotation. Bulging on laminated pads should only occur within the individual layers of elastomer.

- Checking for any uplift along the bottom edges. This could be the result of excessive shear, rotation or a combination of both.

- Looking for splits or tears in the pad. These may be oriented vertically or horizontally. If a laminated pad was poorly constructed, the reinforcement layers can start to separate from the elastomer, resulting in horizontal splits. This is a serious condition and should be reported.

- Checking for pad misalignments from its original position. This indicates that it is no longer bonded to the sole plate or substructure, and is beginning to walk out from under the girder.

- Measuring the amount of longitudinal expansion/contraction in expansion bearings. This horizontal measurement is taken between the top and bottom edges of the pad. Longitudinal displacement should have an upper limit of about 25 percent of the bearing pad height. Displacement beyond this limit contributes to high shear stresses and can result in bearing deterioration. An example of the measurement to be taken is shown in Figure 2.4.6.1-1. The ambient temperature at which measurements were taken should be recorded as well.

- Measuring the rotation of the pad if the pad height is vastly different from front to back. The height at the front and back should be recorded, as should the pad length from front to back. An angle of rotation can then be calculated. An example of the measurements to be taken is shown in Figure 2.4.8.1-1.

- Checking for variable thickness in the lateral direction, suggesting lateral rocking of the girder. This would be an unusual occurrence, and signs of distress in other parts of the bridge should be investigated as well.

- Looking for excessive wear at the interface with the sole and/or masonry plates. This may be caused by several expansion/contraction cycles.

- Checking for signs of pad debonding from the sole plate, masonry plate or substructure concrete. The bearing pads should be in full contact with the sole plate and the masonry plate or substructure surface. Should the pad become completely torn from one of these surfaces, it could begin to walk out from under the girder. Out-of-position pads are a potentially serious condition and should be reported to the Inspection Program Manager.

- Checking for cracking or spalling of the abutment and pier surfaces near the bearing pad.

- Checking for any elongation of the pad length at the masonry plate for plain pads. Elongation is the result of increased shear strain that can lead to shear failure. If a plain pad is designed/constructed too thick or if the bearing loads are too high, the pad may exhibit bulging or splits.

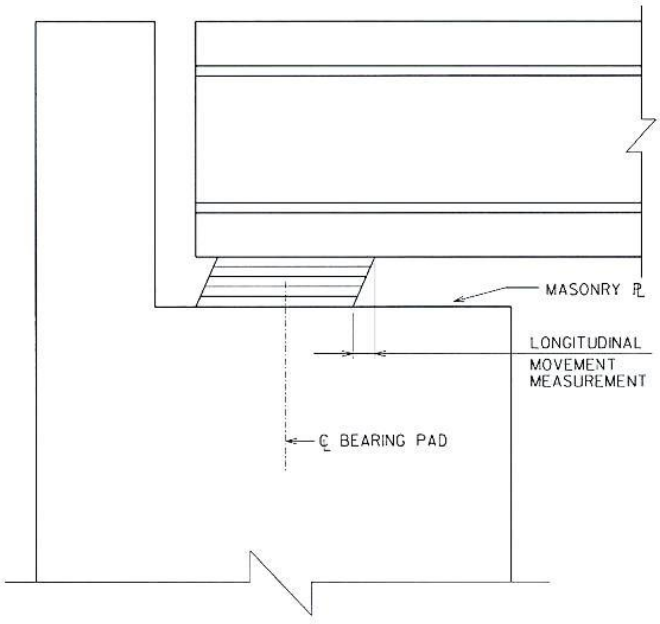

Figure 2.4.8.1-1: Elastomeric Bearing Movement Measurement.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1120)
- Connection (1140)
- Movement (2210)
- Alignment (2220)
- Bulging, Splitting or Tearing (2230)
- Loss of Bearing Area (2240)

Condition State Commentary

The inspector is responsible to carry all necessary equipment to make accurate measurements if necessary. The inspector should make every available attempt to measure the expansion, contraction, rotation or bulging of the bearing.

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.6.1-2: Elastomeric Bearing - Condition State 1.

Figure 2.4.6.1-3: Elastomeric Bearing Experiencing Shear Deformation – Condition State 1.

Figure 2.4.6.1-4: Uplift at Corner of Elastomeric Bearing – Movement Condition State 2.

2.4.8.2 Movable Bearing (Element 311)

There are several types of movable steel bearings that are used to support bridge superstructures. These include rollers, rockers, and sliding plates. Movable bearings are designed to accommodate superstructure expansion, contraction, and rotation. Corrosion is often a problem with movable bearings because they are always used at expansion joints. When these joints leak, water, deicing chemicals, and road debris are allowed to fall directly onto the bearing assemblies.

Rockers

Rockers are steel devices that pivot and roll to accommodate superstructure expansion and contraction. The pivot point/center of rotation is located at the sole plate and longitudinally moves along with the end of the girder. The bottom of a rocker rolls along top of the masonry plate.

Rockers are usually wedge-shaped bearings of variable size. They are used to accommodate large girder reactions in conjunction with large superstructure longitudinal movements. Pintles are often used to hold the rocker in place at the masonry plate. Pintles are short steel pins press fitted into the masonry plate. The pintles fit into oversize holes drilled into the rocker's rolling surface. A pin is sometimes used to connect the rocker to the sole plate. More often, the top stem of the rocker is cast or machined into a cylindrical shape, and the stem fits into a pocket formed in the sole plate. Both pins and stems help to maintain the bearing alignment.

Rollers

Rollers are steel cylinders or segments of cylinders that roll between the sole and masonry plates as the superstructure expands and contracts. Two types of roller bearing assemblies may be found.

1. Roller nest: a nest is a series of small rollers, 1.5 to 2 inches in diameter, assembled together in a fixture. A pin at the top of the fixture allows rotation of the superstructure. Nests often fail because the small diameter rollers offer many places for dirt and debris to accumulate. The dirt and debris will not allow the rollers to roll properly, will trap moisture, and will cause corrosion. Roller nests were widely used around the beginning of the twentieth century, but are no longer used for new construction due to their inherent maintenance problems.

2. Single roller: the single roller is a widely used bearing. A steel cylinder, usually between 6 and 15 inches in diameter, functions as the bearing surface or roller. To keep a roller from walking out over time, flanges are used at their ends that straddle the sole and masonry plates. Pintles press fitted into the roller may also be used. Corrosion on a roller bearing begins at the contact areas between the roller and the plates from trapped dirt, debris, and moisture. When corrosion or proper movement becomes a problem, larger rollers can often be cleaned, rotated 90 degrees, and reused. Smaller rollers tend to corrode uniformly, and must usually be replaced.

Sliding Plates

Sliding plate bearings are typically used on spans under 40 feet. A number of different sliding plate configurations that have been used.

1. Lubricated steel plates: this type of bearing consists of two sliding steel plates, usually the sole plate and masonry plate. After the masonry plate is set in position on top of the substructure, a lubricant consisting of grease, graphite and tallow is applied to the sliding surface. The girder/sole plate is then set on top. The system works well until the lubricant collects dirt and debris. These foreign particles have a tendency to hold moisture, which eventually causes the plates to corrode and bind together, resisting movement. Past attempts to remedy this problem included using a small plate to slide against a larger one, thereby reducing the area of contact available for corrosion. Unfortunately, the smaller plate tended to wear a groove in the larger one. These are older details no longer used in new construction.

2. Lead sheets: a layer of lead sheet was once used between the steel sliding plates to prevent them from corroding together. The lead sheet, however, tended to walk out over time.

3. Asbestos sheet packing: graphite impregnated asbestos sheets have been used in a manner similar to lead sheets.

4. Roofing felt/tar paper: oil-soaked roofing felt or tarpaper has been used in conjunction with graphite to provide movement for small concrete bridges. Several layers of the felt/paper are placed directly on top of the substructure concrete, and the slab or girders are placed directly on top.

5. Bronze bearing plates: bronze plates were used because they corroded less dramatically than steel. However, because bronze is softer than steel and tends to wear faster, dirt and debris between the plates often caused rapid deterioration of the metal, leading to binding.

6. Lubricated bronze plates: lubricant can minimize the problem of mechanical wear and locking up. Dimples cut into a bronze bearing plate act as small pots to hold a graphite lubricant. As the bearing plate wears down, the lubricant is constantly applied to the bearing surfaces. However, the lubricant can eventually wear out and the bearing may still bind over time. As a result, these bearings should be periodically re-lubricated. In Wisconsin, the standard bronze sliding bearing has no dimples. The sole plate has a thin plate of finished stainless steel welded to it that rides over the bronze bearing plate. In order to accommodate rotation, a curved rocker plate is placed beneath the bearing plate, and pintles keep it aligned. Keeper bars and ridges on the rocker plate restrain the bearing plate from walking out of position.

7. PTFE on stainless steel: Polytetrafluoroethylene (PTFE) is commonly known by its trade name, Teflon. Teflon has a very low coefficient of friction, which makes it an ideal material for a sliding bearing. In Wisconsin, the standard PTFE sliding bearing has a thin surface of Teflon bonded to a steel bearing plate. The sole plate has a thin plate of finished stainless steel welded to it that rides over the Teflon. In order to accommodate rotation, a curved rocker plate is placed beneath the bearing plate, and

pintles keep it aligned. Keeper bars and ridges on the rocker plate restrain the bearing plate from walking out of position.

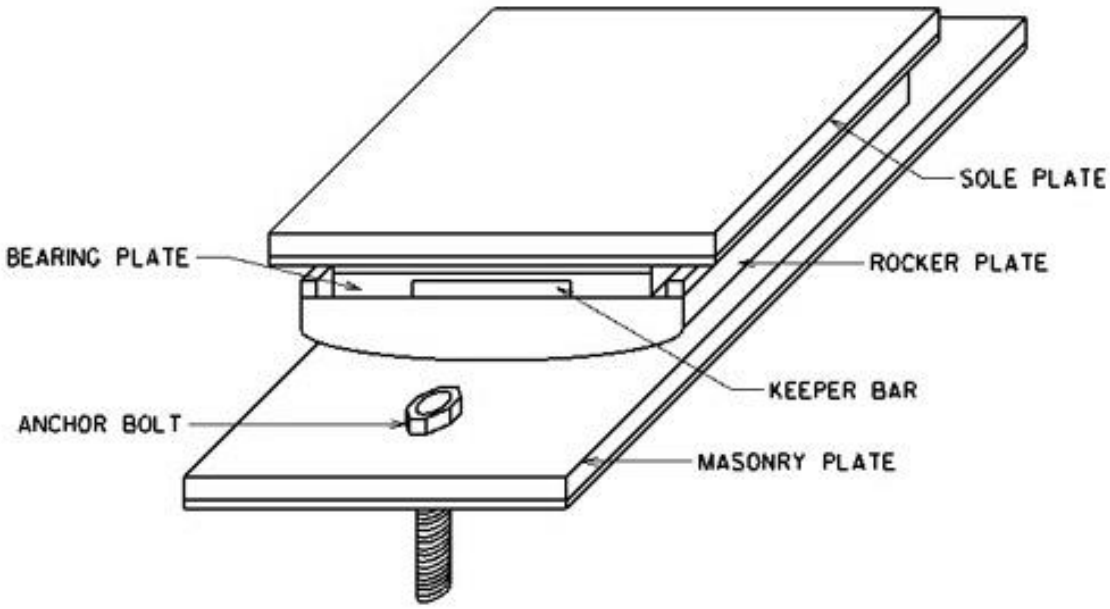

Figure 2.4.8.2-1: Sliding Plate Bearing.

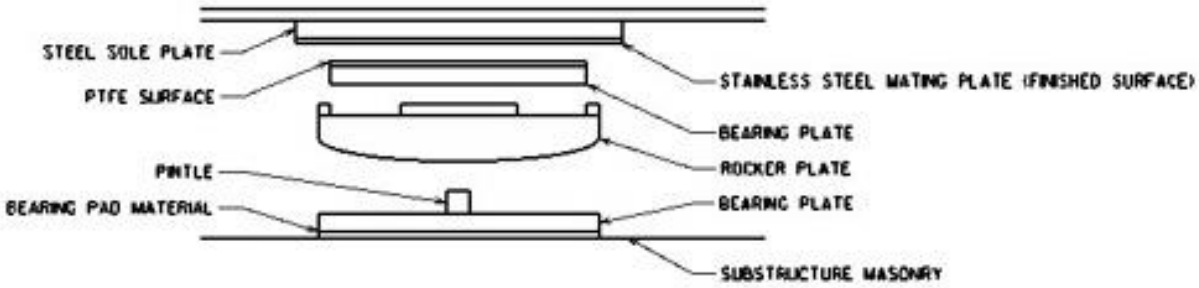

Figure 2.4.8.2-2: PTFE Sliding Plate Bearing Components.

When the main span length is substantially greater than the approach span lengths of continuous bridges, girder ends at the abutments may actually experience uplift forces at the abutments. This could occur when high live loads are placed on the main span but not on the approach spans. It is normally only a problem for steel superstructures because of their lighter dead loads compared to concrete. To prevent physical uplift of the girders at the abutments, restraining or hold-down bearings are used. Restraining bearings consist of large pins inserted through the girder web, with the ends of the pin linked with steel plates to the masonry plate. On large structures such as cantilever trusses or cantilever arches, link bars

may be used at the hold down end of the anchor span. Link bars work in a manner similar to pin and hanger assemblies, but they hold a span down instead of suspending it.

Element Level Inspection

This element encompasses those bearings that provide a superstructure component both rotational and longitudinal movement by means of roller, rocker or sliding assembly.

On the inspection report form, bearings are recorded in units of each. The correct method for calculating the total quantity is the sum of the bearing assemblies throughout the structure. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. For a bearing, only one defect will be applied to each bearing assembly. Therefore it is the inspector's responsibility to determine the predominant defect for the entire assembly. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of movable bearings should include the following items:

- Looking for deteriorated or spalled concrete underneath of the masonry plate. This reduces the bearing area and increases bearing stress on the concrete. It is a common occurrence when the masonry plate has been placed too close to the edge of the substructure unit. Concrete deterioration may also indicate the bearing is not handling lateral forces as intended.

- Looking for anchor bolts/nuts which have risen up above the masonry plate. This is usually caused by water from leaky expansion joints that has migrated into the embedment space of the bolt, froze, expanded, and pushed the bolt upward. Check also for bent anchor bolts.

- Looking for masonry plates that are walking out from underneath of sliding plate bearings. When sliding plates lose their lubrication, substantial longitudinal forces are developed. The forces may be large enough to shear off anchor bolts and push the masonry plate during superstructure expansion and contraction cycles.

- Noting pack rust between sliding plates or between the masonry plate and rocker or roller. Pack rust can freeze up a movable bearing and prevent it from operating properly.

- Noting debris located under a rocker or roller that may be hindering its rolling movement.

- Checking for any broken keeper bars or retainer angles. These deficiencies may allow bearing components to walk out over time.

- Looking for signs of proper movement/wear on sliding plates. On bronze plates, proper movement may be indicated by longitudinal scrape marks or clean bronze. Surface rust ground off steel components of the bearing adjacent to the sliding

surface is also a sign the bearing is operating properly. The lack of these signs suggests a frozen bearing.

- Taking expansion/contraction measurements on bearings. Measurements should be made to the nearest 1/8 inch. Examples of measurements to be taken are shown in Figure 2.4.8.2-3. The ambient temperature at which the expansion/contraction measurement is taken should be recorded as well.

- Checking the bearing assembly for excessive corrosion that may be reducing the cross-sectional area of the bearing device or anchor bolts. Also, excessive corrosion on the bearing surface of masonry plates will inhibit smooth rolling action of rollers and rockers.

- Looking for full and even contact of all bearing components. Gaps or uplift between the bearing surfaces should not be present. A bearing having only partial contact exerts higher stresses on all assembly components. This could result in concrete crushing or steel component buckling.

- Looking and listening for signs of bearing looseness, such as visual movements or rattling under live loads, uplift, and loose or missing fasteners/welds.

- Checking for broken or loose pintles, if visible. This situation suggests excessive superstructure movements (longitudinal or lateral) or substructure settlements.

- Checking for proper bearing alignment. Improper alignments suggest a failing bearing, excessive superstructure movement, or substructure settlement. One sign of improper alignment for any bearing is a superstructure that is tight against the backwall of the abutment. Other signs for specific bearing types are described below.

  a. Sliding plates: there should be no exposure of the sliding surface on the smaller sliding plate. Look for excessive overhang of the top sliding plate over the bottom sliding plate. Improper alignment may allow the girder to slide off the bearing plate on days of temperature extremes. The sole plate of a sliding plate bearing should normally line up with the masonry plate between temperatures of 60 to 70 degrees Fahrenheit, although this could vary for an individual bridge.

  b. Rockers: rockers should be tipped in the proper direction for the ambient temperature. The top of the rocker should be tipped away from the fixed bearing on hot days and towards the fixed bearing on cold days. Rockers are normally set vertical between temperatures of 60 to 70 degrees Fahrenheit, although this could vary for an individual bridge. Excessive tipping in the proper direction for the ambient temperature is also undesirable. Improper alignment may allow the rocker to roll off its bearing surface on days of temperature extremes, allowing the superstructure to drop several inches.

  c. Rollers: the bearing point of rollers on the masonry plate should be located in the proper position for the ambient temperature. The roller should be rolled away from the fixed bearing on hot days and towards the fixed bearing on cold days. Rollers are normally positioned on the centerline of the masonry plate between temperatures of 45 to 65 degrees Fahrenheit. Excessive movement in the proper

direction for the ambient temperature is also undesirable. Improper alignment may allow the roller to roll off of its bearing surface or roll out from under the girder on days of temperature extremes, allowing the superstructure to drop several inches.

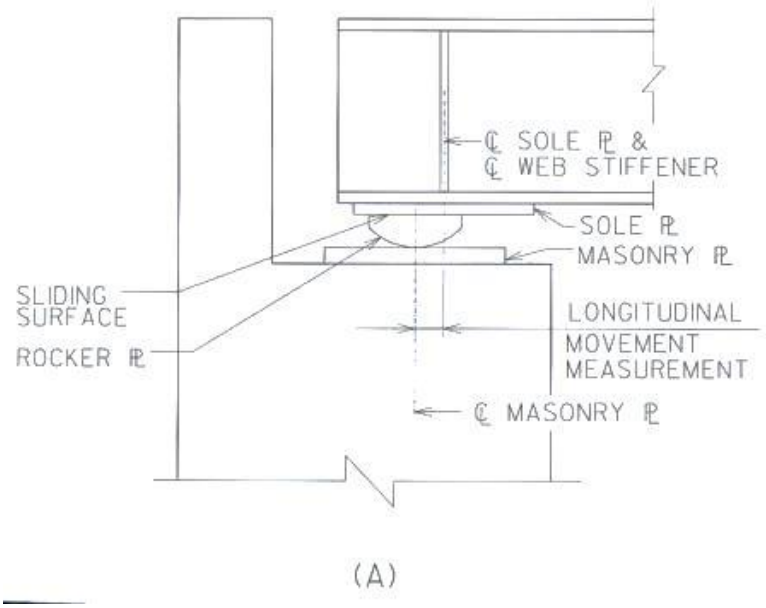

Figure 2.4.8.2-3: Expansion Bearing Movement Measurements.

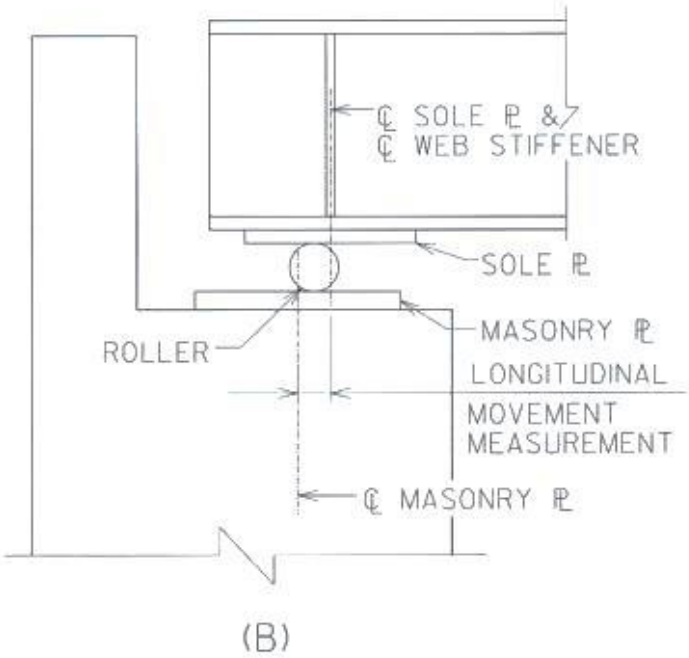

Figure 2.4.8.2-4: Expansion Bearing Movement Measurements.

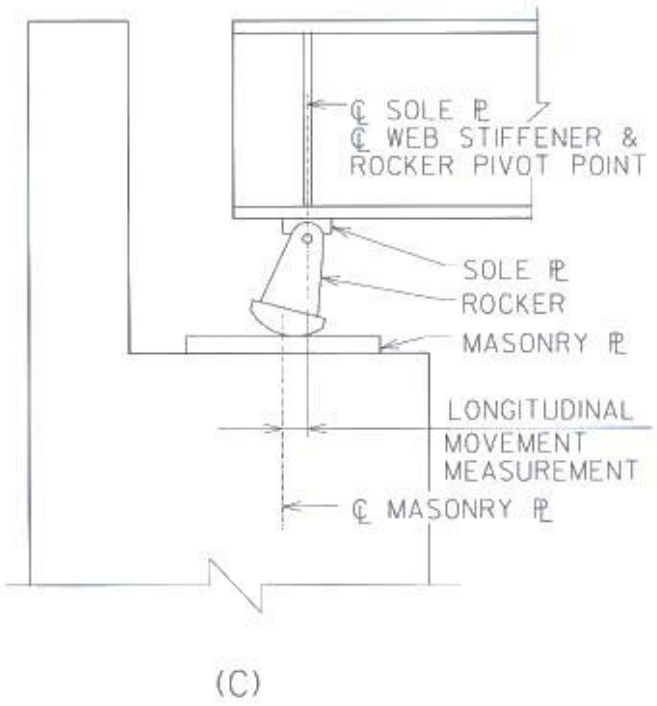

Figure 2.4.8.2-5: Expansion Bearing Movement Measurements.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Connection (1020)
- Movement (2210)
- Alignment (2220)
- Loss of Bearing Area (2240)

Condition State Commentary

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each

defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.8.2-6: Rocker Bearing – Condition State 1.

Figure 2.4.8.2-7: Rocker – Loss of Bearing Area Condition State 2

Figure 2.4.8.2-8: Rocker Bearing – Corrosion Condition State 2.

Figure 2.4.8.2-9: Rocker Bearing – Condition State 2.

Figure 2.4.8.2-10: Hold-Down (Restraining) Bearing - Corrosion Condition State 2.

Figure 2.4.8.2-11: Critically Misaligned Roller Bearing – Alignment Condition State 3.

Figure 2.4.8.2-12: Rocker Bearing – Movement Condition State 3.

Figure 2.4.8.2-13: Masonry Plate Walking Out Due to Failed Anchor Bolt – Connection Condition State 3.

Figure 2.4.8.2-14: Masonry Plate Walkout Due to Failed Anchor Bolt – Connection Condition State 3.

Figure 2.4.8.2-15: Severely Tipped Sliding Bearing – Movement Condition State 3.

2.4.8.3 Fixed Bearing (Element 313)

Fixed bearings prevent longitudinal or transverse movement of the supported element. The sole purpose of a fixed bearing is to transfer load from the superstructure to the substructure while allowing for rotation of the supported element.

Fixed bearings are very simple elements. Those carrying relatively light reaction loads may consist of a sole plate with a curved underside that bears on a masonry plate. For larger bridges and reaction loads, large cast or fabricated fixed shoes are used. Fixed shoes have either a bearing pin or cylindrical shaped stem at their tops. The sole plate has a curved pocket to receive the pin or stem. Fixed shoes widen out at their bases for better load distribution. They are connected to the masonry plate.

Element Level Inspection

This element encompasses those bearings that are fixed and only provide rotation of the superstructure and not expansion and contraction. This element should be used for box girder hinges.

On the inspection report form, bearings are recorded in units of each. The correct method for calculating the total quantity is the sum of the bearing assemblies throughout the structure. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. For a bearing, only one defect will be applied to each bearing assembly. Therefore it is the inspector's responsibility to determine the predominant defect for the entire assembly. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of fixed bearings includes the following items:

- Checking for detachment of the masonry plate or fixed shoe to the substructure. This may be evident by a gap/uplift between the plate and substructure or anchor bolts that are bent, sheared off or pulled out of the substructure concrete.

- Looking for deteriorated or spalled concrete underneath of the masonry plate. This reduces the bearing area and increases bearing stress on the concrete. It is a common occurrence when the masonry plate has been placed too close to the edge of the substructure unit.

- Looking for corrosion on the pins, stem, or bearing surfaces. Advanced corrosion can cause a fixed bearing to resist rotation and may reduce the cross-sectional area of the bearing device or anchor bolts.

- Looking for signs of lateral movement.

- Looking and listening for signs of bearing looseness, such as visual movements or rattling under live loads, uplift, and loose or missing fasteners/welds.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Connection (1020)
- Movement (2210)
- Alignment (2220)
- Loss of Bearing Area (2240)

Condition State Commentary

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.8.3-1: Fixed Shoe Bearing - Condition State 1.

Figure 2.4.8.3-2: Slight Uplift on Masonry Plate – Movement Condition State 2.

2.4.8.4 Pot Bearing (Element 314)

Pot bearings are often used for large curved bridges where the directions for lateral movement and rotation axes are less certain than on straight bridges. They have large bearing capacities and may be designed as fixed bearings as well. Corrosion is often a problem with movable pot bearings because they are always used at expansion joints. When these joints leak, water, deicing chemicals, and road debris are allowed to fall directly onto the bearing assemblies.

Pot bearings allow large multidimensional rotations and lateral movements of the superstructure. Two types of pot bearings may be found.

1. **Neoprene:** These types of pot bearings have several components, including a shallow steel cylinder or hollowed out plate called the pot or pot base, a thin neoprene pad, a steel piston, and a steel top plate. The neoprene pad fits tightly within the pot, and the piston is set on top of the neoprene. The top plate (sole plate), which receives the girder loads, is set on top of the piston. Because of the neoprene pad's tight fit within the pot, deformations are controlled. This allows the pad to withstand much higher loads than an unconfined pad, which would be allowed to bulge. The pad behaves in a manner similar to hydraulic fluid, allowing rotational movements in any direction. Since neoprene pad confinement is critical to the proper operation of a pot bearing, brass seal rings are set between the cylinder and piston. These prevent the neoprene from extruding out of the pot under high loads. To accommodate lateral movements on expansion pot bearings, a Teflon disk is adhered to the top of the piston, and stainless steel is attached to the underside of the top plate.

2. **Spherical:** These types of pot bearings also have several components, including a shallow steel cylinder or hollowed out plate called the pot or pot base, a dished aluminum alloy casting, an aluminum alloy top casting with flat top and spherical bottom, and a steel top plate (sole plate). Teflon sheets are adhered to the curved surfaces of the aluminum alloy castings. The aluminum alloy bottom casting is set into the pot, and the aluminum alloy top casting is set into the bottom casting. The two castings work together like a ball-and-socket joint, accommodating rotational movements in any direction. The top plate sits on top of the top casting. To accommodate lateral movements on expansion pot bearings, a Teflon disk is adhered to the top aluminum alloy casting, and stainless steel is attached to the underside of the top plate.

Element Level Inspection

This element encompasses those bearings that contain confined elastomeric material or configured with a pot base (uncommon spherical pot bearings which do not contain elastomeric material would be assessed under this element). The bearing assemblies may be fixed against horizontal movement, fitted with guide bars to prevent movement in one horizontal direction or floating to allow free horizontal movement depending on the application.

On the inspection report form, bearings are recorded in units of each. The correct method for calculating the total quantity is the sum of the bearing assemblies throughout the structure. Where multiple condition states exist within a unit of measure only the predominant defect in

severity and extent is recorded. For a bearing, only one defect will be applied to each bearing assembly. Therefore it is the inspector's responsibility to determine the predominant defect for the entire assembly. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Due to their limited height and confinement of working parts, pot bearings are often difficult to inspect. However, movement related items can be inspected. Maintenance inspection of pot bearings should include the following items:

- Checking for neoprene pad extrusion above the pot rim, as this indicates serious distress.

- Looking for wear or binding on guide bars. Guide bars are sometimes used on expansion pot bearings to restrict lateral movements in the transverse direction.

- Checking for proper bearing alignment. Improper alignments suggest a failing bearing, excessive superstructure movement, or substructure settlement. One sign of improper alignment is a superstructure that is tight against the backwall of the abutment. There should also be no exposure of the piston top or top surface of the top aluminum alloy casting. Look for excessive overhang of the top sliding plate over the piston or top aluminum alloy casting. Improper alignment may allow the girder to slide off of the bearing plate on days of temperature extremes. The top plate and pot should normally line up between temperatures of 60 to 70 degrees Fahrenheit, although this could vary for any individual bridge.

- Taking expansion/contraction measurements on bearings that are improperly aligned. An example of the measurements to be taken is shown in Figure 2.4.8.4-1. Lateral movements should also be measured. The ambient temperature at which the expansion/contraction measurements are taken should be recorded as well.

- Taking rotation measurements on bearings that are improperly aligned. An example of the measurements to be taken is shown in Figure 2.4.8.4-1.

- Looking for corrosion, pitting, or section loss on the components or bearing surfaces. Advanced corrosion can cause a bearing to resist intended movement and may reduce the cross-sectional area of the bearing devices.

- Checking the bearing components for proper seating and alignment with one another.

- Looking for debris build up that would inhibit proper bearing operation.

- Looking for any cracked welds.

- Looking and listening for signs of bearing looseness, such as visual movements or rattling under live loads, uplift, and loose or missing fasteners/welds.

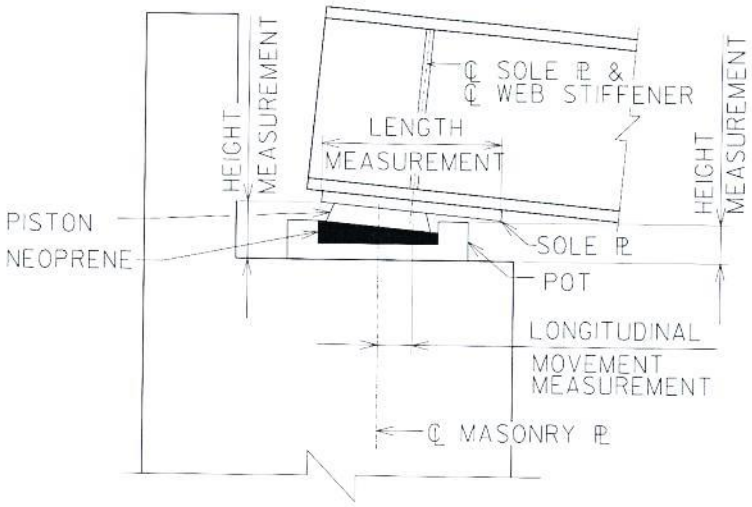

Figure 2.4.8.4-1: Pot Bearing Lateral Movement and Rotation Measurements.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Connection (1020)
- Movement (2210)
- Alignment (2220)
- Bulging, Splitting or Tearing (2230)
- Loss of Bearing Area (2240)

Condition State Commentary

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow

- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.4.8.4-2: Tipped Pot Bearing – Movement Condition State 2.

2.4.8.5 Disk Bearing (Element 315)

Similar to pot bearings, disk bearings are low profile. However, the disk bearing device is different than a pot bearing. A sole plate rests on top of a polyether urethane disk. The disk rests on top of a masonry plate that then distributes the load to the substructure. In fixed applications a shear pin with a rounded tip, fixed to the sole plate extends down through the disk (through an oversized hole in the disc) down into the masonry plate to prevent longitudinal and transverse movement. In multi directional applications, the sole plate is comprised of two plates. The sole plate is anchored to the superstructure with the bottom surface polished (typically stainless steel). Guide plates along the outside edge of the sole plate contain the sliding plate within the assembly. The top surface of the sliding plate contains a PTFE interface. The bottom surface contains the fixed shear pin which extends down through the disk into the masonry plate. Other modifications can be made to disk bearings to allow for high rotation or to dampen large translational movements in case of earthquake events (this configuration will not be found in Wisconsin bridges as it is not considered within a seismic zone).

Element Level Inspection

This element encompasses those bearings that contain a hard plastic disk. The bearing assemblies may be fixed against horizontal movement, fitted with guide bars to prevent movement in one horizontal direction or floating to allow free horizontal movement depending on the application.

On the inspection report form, bearings are recorded in units of each. The correct method for calculating the total quantity is the sum of the bearing assemblies throughout the structure. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. For a bearing, only one defect will be applied to each bearing assembly. Therefore it is the inspector's responsibility to determine the predominant defect for the entire assembly. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Due to their limited height and oft confined working parts, disk bearings are often difficult to inspect. However, movement related items can be inspected. Maintenance inspection of disk bearings should include the following items:

- Looking for deteriorated or spalled concrete underneath the bearing joint. This may reduce the bearing area and increase bearing stress on the concrete. Concrete deterioration may also indicate the bearing is not handling lateral forces as intended.

- Looking for corrosion, pitting, or section loss on the components or bearing surfaces. Advanced corrosion can cause a bearing to resist intended movement and may reduce the cross-sectional area of the bearing devices.

- Looking and listening for signs of bearing looseness, such as visual movements or rattling under live loads, uplift, and loose or missing fasteners/welds.

- Checking for proper bearing alignment. Improper alignments suggest a failing bearing, excessive superstructure movement, or substructure settlement. One sign of improper alignment is a superstructure that is tight against the backwall of the abutment. There should also be no exposure of the shear piston or top surface of the sliding plate. Look for excessive overhang of the sole plate over the sliding plate. Improper alignment may allow the girder to slide off of the bearing plate on days of temperature extremes. The top plate and disk should normally line up between temperatures of 60 to 70 degrees Fahrenheit, although this could vary for any individual bridge.

- Taking expansion/contraction measurements on bearings that are improperly aligned. An example of the measurements to be taken is shown in Figure 2.4.6.4-1. Lateral movements should also be measured. The ambient temperature at which the expansion/contraction measurements are taken should be recorded as well.

- Taking rotation measurements on bearings that are improperly aligned. Follow the example describing pot bearing measurements, shown in Figure 2.4.6.4-1.

- Checking the bearing components for proper seating and alignment with one another.

- Looking for debris build up that would inhibit proper bearing operation.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Connection (1020)
- Movement (2210)
- Alignment (2220)
- Bulging, Splitting, or Tearing (2230)
- Loss of Bearing Area (2240)

Condition State Commentary

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.8.6 Other Bearing (Element 316)

This element should be used when a bearing is found to not compare with the previously stated bearing elements.

This element should not be used to capture enclosed or concealed bearings. If the inspector does happen across enclosed or concealed bearings, no bearing element will be reported on the inspection report. It is the department's discretion that bearings that cannot be seen cannot be inspected and therefore should not be reported on the inspection report.

Element Level Inspection

This element encompasses those bearings that cannot be classified under the bearing descriptions previously mentioned and regardless of permitted movements.

On the inspection report form, bearings are recorded in units of each. The correct method for calculating the total quantity is the sum of the bearing assemblies throughout the structure. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. For a bearing, only one defect will be applied to each bearing assembly. Therefore it is the inspector's responsibility to determine the predominant defect for the entire assembly. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

When confronted with an Other Bearing, the inspector should approach the inspection as any other bearing visual inspection. Maintenance inspection of other bearings should include the following items:

- Looking for deteriorated or spalled concrete underneath the bearing joint. This may reduce the bearing area and increase bearing stress on the concrete. Concrete deterioration may also indicate the bearing is not handling lateral forces as intended.

- Looking for wear or binding on guide bars. Guide bars are sometimes used on expansion pot bearings to restrict lateral movements in the transverse direction.

- Checking for proper bearing alignment. Improper alignments suggest a failing bearing, excessive superstructure movement or substructure settlement. One sign of improper alignment is a superstructure that is tight against the backwall of the abutment. There should also be no exposure of the shear piston or top surface of the sliding plate. Look for excessive overhang of the sole plate over the sliding plate. Improper alignment may allow the girder to slide off of the bearing plate on days of temperature extremes. The top plate and disk should normally line up between temperatures of 60 to 70 degrees Fahrenheit, although this could vary for any individual bridge.

- Taking expansion/contraction measurements on bearings that are improperly aligned. An example of the measurements to be taken is shown in Figure 2.4.8.4-1. Lateral movements should also be measured. The ambient temperature at which the expansion/contraction measurements are taken should be recorded as well.

- Taking rotation measurements on bearings that are improperly aligned. An example of the measurements to be taken is shown in Figure 2.4.8.4-1.
- Checking the bearing components for proper seating and alignment with one another.
- Looking for debris build up that would inhibit proper bearing operation.
- Looking for any cracked welds.

Element Defects

Refer to Appendix A for Defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

- Corrosion (1000)
- Connection (1020)
- Movement (2210)
- Alignment (2220)
- Bulging, Splitting, or Tearing (2230)
- Loss of Bearing Area (2240)

Condition State Commentary

The defects and condition state definitions are based on the AASHTO Manual for Bridge Element Inspection.

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Bridge Inspection Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.4.9 Superstructure NBI Condition Ratings

Part of every Routine Inspection is rating the superstructure according to the Federal Highway Administration (FHWA) General Condition Rating Guidelines. The numeric condition ratings of these guidelines describe existing bridge components as compared to their as-built condition. Ratings range from 9 to 0, with 9 describing components in excellent condition, and 0 describing failed components.

Because only a single number is used to rate the superstructure, the rating must characterize its overall general condition. The rating should not be used to describe local areas of deterioration, such as isolated heavy corrosion or a bent flange due to a traffic impact. However, widespread heavy corrosion or widespread cracked welds would certainly influence the rating. A proper rating will therefore consider deterioration severity plus the extent to which it is distributed throughout the superstructure.

National Bridge Inventory (NBI) ratings are used to evaluate the state of deterioration of the superstructure material. Since material condition is independent of a bridge's design load-carrying capacity, postings or original design capacities less than current legal loads will not influence the rating. Similarly, temporary superstructure support does not change or improve the condition of the superstructure material. Temporary strengthening methods will therefore not influence the superstructure rating.

Decks that are built integral with the superstructure (steel or concrete box girders, decks of reinforced concrete tee beam bridges, etc.) are rated as separate components from the superstructure. In other words, since the superstructure is not part of the deck, the superstructure NBI rating should not influence the deck NBI rating. However, since integral decks form part of the superstructure's cross-section, deck deterioration is essentially the same as superstructure deterioration. Because of this, the superstructure on integral deck bridges should never receive a higher NBI rating than the deck.

On slab bridges, the deck is the same structural component as the superstructure. The FHWA Guidelines specifically state that ratings of decks built integral with superstructures (including slabs) should not be influenced by the superstructure rating. However, since the deck NBI rating accounts for inspection findings on both the top and underside, NBI condition ratings for the deck and superstructure must be the same.

The NBI general condition ratings found in the FHWA guidelines apply to decks, superstructures, and substructures. Rating 9 to 6 apply to components built of any material, while ratings 5 to 0 mention specific defects or deterioration that can be applied to certain materials. Because the NBI general condition ratings apply to a wide range of components and materials, Wisconsin has developed supplemental rating guidelines. These supplemental rating guidelines are used to assist the inspector in properly assigning condition ratings to specific components constructed of the most commonly used materials. The general condition ratings, along with the Wisconsin supplemental rating guidelines for superstructures, are as follows:

Code (Rating)	Description

N **NOT APPLICABLE**

Wisconsin Supplemental Rating Guidelines:

Used for culverts only.

9 **EXCELLENT CONDITION**

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – new condition.

Prestressed Concrete Superstructure – new condition.

Steel Superstructure – excellent condition.

Timber Superstructure – excellent condition.

8 **VERY GOOD CONDITION** – no problems noted.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – there are no noteworthy deficiencies which affect the structural capacity of the members.

Prestressed Concrete Superstructure – no problems noted.

Steel Superstructure – there are no noticeable or noteworthy deficiencies which affect the condition of the superstructure.

Timber Superstructure – there are no noteworthy deficiencies which affect the structural capacity of the members.

7 **GOOD CONDITION** – some minor problems.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – some minor problems. Non-structural hairline cracks without disintegration may be evident. Load-carrying capacity of structural members unaffected.

Prestressed Concrete Superstructure – non-structural cracks less than 1/64 inch (hairline crack) in width may be evident. No rust stains apparent.

Steel Superstructure – some rust may be evident without any section loss.

Timber Superstructure – minor decay, cracking or splitting of beams or stringers at non-critical locations.

6 **SATISFACTORY CONDITION** – structural elements show some minor deterioration.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – structural members show some minor deterioration or collision damage. Hairline structural cracks or spalls may be present with evidence of efflorescence. Minor water saturation marks. Generally, the reinforcing steel is unaffected.

Prestressed Concrete Superstructure – minor concrete damage or deterioration. Non-structural cracks are over 0.015 inch. Isolated and minor exposure of mild steel reinforcement may be present.

Steel Superstructure – rusting evident but with minor section loss (minor pitting, scaling or flaking) in critical areas.

Timber Superstructure – some decay, cracking or splitting of beams or stringers. Fire damage limited to surface scorching with no measurable section loss.

5 **FAIR CONDITION** – all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – structural members are generally sound (structural capacity unaffected) but may have evidence of deterioration or disintegration. Numerous hairline structural cracks or spalls may be present with minor section loss of reinforcing steel possible.

Prestressed Concrete Superstructure – isolated and minor exposure of prestressing strands may be present. There may be structural cracks with little or no rust staining. Primary members are sound, but may be cracked or spalled.

Steel Superstructure – there is minor section loss in critical areas. Fatigue or out of plane distortion cracks may be present in non-critical areas. Hinges may be showing minor corrosion problems.

Timber Superstructure – moderate decay, cracking, splitting or minor crushing of beams or stringers. Fire damage limited to surface charring with minor, measurable section loss.

4 **POOR CONDITION** – advanced section loss, deterioration, spalling or scour.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – there is extensive disintegration. There are measurable structural cracks or large spall areas. Corroded reinforcing steel evident with measurable section loss. Structural capacity of some members may be diminished.

Prestressed Concrete Superstructure – moderate damage or deterioration to concrete portions of the member exposing reinforcing bars or prestressing strands. There is possible bond loss. Structural cracks with medium to heavy rust staining may be present. There may be a loss of camber.

Steel Superstructure – significant (measurable) section loss in critical areas. Fatigue or out of plane distortion cracks may be present in critical areas. Hinges may be frozen from corrosion. Load-carrying capacity of structural members affected.

Timber Superstructure – extensive decay, cracking, splitting or crushing of beams or stringers or significant fire damage. Diminished load-carrying capacity of members is evident.

3 **SERIOUS CONDITION** – loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – there is severe deterioration and/or disintegration of primary concrete members. Large structural cracks may be evident. Reinforcing steel is exposed with advanced stages of corrosion. Local failures or loss of bond are possible.

Prestressed Concrete Superstructure – severe damage to concrete and reinforcing elements of the member. Severed prestressing strand(s) or strand(s) are visibly deformed. Major or total loss of concrete section in the bottom flange. Major loss of concrete section in the web, but not occurring at the same location as concrete section loss in the bottom flange. Horizontal misalignment or negative camber to the member. Unless closely

monitored it may be necessary to restrict or close the bridge until corrective action is taken.

Steel Superstructure – severe section loss or cracking in critical areas. Minor failures may have occurred. Significant weakening of primary members evident.

Timber Superstructure – severe decay, cracking, splitting or crushing of beams or stringers or major fire damage. Load-carrying capacity is substantially reduced. Local failure may be evident.

2 **CRITICAL CONDITION** – advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – advanced deterioration of primary concrete members. There is concrete disintegration around reinforcing steel with loss of bond. Some reinforcing steel may be ineffective due to corrosion or loss of bond. Numerous large structural cracks may be present. Localized failures of bearing areas may exist. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.

Prestressed Concrete Superstructure – critical damage to concrete and reinforcing elements of members. This damage may consist of one or more of the following:

1. Cracks which extend across the bottom flange and possibly into the web that are not closed below the surface damage (this indicates that the prestressing strands have exceeded yield strength).

2. An abrupt lateral offset as measured along the bottom flange or lateral distortion of exposed prestressing strands (this also indicates that the prestressing strands have exceeded yield strength).

3. Loss of prestress force to the extent that calculations show that repair cannot be made.

4. Excessive vertical misalignment.

5. Longitudinal cracks at the interface of the web and top flange that are not substantially closed below the surface damage (this indicates permanent deformation of the stirrups).

Steel Superstructure – severe section loss in many areas with holes rusted through at numerous locations in critical areas.

Timber Superstructure – severe decay, cracking, splitting or crushing of beams or stringers or major fire damage have resulted in significant local failures. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.

1 **"IMMINENT" FAILURE CONDITION** – major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structural stability. Bridge is closed to traffic but corrective action may put it back in light service.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – bridge is closed to traffic. There is major deterioration or section loss present on primary structural elements. Obvious vertical or horizontal movement is affecting the structure's stability. Corrective action may put the bridge back in light service.

Prestressed Concrete Superstructure – critical damage requiring the replacement of a member. Bridge is closed to traffic, and installation of temporary falsework to safeguard the public and the bridge should be taken at the time of the inspection.

Steel Superstructure – bridge is closed. Corrective action may put it back in light service.

Timber Superstructure – bridge is closed. Corrective action may put it back in light service.

0 **FAILED CONDITION** – out of service, beyond corrective action.

Wisconsin Supplemental Rating Guidelines:

Concrete Superstructure – bridge is closed; out of service. It is beyond corrective action; replacement is necessary.

Prestressed Concrete Superstructure – bridge is closed and out of service.

Steel Superstructure – bridge is closed. Replacement is necessary.

Timber Superstructure – bridge is closed. Replacement is necessary.

One suggested method for establishing a superstructure rating is to identify phrases within the general condition/Wisconsin supplemental guideline language that describes a superstructure condition more severe than what actually exists. The correct rating number will be one number higher than the one describing the more severe condition.

For example, suppose a steel superstructure has a failed paint system resulting in extensive corrosion, but without measurable section loss in critical areas. Minor fatigue cracks were found at diaphragm connections, but they only occurred at a few locations. Condition rating 6 indicates that there is minor deterioration and that rust is present but with only minor section loss in critical areas. Condition rating 5 indicates that there is minor section loss on primary elements in critical areas and that fatigue or out of plane distortion cracks may be present in non-critical areas. Even though condition rating 5 describes the presence of fatigue or out-of-plane distortion cracks, it represents a more severe condition than is actually occurring on the superstructure. Cracks in the actual bridge do not generally occur throughout and therefore do not represent the superstructure as a whole. Therefore, a rating of 6 would be appropriate.

Another method to help narrow down the superstructure rating number is to group the numbers in more general categories. Ratings of 9 to 7 apply to superstructures in good condition, 6 to 5 suggest fair condition, 4 to 3 suggest poor condition, 2 suggests poor/critical condition, and 1 to 0 suggest critical condition. It is also important to note that there is a significant change from a superstructure in condition rating 5 (minor section loss, structural elements sound) to condition rating 4 (advanced section loss, advanced deterioration). A reduction in load-carrying capacity can be measured/calculated when a superstructure enters condition rating 4.

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2.5 SUBSTRUCTURE

2.5.1 Introduction

The bridge substructure includes all elements that support the superstructure. Substructure elements deliver the superstructure reaction loads down to the foundation soil or bedrock. In addition, substructures must also control deflections and settlements so as not to create serviceability problems for the riding surface or unintended overloads of the superstructure. There are three main substructure components: abutments, piers, and wingwalls.

2.5.1.1 Abutments

Abutments function to support the ends of the bridge and to retain the soil fill under the approach. Refer to Figure 2.5.1.1-1 for details of common abutment types. Abutments must therefore resist vertical loads from the superstructure (live loads and superstructure self-weight), plus lateral loads due to soil pressure under the approach. Lateral loads may also come from superstructure longitudinal forces due to temperature effects, vehicle braking forces, etc. An abutment is designed to resist these longitudinal forces only if the bearings above are fixed. Unintended superstructure longitudinal forces are delivered to the abutment when abutment expansion bearings have “frozen” due to corrosion or debris accumulation.

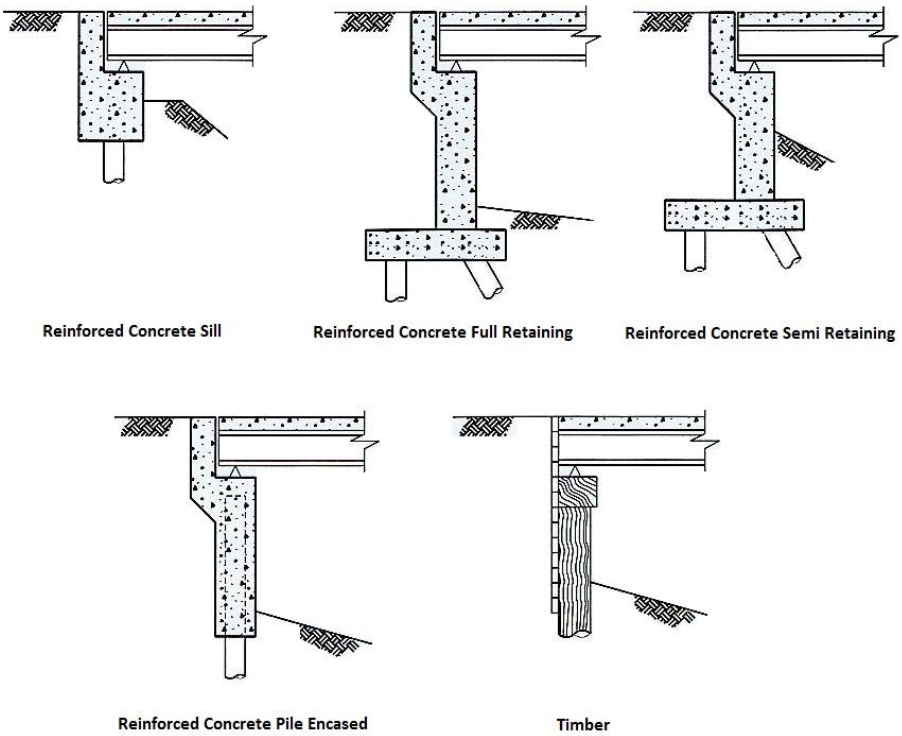

Figure 2.5.1.1-1: Typical Abutments

To resist the loads described above, abutments must act as both compression and bending elements. For shorter abutment heights (less than 6 feet), bending due to lateral soil

pressure is not significant. For taller abutment heights, the bending becomes a significant factor in the structure's safe design.

Almost any type of material may be used to construct an abutment. The most common is reinforced concrete, although masonry, timber, plain (un-reinforced) concrete, and (rarely) steel abutments have been built. Current Wisconsin Department of Transportation (WisDOT) standards detail reinforced concrete and timber as materials to be used for typical highway crossings. Abutment material type is defined by the material retaining the embankment.

To accurately fill out an inspection report, an inspector should know the correct terminology for the various abutment components. These include:

- **Stem/breast wall:** The stem, sometimes called breast wall, is the main body of the abutment. It functions to deliver the superstructure reaction loads to the foundation and to retain much of the soil behind the abutment.

- **Bearing/bridge seat:** The bearing seat, sometimes called bridge seat, is the top surface of the stem/breast wall or cap upon which the bearing devices for the superstructure are placed.

- **Backwall:** Backwalls, located at the ends of the girders, retain the soil under the approach from spilling onto the bearing seat. It may also provide support for concrete approaches and provide anchorage for expansion joint devices.

- **Cheek wall:** Cheek walls are features placed at either end of the abutment to protect the fascia bearings from the elements. They also serve as architectural features to hide the bearings.

Figure 2.5.1.1-2: Reinforced Concrete Sill Abutment

2.5.1.2 Piers

Piers are intermediate support points for a bridge, used mainly for medium to long structures. Piers must resist vertical loads from the superstructure (live loads and superstructure self-weight) and superstructure longitudinal forces due to temperature effects, vehicle braking forces, etc. A pier is designed to resist these longitudinal forces only if the bearings above are fixed. Unintended superstructure longitudinal forces are delivered to the pier when the pier expansion bearings have “frozen” due to corrosion or debris accumulation. To resist the above loads, piers must act as both compression and bending elements. Piers must also resist lateral forces transverse to the bridge centerline. These forces come from wind pressures against the girders, centrifugal effects of traffic on curved bridges, stream flow, etc. Most piers act as cantilever beams to resist loads longitudinal to the bridge centerline. Depending on the configuration of its elements, piers act as frames, cantilevers beams or shear walls to resist loads transverse to the bridge centerline. Refer to Figure 2.5.1.2-1 for details of common pier types.

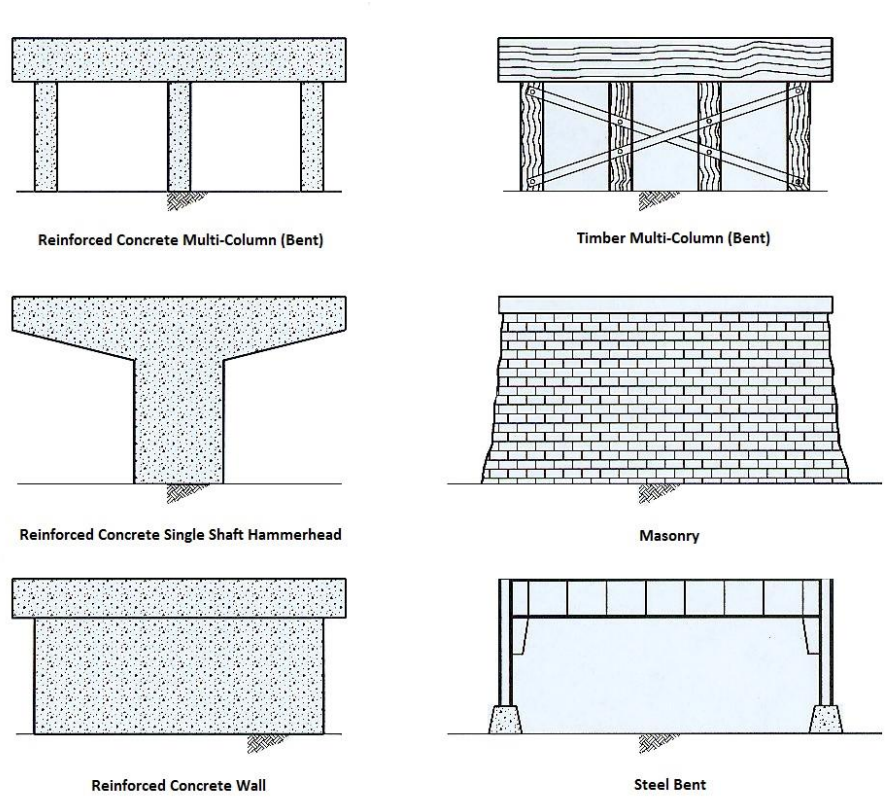

Figure 2.5.1.2-1: Typical Piers

Almost any type of material may be used to construct a pier. The most common is reinforced concrete, although masonry, timber, steel, and (rarely) plain concrete have been built. Current WisDOT standards detail reinforced concrete as the material to be used for typical highway crossings.

To accurately fill out an inspection report, an inspector should know the correct terminology for the various pier components. These include:

- **Pier cap:** A pier cap is the horizontal component of a pier upon which the bearing devices for the superstructure are placed. It also acts to tie the column tops together on multi-column piers to form a frame for resisting loads transverse to the bridge centerline. When used on a multi-column pier, pier caps behave as bending members. When used above pier walls, pier caps are simply an architectural feature formed by thickening the wall, although bending may come into play if the cap cantilevers over the ends of the wall.

  Column: A column differs from a pile in the way that it is supported. A column will be supported by a concrete pedestal or footing that can be beneath the ground level or exposed. A pile will extend well past the ground level into more substantial bearing material, such as bedrock or stone that will provide the substructure with structural stability. If a column's pedestal or footing is buried it may be difficult to tell the difference between a column and a pile without looking at the bridge plans.

- **Solid wall:** A solid pier wall is another component of a pier, many times the only component visible. They are essentially very wide solid shafts of constant thickness that behave as shear walls. They behave as beam/columns to resist axial compressive forces and bending longitudinal to the bridge centerline. They are often used within streams or rivers because they offer less resistance to water flow than multicolumn piers. For recording purposes, a vertical member may be considered a wall (versus a stem) when its height is less than its width.

- **Web wall:** Web walls are the concrete infill between the columns and pier caps of multicolumn piers. Web wall thicknesses are always less than column widths. They are used to change multicolumn pier lateral behavior from a frame to a shear wall,.

- **Hammerhead:** Hammerhead piers are comprised of a horizontal component (similar to a pier cap) of a pier upon which the bearing devices for the superstructure are placed. However, the horizontal members are placed over a single vertical pier stem with a width much smaller than a pier wall. Hammerheads caps are pure bending members that cantilever over either side of the stem. Hammerhead piers are coded as a cap and a column when exposed height is greater than width. If width is greater than the exposed height the hammerhead pier is coded as a cap and pier wall.

- **Stem:** Stems are solid shaft vertical pier components that behave as a cantilever beam/column to resist axial compressive forces and bending longitudinal and transverse to the bridge centerline. For recording purposes, a vertical member may be considered a stem or column (versus a wall) when its height exceeds its width.

- **Crash wall:** Crash walls are placed between the columns of multicolumn piers or between the stems of two individual piers supporting separate bridges. Their purpose is to protect the pier base from rail car, ship or vehicle impacts. Normally, the thickness of a crash wall is the same as the column/stem width to prevent snagging during a collision. For recording purposes, when a crash wall is fully supported by a foundation (footing or piling) and the wall supports columns, the crash wall may be considered a pier wall. If the crash wall is placed between the columns after-the-fact, the crash wall is considered secondary (it does not support the columns but rather braces) and shall not be considered a pier wall.

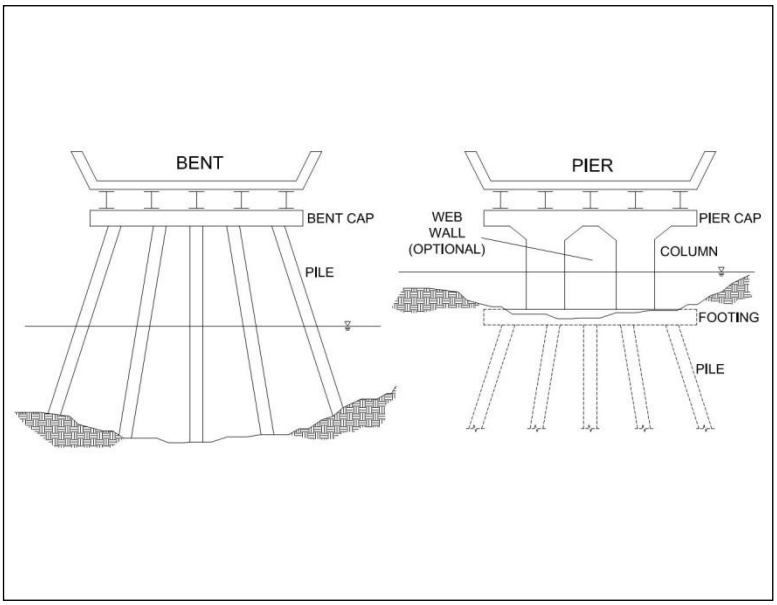

Figure 2.5.1.2-2: Diagram showing differences between a Bent and Pier

Figure 2.5.1.2-3: Reinforced Concrete Columns and Pier Cap

Figure 2.5.1.2-4: Hammerhead Pier

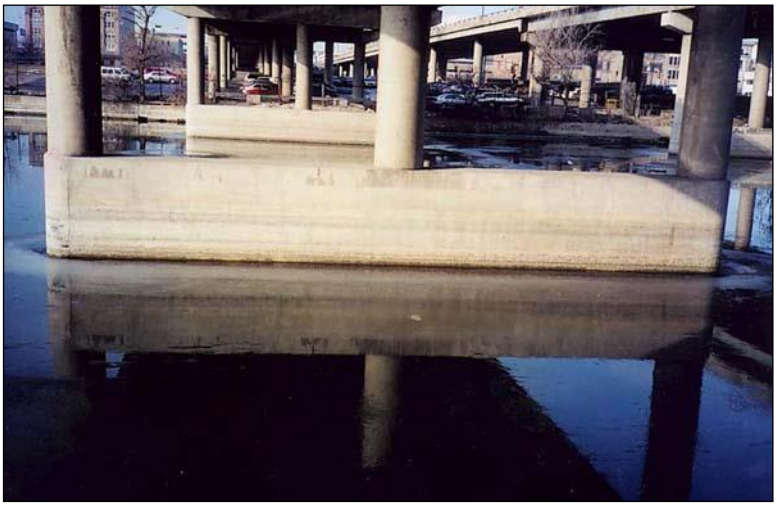

Figure 2.5.1.2-5: Crash Wall

2.5.1.3 Wingwalls

Wingwalls are found at abutment ends to retain and enclose the approach fill. Without them, the approach fill would spill or wash out, causing settlement of the roadway. Wingwalls resist lateral pressures due to the approach fill and carry no vertical loads other than their self-weight. Depending on the original design criteria, three geometries may be used to properly retain the fill. These are straight wings parallel to the abutment, U-wings parallel to the roadway, and flared wings that form an acute angle between both the roadway and abutment. Wingwalls may or may not be rigidly attached to the abutment body.

Almost any type of material may be used to construct a wingwall. The most common is reinforced concrete, although masonry, timber, and steel have been built. Current WisDOT

standards detail reinforced concrete and timber as the material to be used for typical highway crossings.

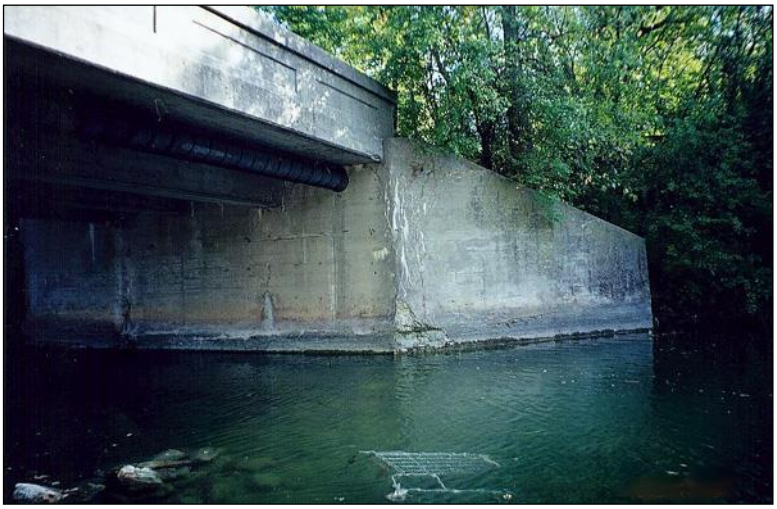

Figure 2.5.1.3-1: Reinforced Concrete Wingwall

For recording purposes, only integral wingwalls should be evaluated as a wingwall. A reinforced concrete wingwall is considered to be integral if it is monolithic with the abutment. Timber and steel wingwalls are considered to be integral wingwalls, even with the presence of a joint at the end of the abutment on flared wingwalls. R-numbered structures (numbered retaining walls) are not considered wingwalls and should be inspected and reported separately from the bridge inspection. Non-integral wingwalls or non R-numbered structures shall be evaluated under the most appropriate retaining wall element.

2.5.1.4 Integral Wingwall (Element 8400)

This element defines the wingwalls integral with the abutment which extend past the bridge seat for parallel wingwalls, or at the skew point when the wingwalls are turned back.

Steel wingwalls and timber wingwalls are considered to be integral wingwalls, even with the presence of a joint at the end of the abutment on flared wingwalls. Steel wingwalls and timber wingwalls are considered monolithic up to the first construction joint (integral plank butt joint, etc.) past the bridge fascia. All other extensions are not to be included within the quantity measurement nor the condition ratings.

Reinforced concrete wingwalls and prestressed concrete wingwalls are considered integral wingwalls when poured monolithic with the concrete abutment. Non-monolithic wingwalls without “R” numbers are considered retaining walls. Therefore, retaining wall elements will be used for the evaluation of these wingwalls. Non-integral wingwalls with “R” numbers will not be coded as part of the bridge inspection.

Masonry wingwalls that are monolithic with the abutment (i.e. have the stones overlapping at the corners) will be considered integral wingwalls. Non-integral wingwalls without “R” numbers are considered retaining walls. Therefore, retaining wall elements will be used for the evaluation of these wingwalls. Non-integral wingwalls with “R” numbers will not be coded as part of the bridge inspection.

Element Level Inspection

On the inspection report form, integral wingwalls are recorded in units of “each”. For each wingwall, the most severe defect Condition State is assigned to the entire element. This will quantify the wingwall’s condition and help to generate quantity/cost estimates for future remedial work.

Element Defect

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Wall Movement (8902)
- Wall Deterioration (8903)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.1.5 Foundation Types

Three foundation types are used to support substructure elements. Piles are by far the most common of the three in Wisconsin. Piles are structural members that transmit all of the bridge live and dead loads into the underlying soil or bedrock. They are often used when soils immediately below the substructure unit are inadequate to resist the bearing pressures or satisfy settlement criteria. They are driven into the ground with a pile driver and rely on soil friction and/or end bearing to deliver the bridge loads into the earth. Piles may be driven vertically or at a batter (angled) to resist lateral loads. Materials used for piles include steel,

reinforced concrete, timber, and prestressed concrete. In Wisconsin, steel H-piles and pipe piles filled with concrete (cast-in-place or CIP piles) are the most common.

Footings are the second type of foundation. Located at the base of the substructure unit, footings spread out and transmit the weight of all bridge live and dead loads to the supporting soil or bedrock. They also provide stability against substructure unit overturning and sliding due to lateral soil pressures. In Wisconsin, footings are normally only used to bear on sound bedrock and only when the bedrock is located close to the bottom of the substructure.

Caissons are the third type of foundation. Caissons are drilled shafts which can offer larger diameters and deeper depths when compared to driven piles. Caissons will typically be used for larger construction projects with large loads or on bridge projects with restrictive site locations. Caissons can have rebar cages placed into the drilled shafts before the concrete is poured for added strength depending on the diameter and depth. A common failure is the caisson walls collapsing prior to the addition of concrete.

Typically, foundations are buried underground and should not be visible when the bridge is in service. However, the top of foundation may be designed to be exposed. An exception is pile bent foundations. Pile bents are substructure units with the piles extending above grade. After driving, the pile tops are tied together with a conventional pier cap.

2.5.2 Steel Substructure Elements

Other than piles, steel is not commonly used for substructure elements in new designs. However, it was often used in the past to form the bents or bent towers of large bridges. Some shorter span structures even used this substructure type. It is unusual, but not unheard of, to see an abutment or wingwall built from steel.

The most common defect found on steel substructures is corrosion, and the heaviest corrosion is generally found below failed and leaking expansion joints. Other defects of concern are fatigue cracks, vehicle collision damage, overload damage, and fire damage.

2.5.2.1 Steel Column (Element 202)

The columns of steel piers are primary load-carrying elements, resisting both compressive axial loads and bending moments. Columns may be pipes or fabricated box shapes. Steel piers consist of two or more steel columns connected along their tops by a pier cap built of steel or reinforced concrete.

A column differs from a pile in the way that it is supported. A column will be supported by a concrete pedestal or footing that can be beneath the ground level or exposed. A pile will extend well past the ground level into more substantial bearing material, such as bedrock or stone that will provide the substructure with structural stability. If a column's pedestal or footing is buried it may be difficult to tell the difference between a column and a pile without looking at the bridge plans.

Element Level Inspection

On the inspection report form, columns are recorded in units of "each". For each column, the most severe Condition State is assigned to the entire element. This will quantify the column's condition and help to generate quantity/cost estimates for future remedial work.

Safety Inspection

During the Element Level Inspection of steel columns, it is important to remember that the entire purpose of bridge inspection is to ensure public safety. A structural inspection must also be carried out, regardless of the coating condition. The following will serve as a guide for what the inspector should be looking out for to judge an element's ability to carry the design loads, and to identify current or future structural problems.

Inspection of steel columns should include the following items:

- Looking for local compression overload damage in the form of local member component buckling, plate waviness or crippling. This may be evident near the ground line of abutment piles where maximum bending compressive stresses occur.
- Looking for global buckling which will take the form of a bow or sweep in the member. This could be the result of a structural overload or differential settlement.
- Examining the member ends for cracks and loose fasteners. Suspect fasteners may be checked for looseness by twisting by hand or tapping the heads with a hammer.

- Checking corroded areas for excessive section loss that may be increasing member stress. Particular attention should be given to members adjacent to the splash zones of roadways, near the water line for water crossings, and any detail that would tend to trap water and debris.

- Inspecting for cracking and distortion, such as a kink that would suggest the member has experienced collision damage.

- Checking the pier for plumbness visually or with a plumb bob.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)
- Microbial Induced Corrosion (8901)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow

- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.2.2 Steel Tower/Trestle (Element 207)

This element is intended to be used for truss framed tower supports or built up steel towers. This element is intended to capture large supports and towers associated with suspension bridges, cable stayed bridges, moveable bridges, or similar structural configurations.

A tower or trestle element will be three dimensional in nature with lateral and torsional bracing along all sides of the substructure unit. Pier bents with a single strut running between the two does not constitute trestle bracing.

Element Level Inspection

On the inspection report form, a steel column tower is recorded in units of lineal feet of vertical height. This quantity is the sum of the heights of each built-up or framed tower supports. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

- Microbial Induced Corrosion (8901)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

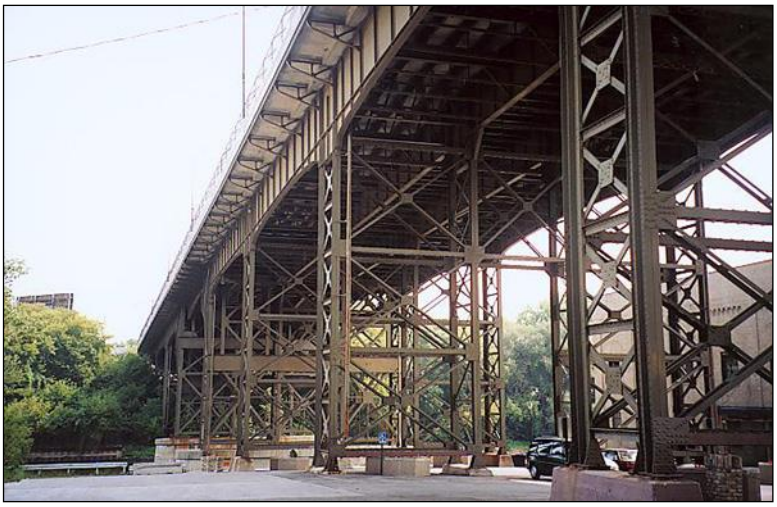

Figure 2.5.2.2-1: Steel Towers/Trestles – Condition State 1

2.5.2.3 Steel Abutment (Element 219)

This includes the sheet material retaining the embankment and abutment extensions. This is for all steel abutments regardless of protective system. The abutment material type is coded as the main fill retaining material. Therefore, if an abutment is constructed with timber piles and a steel lagging behind retaining the fill, then Steel Abutment will be used in conjunction with Timber Pile elements.

Element Level Inspection

On the inspection report form, a steel abutment is recorded in units of lineal feet. This measurement is taken as the length of the abutment taken along the skew (if present). Where multiple condition states exist within a unit of measure only the predominant defect in

severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Settlement Defects

- Settlement (4000)
- Scour (6000)
- Microbial Induced Corrosion (8901)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.2.4 Steel Pile (Element 225)

This element defines steel piles that are visible for inspection. Piles exposed by erosion or scour and piles visible during an underwater inspection are also included in this element. This element is for all steel piles regardless of protective system. Some abutment types may also use exposed steel piles, most notably timber abutments. Steel piles are driven and left to extend from the ground to the abutment cap. After the abutment cap (usually reinforced concrete) is placed, timber lagging is placed at the back face of the piles. Backfill is placed behind the lagging, and this process is repeated until the pile cap is reached. The steel abutment piles must therefore resist vertical loads from the superstructure and pier cap, as well as lateral soil pressures from the fill under the approach. In this sense, the piles are acting as beam/columns to resist axial compressive loads and bending moments. Maximum bending stresses occur near the ground line, which is also where corrosion is most likely to take place.

As discussed in the Steel Column Element section, a pile will extend well past the ground level into more substantial bearing material, such as bedrock or stone that will provide the substructure with structural stability. Piles can be easily confused with columns without first looking at the bridge plans if a pedestal or footing is not exposed.

For coding purposes, piles used to support abutment lagging are considered separate from the abutment and shall be coded separately as pile elements. Piles used to support wingwall lagging are considered part of the wingwall and shall be evaluated under Integral Wingwall (Element 8400).

Cast-in-place or CIP piling shall be evaluated as Steel Pile (Element 225). Inspectors can only evaluate what they can inspect, which in the case of these piles is the exposed steel shell. The inspector shall note under the Steel Pile element that the piling is CIP piling. The inspector should verify from the original bridge plans if the concrete is reinforced or not.

Figure 2.5.2.4-1: Heavy Corrosion at the Ground Line on Painted Steel Piles - Condition State 4

Element Level Inspection

On the inspection report form, steel piles are recorded in units of “each”. For each pile, the most severe Condition State is assigned to the entire element. This will quantify the pile’s condition and help to generate quantity/cost estimates for future remedial work. The inspector should pay close attention to corrosion and section loss which can lead to structural reduction of the member. These defects can be accelerated in wet environments such as waterways. If a pile is partially exposed, the unit of the element remains “each” and the entire pile shall be evaluated based on the exposed area.

Element Defect

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)

- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)
- Microbial Induced Corrosion (8901)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.2.5 Steel Pier Cap (Element 231)

Figure 2.5.2.5-1: Steel Box Pier Cap

Pier caps are primary load-carrying bending members. Because they must carry large girder reaction loads, steel pier caps are often fabricated into box shapes, although I-shaped members are also used for short crossings. Pier cap boxes are usually large enough for an

inspector to enter and examine its interior. Steel pier caps may work in conjunction with steel columns to form a frame or they may bear on top of individual concrete pier columns.

A distinction needs to be made regarding the difference between a steel pier cap and a steel cross girder. Both function to deliver the girder end reactions to the pier columns. However, on pier caps, the superstructure girders bear on the cap's top flange. Bearing devices keep the girders independent from the cap.

Similar to a cap, cross girders support superstructure girders which are welded or bolted directly to the cross girder web. There are no bearing devices separating these two components. Any bearing devices are located on the underside of the cross girder at the pier columns. Cross girders are considered superstructure elements as they are supported by bearing. Refer to Chapter 4 for additional information on girder elements.

Element Level Inspection

On the inspection report form, a steel pier cap is recorded in units of lineal feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Safety Inspection

During the Element Level Inspection of steel pier caps, it is important to remember that the entire purpose of bridge inspection is to ensure public safety. The main purpose of pier caps is to transmit superstructure loads to the pier columns. A structural failure could mean a localized bridge failure requiring shutting down part or all of the bridge.

Flexural Areas: Bending zones are located throughout the length of a pier cap, except at the ends when rotation is allowed at the top of exterior columns. Positive bending areas are located between the supports. Negative bending locations are directly over the interior columns and above the exterior columns when a rigid connection is provided.

Maintenance inspection in the flexural areas of steel pier caps should include the following items:

- Examining the flexure zones and tension flanges for corrosion and loss of cross-sectional area, which is the most common steel defect. About 10 percent flange section loss or greater will begin to raise the stress level an appreciable amount.

- Removing spot areas of debris accumulation to check for corrosion. Bird waste often found on the flanges is acidic and traps moisture and debris, accelerating corrosion.

- Checking rivet/bolt heads on built-up components, as corrosion on the heads may indicate corrosion along the entire fastener length, reducing structural integrity.

- Looking for pack rust, noted by individual plate bending between fasteners. Pack rust may be present between the plies of riveted/bolted connections such as field splices or secondary member connections.

- Looking for overload damage in the form of compression flange buckling and tension flange elongation or fracture in the high moment flexural regions.

- Looking for rotation in the pier cap due to eccentric connections.

- Examining suspect fasteners for looseness by twisting by hand or tapping the heads with a hammer.

- Checking the pier cap for distortion or scraping from traffic impacts.

- Sighting down the member's length to check vertical and horizontal alignments, as well as for any canting (lateral bending or twisting). This type of damage may be due to overloads, traffic impact or support settlement.

Shear Zones: The zones of highest shear stresses are located at the columns or piles. Most steel pier caps make use of vertical bearing stiffeners at their supports.

Maintenance inspection in the shear areas of pier caps should include the following items:

- Looking for web crippling where bearing stiffeners are not used. Web crippling is a permanent wrinkling or buckling of the web due to overloads.

- Checking for web section loss due to corrosion. Web section loss makes the web less stiff and more susceptible to crippling.

- Checking the bearing stiffeners for corrosion and any associated buckling due to overloads.

Safety Inspection - Fatigue

Primary bending members are susceptible to fatigue damage. Fatigue cracks usually show up as rust stains or rusty breaks in the paint, propagating perpendicular to the direction of stress.

Hot-rolled Pier Caps – Hot-rolled beams may sometimes be used as pier caps for smaller bridges with multi-column piers. Fatigue inspection of these hot-rolled steel beams should include the following items:

- Looking for welded repairs which increase the static strength of a member, but greatly reduce the fatigue strength. These include patch plates fillet welded over heavily corroded areas producing sudden geometric changes and poor quality plug welds used to fill mis-drilled bolt holes. Weld cooling also creates high residual tensile stresses in the base material.

- Investigating bearing stiffeners that are erroneously welded to the tension flange. These welds act as stress risers and could be a crack initiation point. Carefully check the welds and flange on these floor system components for cracks.

Fabricated Pier Caps: In-plane bending fatigue prone details on fabricated girder pier caps are similar to those found on hot-rolled beams. In addition, fatigue inspection in the flexural areas of fabricated steel pier caps should include the following items:

- Investigating groove welds used to join the ends of web plates or different size flange plates. These welds may be found ground flush or with the reinforcement left in place. In either case, though fatigue cracks are not normally expected to be found, poor welding or poor inspection may have left internal flaws within the weld metal, especially on older bridges.

- Examining intersecting welds. High residual tensile stresses, internal flaws, and low fatigue strengths are created when welds intersect at attachments. Common intersecting weld locations are at a transverse stiffener/web/flange junction or bearing stiffener/web/flange junction.

Box Pier Caps: In-plane bending fatigue prone details on box girder pier caps are similar to those found on fabricated open girders. In addition, fatigue inspection in the flexural areas of steel box girders should include the following items:

- Investigating back-up bars that are welded together end-to-end located within tension or stress reversal zones. These bars carry the same longitudinal stresses as the pier cap box and may produce fatigue cracks due to low quality welds joining the bars or discontinuity of the bars.

- Investigating any web or flange longitudinal stiffeners that are welded together end-to-end and are within tension or stress reversal zones. Pay particular attention to all questionable details located along the tension flanges.

- Checking all welded attachments inside the box pier cap including the transverse stiffeners and diaphragms.

Riveted and Bolted Members: Though welded structures are most often associated with fatigue concerns, mechanically fabricated members are also susceptible to fatigue damage. Vulnerable locations are essentially the same for welded and riveted members, that is, at connections.

Miscellaneous: Fatigue inspection of miscellaneous components should include looking for stress risers on tension flanges such as tack welds, gouges, and indiscriminately placed attachment welds. These may occur on any welded or riveted/bolted member listed above. Flaws such as these should be marked, recorded, and ground smooth. Until the areas are repaired, the member should be closely monitored to spot crack development.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering

indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)
- Microbial Induced Corrosion (8901)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.2.6 Nonredundant Steel Tension Member Steel Substructures

Bridges can be either Nonredundant Steel Tension Member (NSTM) or non-NSTM depending on whether they are load path redundant or not. A bridge is considered load path redundant if it has three or more primary load-carrying members spanning between supports. If a bridge has two or less primary load-carrying members running between supports it does not have load path redundancy and therefore considered a NSTM bridge.

NSTM bridges contain NSTM (NSTMs), and these members require special attention during an inspection. A NSTM is in tension or has a tension element whose failure would probably cause a portion of or the entire bridge to collapse. There must not be any other member or system of members that will serve the functions of the member in question should it fail. Fatigue failures are the main cause of concern for steel NSTMs since fatigue failures are brittle and give no warning before imminent collapse.

There is one type of steel substructure member classified as NSTM. This member is a single I-girder or box girder pier cap supported by two columns. The pier cap's tension flange is the NSTM element.

Steel pier caps should not be confused with steel cross girders. Pier caps are elements separate from the superstructure. Superstructure girders bear on the pier cap's top flange, separated by bearing devices. Cross girders are part of the superstructure. The superstructure girders are directly connected to the cross girder web by welding or bolting. Any bearing devices are located on the underside of the cross girder.

Wisconsin policy requires that qualified personnel conduct NSTM bridge inspections. Prior to performing the fieldwork, the policy requires this team to develop an inspection plan which identifies all NSTM on the bridge. It also requires a historical review of all available bridge information (original construction documents, rehabilitation plans, etc.) so that the team is aware of any suspect details or previous deficiencies.

Field inspection for a NSTM bridge is "hands-on," meaning a visual inspection of all NSTMs should be performed from within arm's reach. Every square foot of a NSTM should be examined. To accomplish this, debris needs to be cleaned off all members so that a thorough inspection can be made. For box pier caps, the interior and the exterior must be examined. The inspection team should look for the same deficiencies as on any steel member, paying particular attention to any fatigue cracks that may be found. An inspector seeing and reporting such a flaw is the only line of defense against a sudden NSTM failure and potential total collapse of a bridge.

The condition of each NSTM should be recorded on the NSTM Bridge Inspection Form with deficiencies noted or an "ok" to indicate the member has been examined.

Figure 2.5.2.6-1: NSTM Riveted Steel Box Girder Pier Cap. Note that the superstructure girders bear on the pier cap's top flange by way of bearing devices

2.5.3 Reinforced Concrete Substructure Elements

Concrete is by far the most common material used to construct bridge substructure elements. Around the turn of the 20th century, massive concrete substructures were built to replicate the more commonly used masonry substructures. The ease of placement, formability, and long term durability of concrete was quickly recognized, and this led to the near elimination of building masonry substructures.

2.5.3.1 Reinforced Concrete Column (Element 205)

Reinforced concrete columns and shafts refer to pier elements. The term “column” is used for piers with two or more vertical supporting members, while the term “shaft” refers to piers with one vertical supporting member. They are primary load-carrying members that directly support pier caps, hammerheads, and sometimes girders.

Figure 2.5.3.1-1: Reinforced Concrete Multi Column Piers

Columns/shafts are primarily compression members, but they must also resist lateral bending moments due to wind loads, eccentric loading at their tops, superstructure longitudinal forces, and differential substructure settlements.

Element Level Inspection

On the inspection report form, columns and shafts are recorded in units of “each”. It is the inspector’s task to examine each column or shaft and reasonably assign the most severe Condition State to the whole element. This will quantify the column/shaft’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of reinforced/prestressed concrete pier columns and shafts should include the following items:

- Checking the column's base for transverse flexural cracks. These cracks indicate excessive column bending. Sources for this bending may be from expansion bearings that have corroded and locked up above the pier. Superstructure expansion/contraction will then pull the pier cap along, bending the columns. Excessive lateral forces due to wind or centrifugal effects may also cause bending.

- Checking the pier cap/column interface for horizontal or diagonal flexural cracks. These cracks will originate at the inside corner of the cap/column junction and are a sign of excessive lateral bending.

- Checking the mid-height of the column for flexural cracks, as this is a sign of structural overloads or differential substructure settlement.

- Examining the entire column for vertical cracks and crushed concrete. This could be the result of a serious structural overload.

- Checking the entire column for delaminations, spalls, and exposed reinforcing/prestressing steel. Suspect areas are in roadway splash zone or bridge drainage areas, at water lines for water crossings or at grade. These defects reduce these members' cross sectional area, resulting in higher stresses.

- Looking for leaching, and noting if it is stained with rust since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Checking the column/shaft for plumb visually or with a plumb bob.

- Checking previously repaired areas for soundness by hammer tapping.

- Noting if any soil, rock or debris has been piled against the column/shaft. This will cause lateral forces on the member not originally accounted for in the original design.

- Noting any abrasion of the concrete surface for columns or shafts located within a waterway.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)

- Exposed Rebar (1090)

- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.5.3.1-2: Hammerhead Pier Reinforced Concrete Column – Condition State 1 (The cap of the hammerhead pier is reported under Element 234).

Figure 2.5.3.1-3: Concrete Column with Vertical Crack - Condition State 2

Figure 2.5.3.1-4: Transverse Flexural Cracks at the Base of a Column - Condition State 2

Figure 2.5.3.1-5: Column Spall with No Exposed Rebar - Condition State 2

Figure 2.5.3.1-6: Delaminated and Spalled Column - Condition State 3

Figure 2.5.3.1-7: Spalling on a Reinforced Concrete Shaft - Condition State 3

Figure 2.5.3.1-8: Heavily Spalled Column - Condition State 4

2.5.3.2 Reinforced Concrete Pier Wall (Element 210)

Reinforced concrete pier walls refer to solid pier walls,. Solid pier walls are primary load-carrying members that support the superstructure. Crash walls are safety devices located near grade or the water line, and web walls are secondary members used for lateral pier support. All web walls and most crash walls shall be coded under Crash Walls/Web Walls/Cross Bracing or Struts (Assessment 9250). The exception is when a crash wall also acts as a pier wall. That is, it has a full length foundation and supports the pier columns.

Figure 2.5.3.2-1: Reinforced Concrete Pier Wall

Pier walls are primarily compression members, but they must also resist lateral forces due to wind loads, and bending moments due to eccentric loading at their tops, superstructure longitudinal forces, and differential substructure settlements. Crash walls typically only resist traffic impact loads, and web walls resist lateral wind forces.

Element Level Inspection

On the inspection report form, a pier wall is recorded in units of lineal feet. This measurement is taken as the length of the pier wall from end to end taken along the skew (if present.) The total quantity is the sum of all pier wall lengths in the structure. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of reinforced concrete pier walls should include the following items:

- Checking the wall's base for transverse flexural cracks. These cracks indicate excessive column bending. Sources for this bending may be from expansion bearings that have corroded and locked up above the pier. Superstructure expansion/contraction will then pull the pier cap along, bending the wall.

- Examining the entire wall for vertical or diagonal cracks. Diagonal cracks in web walls may be the result of excessive lateral shear.

- Checking the entire wall for delaminations, spalls, and exposed reinforcing steel. Suspect areas are in roadway splash zone or bridge drainage areas, at water lines for water crossings or at grade. These defects reduce these members' cross sectional area, resulting in higher stresses.

- Looking for leaching, and noting if it is stained with rust since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Examining the top surface (bearing seat) of walls without pier caps for cracking and spalling. The pedestals and grout pads under the bearings should also be checked for cracking, spalls, and deterioration that reduce the bearing area. Deterioration in these areas may be caused by the lack of reinforcing steel in older bridges, frozen expansion bearings that transmit lateral forces to the pier not intended in the original design, and salt-laden water leaking through expansion joints.

- Visually checking the wall for plumb.

- Checking previously repaired areas for soundness by hammer tapping.

- Noting if any soil, rock or debris has been piled against the wall. This will cause lateral forces on the member not originally accounted for in the original design.

- Noting any abrasion of the concrete surface for pier walls located within a waterway.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Rebar (1090)
- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.5.3.2-2: Cracked, Delaminating and Spalled Reinforced Concrete Pier Wall – Condition State 3

Figure 2.5.3.2-3: Disintegration of a Reinforced Concrete Pier Wall – Condition State 3

2.5.3.3 Reinforced Concrete Abutment (Element 215)

Reinforced concrete abutments are classified according to their function and placement relative to the embankment. The most common types in Wisconsin are:

- **Sill:** Short height abutments using a single row of vertical piles for support. They are placed at the top of the embankment and use a sloped berm in front to contain the soil under the approach. Reinforcing steel is nominal since bending between the piles and shear forces are small relative to the capacity of the abutment body. Although this is the least expensive and easiest abutment to construct, it requires the longest bridge spans in order to clear the berms.

- **Full Retaining:** Tall abutments designed as cantilever retaining walls to hold back soil under the approach. Because of this function, the main reinforcing steel is placed vertically at the back face of the abutment. A spread footing, supported by the soil or two rows of piles, anchors the abutment stem below grade. Full retaining abutments are advantageous in that span lengths are minimized, but their disadvantages include high construction costs, minimal horizontal clearances and sight distances, and they are collision hazards.

- **Semi Retaining:** Similar to full retaining abutments, semi retaining abutments use small sloped berms placed between the roadway underpass and bearing seat. The berm allows greater horizontal clearances, sight distances, and better protection from errant vehicles.

- **Pile Encased (Integral):** Medium to tall abutments used when site conditions render a bridge using sill abutments more costly. Piles are driven into the ground and left extending up the full abutment height. The piles are then encased in concrete, forming part of the abutment stem. This abutment type is called “integral” in that the superstructure is locked to the top of the abutment, allowing the superstructure and substructure to act as a unit. Integral abutments therefore offer the advantage of eliminating expansion joints which often leak, damaging deicing chemicals onto the

superstructure. The drawback is that the abutment and piles must be able to resist any creep, shrinkage, and thermal movements of the superstructure.

Figure 2.5.3.3-1: Reinforced Concrete Semi Retaining Abutment

Girder superstructures, primarily prestressed concrete girder superstructures bear on top of the abutment and have full height concrete diaphragms between them. When the concrete diaphragms retain fill, they shall be evaluated as part of the concrete abutment. If it is known that a backwall is behind the full height diaphragms, the diaphragms will then be evaluated as an assessment. When a diaphragm retains fill it is acting as an abutment element and therefore shall be evaluated with the abutment element.

Mechanically stabilized earth (MSE) walls have been built for bridges in Wisconsin. This relatively new soil retaining system uses precast concrete panels in conjunction with steel straps to reinforce the fill behind the panels and under the approach. The steel straps act as shear reinforcing for the soil, forcing it to act as a large mass rather than as many individual particles that could easily slide. The concrete panels prevent the fill from washing out of the soil mass. MSE walls eliminate the sloped embankment in front of an abutment, similar to a full retaining abutment. When MSE walls are used on bridges, sill abutments supported on either spread footings or piles are normally placed on top of the soil mass and adjacent to the top of the wall. Because of this and due to the fact that the precast panels carry no vertical loads, MSE walls should never be considered as reinforced concrete abutments. They often have their own structure number, and should be treated as retaining walls separate from the bridge structure. MSE walls without their own structure number, however, shall still be treated as a structure separate from the bridge. Since retaining wall inspection is not currently mandatory in Wisconsin, the inspector should provide comments on the Bridge Inspection Report form as to whether the MSE wall is in good, fair, or poor condition. If warranted, MSE walls in poor condition may be considered a critical finding and the Program Manager should be notified.

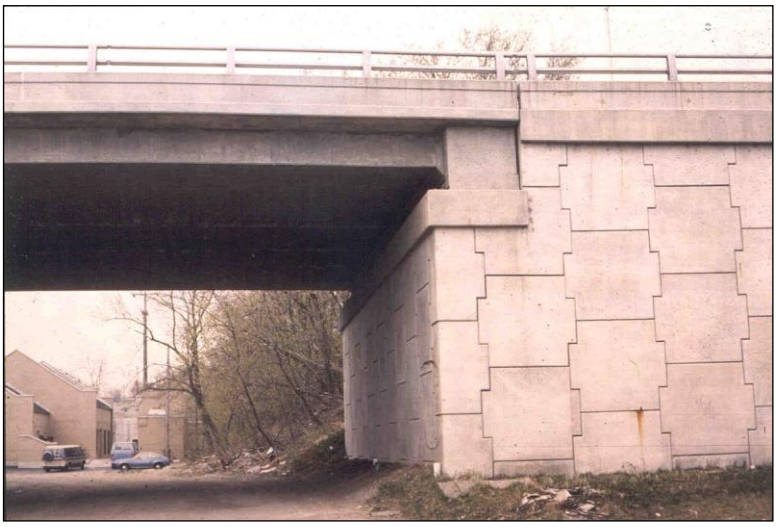

Figure 2.5.3.3-2: Sill Abutment and MSE Wall

Element Level Inspection

On the inspection report form, reinforced concrete abutments are recorded in units of lineal feet. This measurement is taken as the length of the abutment taken along the skew (if present). The total quantity is the sum of all abutments. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of reinforced concrete abutments should include the following items:

- Examining the abutment and backwall for vertical or diagonal cracks. Vertical cracks are often the result of shrinkage, while diagonal cracks may indicate a shear overload or differential pile settlement.

- Checking the entire abutment for delaminations, spalls, and exposed reinforcing steel. Suspect areas are in roadway splash zone or bridge drainage areas, at water lines for water crossings or at grade.

- Looking for leaching, and noting if it is stained with rust, since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls. Fill under the approach may retain moisture during rains or spring thaws, so leaching may be seen at cracks where this moisture escapes.

- Checking the construction joint between the backwall and bearing seat for deterioration.

- Examining the top surface (bearing seat) of abutments for cracking and spalling. The pedestals and grout pads under the bearings should also be checked for cracking,

spalls, and deterioration that reduce the bearing area. Deterioration in these areas may be caused by the lack of reinforcing steel in older bridges, frozen expansion bearings that transmit lateral forces to the abutment not intended for in the original design, and salt-laden water leaking through expansion joints.

- Looking for the presence of debris or standing water on the bearing seat. Debris suggests a failed/leaky expansion joint. Standing water indicates that the bearing seat is back-pitched. Salt-laden standing water ponding on the bearing seat will eventually migrate to the reinforcing steel, causing corrosion, delaminations, and spalls.
- Checking tall abutments for plumb visually or using a plumb bob.
- Checking previously repaired areas for soundness by hammer tapping.
- Checking that any weep holes present are clear and functioning properly.
- Noting any abrasion of the concrete surface for abutments located within a waterway.
- Looking for granular soil deposits outside base of wall caused by failed weep holes or excessive joint gaps.
- Noting any MSE wall panels that are shifting out of place or allowing fill to wash out from behind.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Rebar (1090)
- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)

- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.5.3.3-3: Reinforced Concrete Abutment - Condition State 1

Figure 2.5.3.3-4: Vertical Crack in an Abutment – Condition State 2

Figure 2.5.3.3-5: Abutment Delaminations Under the Bearing – Condition State 2

Figure 2.5.3.3-6: Large Spall on Abutment Bearing Seat – Condition State 3

2.5.3.4 Reinforced Concrete Pile Cap/Footing (Element 220)

These elements are typically located within waterways and serve as foundations for either piers or abutments. Footings are most often supported on piles, however the footing may be a spread footing. It is not common for pile caps or footings to be exposed, so a review of the original plans is very helpful to determine if they are exposed by design or by scour.

Inspection of these two elements is difficult, but can be performed visually and by using a probe. Pile caps and footings located only a foot or two below the water surface can often be visually inspected by wading or from a boat, assuming that the water is not too murky. A probing rod may also be used to check the soundness of concrete. Dragging or scraping the rod along the element surface may reveal locations of cracks or spalls.

Element Level Inspection

On the inspection report form, reinforced concrete pier caps and footings are recorded in units of “lineal feet”. The measurement is taken as the length of the element from end to end along the skew (if present). The total quantity is the sum of all the respective element lengths. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of reinforced concrete pier caps and footings should include the following items:

- Visually examining the element to look for cracks, delaminations, spalls, and exposed reinforcing steel.
- Noting any abrasion of the concrete surface.

- Dragging or scraping a probing rod along the surface of the concrete to check for the presence of cracks, spalls or abrasion. This technique is particularly useful in murky water.

- Walking along the top of the element, if possible, to feel for large spalls or areas of abrasion.

- Note any evidence of scour in the inspection report.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Rebar (1090)
- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow

- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.3.5 Reinforced Concrete Pile (Element 227)

These elements define reinforced concrete piles that are visible for inspection. Piles exposed by erosion or scour and piles visible during an underwater inspection are included in these elements. These elements are for all reinforced concrete piles regardless of protective system.

Element Level Inspection

On the inspection report form, prestressed and reinforced concrete piles are recorded in units of "each". Therefore, each element will be individually rated and assigned to a Condition State. It is the inspector's task to examine each pile and reasonably assign the most severe Condition State to the entire element. This will quantify the concrete's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Rebar (1090)
- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.3.6 Reinforced Concrete Pier Cap (Element 234)

Pier caps are primary load-carrying bending members on single columns (hammerheads) or multicolumn piers. They may be bending members on pier walls if their ends cantilever over the ends of the wall, but they are most often used as architectural features in this situation. They often must carry large girder reaction loads and many times work in conjunction with the columns to form a frame for resisting lateral loads.

Figure 2.5.3.6-1: Reinforced Concrete Pier Cap

Element Level Inspection

On the inspection report form, a pier cap is recorded in units of lineal feet. The measurement is taken as the length of the cap from end-to-end along the skew (if present). The total quantity is the sum of all pier cap lengths. Where multiple condition states exist within a unit of

measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of reinforced concrete pier caps should include the following items:

- Examination for vertical flexural cracks, either on the underside between columns or top side above columns or shafts. Wide cracks in a flexural region indicate a serious structural overload.

- Checking for deteriorated concrete in the flexural zones that is causing debonding of the reinforcing steel. This is especially critical near the ends of the reinforcing steel bars, since a certain length of the bar must be embedded within sound concrete to fully develop its strength. The deterioration may be delaminations, spalls, and longitudinal cracks.

- Looking for shear cracks over and near the supports. Shear cracks will be diagonal, extending up from the column towards mid-span. Wide shear cracks suggest the loss of aggregate interlock, meaning the member could be hanging from the reinforcing stirrups. Maximum crack widths should be measured and noted on the bridge inspection report.

- Checking the entire member for signs of corroding reinforcing steel, as indicated by rust stains or exposed reinforcement or prestressing steel. Since section loss associated with reinforcing and prestressing steel corrosion can reduce a member's strength, measure this loss if possible and record it in the inspection report.

- Looking for leaching and noting if it is stained with rust, since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as delaminations and spalls.

- Checking the member under drains or leaking expansion joints for cracks, delaminations, spalls, and exposed reinforcing steel.

- Checking previously repaired areas for soundness by hammer tapping.

- Checking the entire abutment for delaminations, spalls, and exposed reinforcing steel. Suspect areas are in roadway splash zone or bridge drainage areas, at water lines for water crossings or at grade.

- Examining the top surface (bearing seat) of pier caps for cracking and spalling. The pedestals and grout pads under the bearings should also be checked for cracking, spalls, and deterioration that reduce the bearing area. Deterioration in these areas may be caused by the lack of reinforcing steel in older bridges, frozen expansion bearings that transmit lateral forces to the pier not intended for in the original design, and salt-laden water leaking through expansion joints.

- Looking for the presence of debris or standing water on the bearing seat. Debris suggests a failed/leaky expansion joint. Standing water indicates that the bearing seat is dished. Salt-laden standing water will eventually migrate to the reinforcing steel, causing corrosion, delaminations and spalls.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Rebar (1090)
- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.5.3.6-2: Reinforced Concrete Pier Cap - Condition State 1

Figure 2.5.3.6-3: Delamination on the Underside of a Pier Cap – Condition State 2

Figure 2.5.3.6-4: Delaminations on the Underside of a Pier Cap - Condition State 2

Figure 2.5.3.6-5: Pier Cap with Widespread Spalling – Condition State 3

2.5.4 Prestressed Concrete Substructure Elements

2.5.4.1 Prestressed Concrete Column (Element 204)

This element is for all prestressed concrete columns regardless of protective system. These elements include cast in place columns that are post-tensioned (not externally post-tensioned).

A column differs from a pile in the way that it is supported. A column will be supported by a concrete pedestal or footing that can be beneath the ground level or exposed. A pile will extend well past the ground level into more substantial bearing material, such as bedrock or stone that will provide the substructure with structural stability. If a columns pedestal or footing is buried it may be difficult to tell the difference between a column and a pile without looking at the bridge plans.

Element Level Inspection

On the inspection report form, prestressed concrete columns are recorded in units of “each”. For each column, the most severe Condition State is assigned to the entire element. This will quantify the column’s condition and help to generate quantity/cost estimates for future remedial work.

Element Defect

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Prestressing (1110)
- Cracking (PSC) (1110)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.4.2 Prestressed Concrete Pile (Element 226)

This element defines prestressed concrete piles that are visible for inspection. Piles exposed by erosion or scour and piles visible during an underwater inspection are included in these elements. These elements are for all prestressed concrete piles regardless of protective system.

Element Level Inspection

On the inspection report prestressed concrete piles are recorded in units of “each”. For each pile, the most severe Condition State is assigned to the entire element. This will quantify the pile’s condition and help to generate quantity/cost estimates for future remedial work.

Element Defect

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Prestressing (1110)
- Cracking (PSC) (1110)
- Abrasion/Wear (PSC/RC) (1190)

- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.4.3 Prestressed Concrete Pier Cap (Element 233)

This element defines those prestressed concrete pier caps that support girders and transfer load into piles or columns. Pier caps are primary load-carrying bending members on single columns (hammerheads) or multicolumn piers. They may be bending members on pier walls if their ends cantilever over the ends of the wall, but they are most often used as architectural features in this situation. They often must carry large girder reaction loads and many times work in conjunction with the columns to form a frame for resisting lateral loads.

Element Level Inspection

On the inspection report form, a prestressed concrete cap is recorded in units of lineal feet. This quantity is the length of the cap. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defect

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in

their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Prestressing (1110)
- Cracking (PSC) (1110)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.5 Timber Substructure Elements

Because of its availability, timber is used most frequently for bridge substructure elements in rural areas of the state. However, older bridges with timber substructure elements can be found in urban areas.

Figure 2.5.4.3-1: Timber Abutment and Wingwalls

2.5.5.1 Timber Column (Element 206)

Columns for timber piers bear on reinforced concrete pedestals located at the ground line. Piers of either type are laterally stabilized through the use of timber bracing (these elements should be rated under Assessment 9250 Crash Walls/Web Walls/Cross Bracing or Struts). Tops of the timber columns are tied together with a timber, reinforced concrete or rolled steel pier cap.

A column differs from a pile in the way that it is supported. A column will be supported by a concrete pedestal or footing that can be beneath the ground level or exposed. A pile will extend well past the ground level into more substantial bearing material, such as bedrock or stone that will provide the substructure with structural stability. If a columns pedestal or footing is buried it may be difficult to tell the difference between a column and a pile without looking at the bridge plans.

Figure 2.5.5.1-1: Timber Pier Columns

Figure 2.5.5.1-2: Timber Columns, Cap, and Cross-Bracing – Condition State 1

Since timber columns are vertical elements, the very permeable end grain is directly exposed to rain. Thin lead or zinc sheets are often draped over the element ends to keep them dry.

Element Level Inspection

On the inspection report form, timber columns are recorded in units of “each”. It is the inspector’s task to examine each column and reasonably assign the most severe Condition State to the entire element. This will quantify the column/shaft’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of timber columns found on substructure elements should include the following items:

- Checking the cap/column interface for bearing failures. Bearing failures on timber members loaded parallel to the grain will “broom out”.

- Checking the mid-height of pier columns for flexural cracks, which are signs of structural overloads or differential pier deflection.

- Examining the entire element for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.

- Check for wood crushing due to fasteners. This is often seen at dead-men (tie back rod) connections. This may be due to timber decay or settlement causing overloading on the tie back members.

- Looking for any splitting of sawn timber members. Excessively long or wide splits may be a sign of a structural overload.

- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the member is tapped with a hammer.

- Looking for fire damage. The remaining dimensions should be measured to determine the severity of section loss and potential loss of load capacity.

- Checking fasteners (bolts, lag screws) for corrosion or slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Sighting the substructure to look for out of plumbness. A plumb bob may also be used. The column should also be sighted along its length to check for bowing. Excessive deflections indicate that the member has been overstressed or that the bridge is experiencing differential settlements. The measured or estimated amount of deflection should be recorded.

- Performing probe tests in areas suspected to be experiencing decay. See Section 2.4.4.1 Timber Slab (Element 54) for a description of this procedure.

- Drilling or boring suspect members to estimate the extent of decay.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Connection (1020)
- Decay/Section Loss (1140)
- Checks/Shakes/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.5.2 Timber Tower/Trestle (Element 208)

This element defines framed timber supports, and is for all timber trestle/towers regardless of protective system. This element is intended to be used for truss framed trestles or towers and is intended to capture large supports and towers associated with large deck truss bridges.

A tower or trestle element will be three dimensional in nature with lateral and torsional bracing along all sides of the substructure unit. Pier bents with a single strut running between the two does not constitute trestle bracing.

Element Level Inspection

On the inspection report form, a timber tower/trestle is recorded in units of lineal feet of vertical height. This quantity is the sum of the heights of each built-up or framed tower support. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of

measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Connection (1020)
- Decay/Section Loss (1140)
- Checks/Shakes/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.5.3 Timber Pier Wall (Element 212)

This element defines those timber pier walls that include pile, timber sheet material, and filler. This is for all pier walls regardless of protective system. This element will rarely be found on Wisconsin bridges.

Element Level Inspection

On the inspection report form, a pier wall is recorded in units of lineal feet. This measurement is taken as the width of the pier wall along the skew (if present). The total quantity is the sum of all the pier wall lengths. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shake/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above

and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.5.4 Timber Abutment (Element 216)

This element is for the timber sheet material retaining the embankment. Typically this is the timber lagging running between piling.

Timber abutments are constructed by two methods. The first type is the timber bent abutment. It is built by first driving timber or steel piles into the ground. These piles extend beyond grade approximately to the girder bearing elevation. Sawn timber lagging is then placed along the back face of the piles to the abutment cap, forming a wall. The embankment is then created by backfilling behind the lagging wall. In this manner, the timber lagging holds back the soil by spanning between the piles and would be considered the Timber Abutment. The piles (either timber or steel) would be coded as the respective material pile.

Another type is timber crib abutments. Rectangular timber elements are stacked to form a cell, similar to how a log cabin is built. This cell is then filled with soil to form the embankment. Timber crib abutments act as gravity retaining devices, using the mass of the crib and contained fill material to resist sliding from exterior soil lateral pressures. There are no piles visible in front of crib abutments.

Element Level Inspection

On the inspection report form, an abutment is recorded in units of lineal feet. The measurement is taken as the length of the abutment along the skew (if present). Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of timber abutments should include the following items:

- Examining the lagging or cribbing for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface. Suspect locations are where moisture can become trapped, such as at the abutment pile interface locations.

- Looking for any splits of individual boards.

- Checking the lagging or cribbing for excessive deflections. Excessive deflections may allow the soil behind the boards to spill or wash out, causing settlement to the approach above.

- Looking for any broken lagging boards. This may occur at a weak spot in the wood such as a knot.

- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the element is tapped with a hammer.

- Looking for fire damage.

- Hammer tapping random and suspect areas to evaluate the wood's soundness.

- Performing probe tests in areas suspected to be experiencing decay. See Section 2.4.4.1, Timber Slab (Element 54) for a description of this procedure.

- Drilling or boring suspect members to estimate the extent of decay.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shake/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

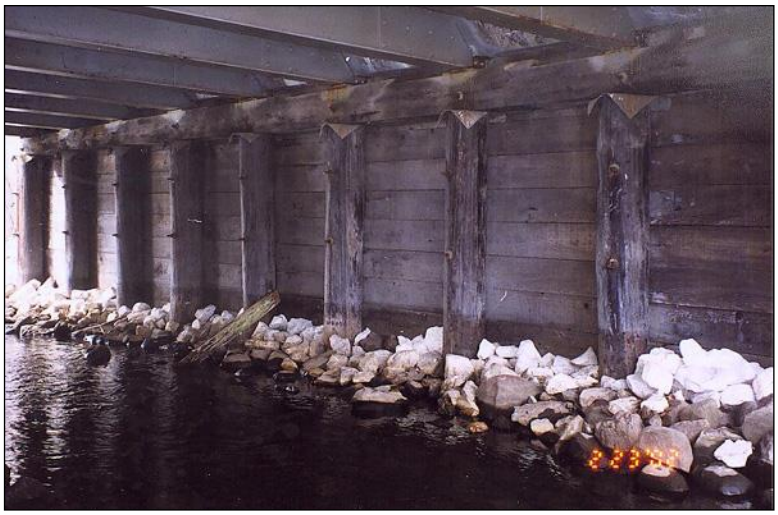

Figure 2.5.5.4-1: Timber Abutment, Pier Cap, and piles - Condition State 2

Figure 2.5.5.4-2: Bulging of a Timber Abutment (Settlement Defect) – Condition State 2

2.5.5.5 Timber Pile (Element 228)

This element defines timber piles that are visible for inspection. Piles exposed to erosion or scour and piles visible during an underwater inspection are included in this element. This element is for all timber piles regardless of protective system.

This element is not to be confused with Element 206 Timber Columns, which are designed to be supported by a footing. Timber piles are driven into the ground and supported by bearing and surface friction forces. Timber piles are typically constructed to be covered, whether it is embankment fill, stream bed, etc. Once erosion or scour exposes the piling, then this element shall be utilized.

Timber piles are often used to support the wood lagging of timber abutments and wingwalls. These piles cantilever up from the ground to receive vertical bearing loads from the superstructure and abutment cap and lateral soil pressures from the fill under the approach. The piles are acting as beam/columns to resist axial compressive loads and bending moments. Maximum bending stresses occur near the ground line, which is also where decay is most likely to take place due to the presence of moisture. Timber or reinforced concrete caps are used to tie the abutment piles together. The piles should be individually rated and should not be considered as part of the abutment element. When timber piling restrains wingwall lagging, the piling is evaluated under the appropriate wingwall element.

Timber piling is also used for timber pile bents, where the piling is intended to be exposed. The method of inspection is the same between abutment and pier piling.

Figure 2.5.5.5-1: Timber Piling – Most in Condition State 2

Figure 2.5.5.5-2: Split and Decayed Timber Piling- Condition State 3

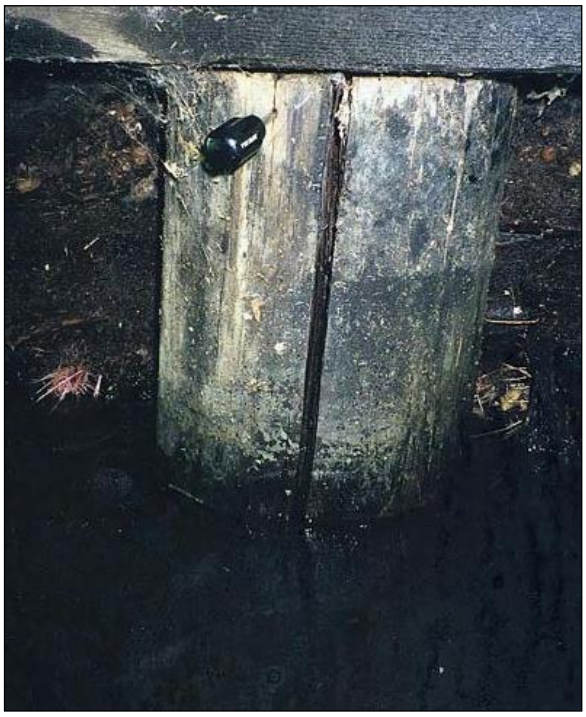

Figure 2.5.5.5-3: Split Timber Pile - Condition State 3

Figure 2.5.5.5-4: Timber Pile Decay at Ground Line – Condition State 3

Element Level Inspection

On the inspection report form, piles are recorded in units of “each”. The total quantity is the sum of all piles visible for inspection. Therefore, each element will be individually rated and assigned to a Condition State. It is the inspector’s task to examine each pile and reasonably assign the appropriate Condition State to the entire element. This will quantify the timber’s condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of timber piling found on substructure elements should include the following items:

- Checking the cap/pile interface for bearing failures. Bearing failures on timber members loaded parallel to the grain will “broom out”.
- Checking the mid-height of pier piles for flexural cracks, which are signs of structural overloads or differential pier deflection.
- Examining the entire element for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.
- Check for wood crushing due to fasteners. This is often seen at dead-men (tie back rod) connections. This may be due to timber decay or settlement causing overloading on the tie back members.
- Looking for any splitting of sawn timber members. Excessively long or wide splits may be a sign of a structural overload.
- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the member is tapped with a hammer.
- Looking for fire damage. The remaining dimensions should be measured to determine the severity of section loss and potential loss of load capacity.
- Checking fasteners (bolts, lag screws) for corrosion or slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.
- Sighting the substructure to look for out of plumbness. A plumb bob may also be used. The pile should also be sighted along its length to check for bowing. Excessive deflections indicate that the member has been overstressed or that the bridge is experiencing differential settlements. The measured or estimated amount of deflection should be recorded.
- Performing probe tests in areas suspected to be experiencing decay. See Section 2.4.4.1 Timber Slab (Element 54) for a description of this procedure.
- Drilling or boring suspect members to estimate the extent of decay.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shake/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.5.6 Timber Pier Cap (Element 235)

Timber pier caps are often used on timber substructure elements. Their purpose is to tie the tops of the piles or columns together to help create a frame and also to deliver superstructure loads to the piles or columns.

Element Level Inspection

On the inspection report form, a pier or abutment cap is recorded in units of lineal feet. The measurement is taken as the length of the element from end to end along the skew (if present). The total quantity is the sum of all the element lengths. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of timber pier caps should include the following items:

- Checking for cap crushing at the girder bearings, piles or columns. These are the most suspect areas because they tend to collect and retain the most moisture and debris, creating ideal environments for fungal growth and insect attack.

- Looking for shear related damage at and near the supports. Overloads result in high shear stresses that cause horizontal splits to form along the length of the cap, approximately mid-height. Splits will allow fungi and insects access to the untreated interior of a cap.

- Examining the high flexural regions of the cap for signs of overload damage such as crushing near the tension surface, and transverse cracking at the compression surface.

- Looking for splits or checks along the entire cap.

- Examining the entire member for signs of decay. Signs include discolored wood with a soft, rotted texture. Look also for fruiting bodies and depressed areas of the wood surface or plant growth on the top surface.

- Checking the ends of the cap for decay and hollowing. These regions tend to collect roadway runoff, debris and sunlight.

- Examining the entire member for signs of insect attack. Signs include piles of sawdust, small holes in the wood surface, insects themselves, and a hollow sound when the beam is tapped with a hammer.

- Looking for fire damage.

- Checking fasteners (nails, bolts, lag screws, deck clips) for corrosion or slipping. Check also for fastener looseness by striking with a hammer. The location of any missing fasteners should be noted.

- Sighting along the length of the cap for excessive vertical or lateral deflections. Check also for cap rotation that may be caused by eccentric beam loading. The measured or estimated amount of deflection should be recorded.

- Hammer tapping random and suspect areas to evaluate the wood's soundness.

- Performing probe tests in areas suspected to be experiencing decay. See Section 2.4.4.1, Timber Slab (Element 54) for a description of this procedure.

- Drilling or boring suspect planks to estimate the extent of decay.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Connection (1020)
- Decay/Section Loss (1140)
- Check/Shake/Cracks/Splits/Delamination (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

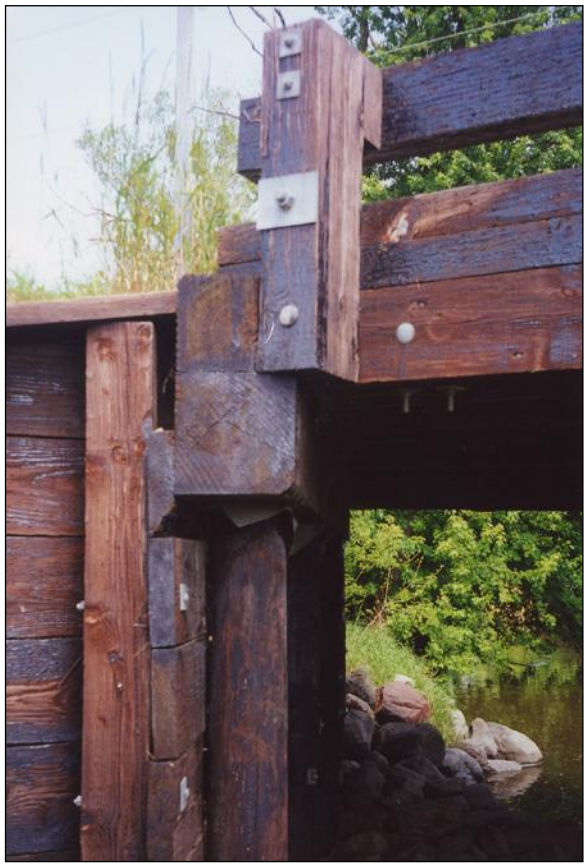

Figure 2.5.5.6-1: Timber Pier Cap and Abutment - Condition State 1

Figure 2.5.5.6-2: Split Pier Cap End (Note Plant Growth) - Condition State 3

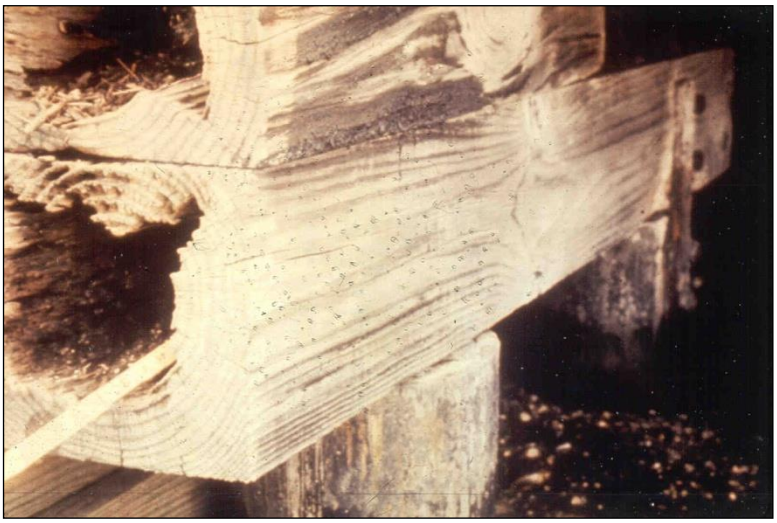

Figure 2.5.5.6-3: Severe Insect Infestation Inside of a Pier Cap – Condition State 4

2.5.6 Masonry Substructure Elements

Masonry has been used for bridge substructure elements for thousands of years. Even after steel and reinforced concrete gained favor over masonry arch superstructures in the 1800s, masonry was still used for piers, abutments, and wingwalls. Eventually, concrete substructures won favor over masonry, and new masonry bridge substructure construction is mostly limited to use as a decorative effect on small, local bridges.

Bridge aesthetics are becoming more of a concern for owners building bridges in highly visible areas. It is interesting to note that concrete form liners having the look of an old masonry pier or abutment are becoming quite popular. The creative use of paint can make these elements look deceivingly realistic when viewed from a distance. The inspector should be careful not to inspect these substructures as masonry units but as reinforced concrete.

2.5.6.1 Masonry Pier Wall (Element 213)

Masonry pier walls are solid, primary load-carrying members that support the superstructure. They are generally compression members, but they must also resist lateral forces due to wind loads, bending moments due to eccentric loading at their tops, superstructure longitudinal forces, and differential substructure settlements. Masonry pier walls are unreinforced masonry and are therefore built wide and heavy enough so that tensile stresses due to bending are virtually eliminated.

Masonry pier caps, or horizontal members comprised of masonry or stone block located on the top of Masonry Pier Walls that extend beyond the faces of the pier wall shall be evaluated under the Masonry Pier Wall element.

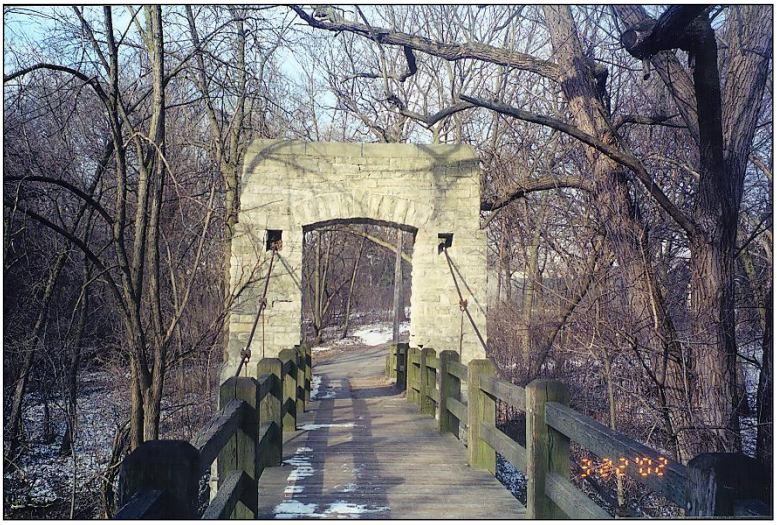

Figure 2.5.6.1-1: Masonry Tower (Pier) for a Suspension Bridge

Element Level Inspection

On the inspection report form, masonry pier walls are recorded in units of lineal feet. The measurement is taken as the length of the wall from end to end along the skew (if present). The total quantity is the sum of all pier wall lengths. Where multiple condition states exist

within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of masonry pier walls should include the following items:

- Looking for cracked, split, spalled, loose or missing stone masonry units. This would suggest weathering due to freeze/thaw effects. Missing or crushed masonry units result in a loss of pier cross sectional area, increasing the axial stresses.
- Check for bulging or misalignment of the masonry units. This may be due to overloading of the masonry wall or settlement of the substructure unit.
- Looking for cracked, broken, deteriorated, loose or missing mortar. This would suggest weathering due to freeze/thaw effects. Missing or crushed mortar results in a loss of pier cross sectional area, increasing the axial stresses.
- Looking for leaching. This indicates water is flowing through the mortar joints, leaching out cementitious minerals. Extended leaching will weaken the mortar.
- Checking areas exposed to drainage and roadway runoff. The runoff may cause scaling of the masonry units.
- Hammer tapping random and suspect areas to evaluate the masonry's soundness.
- Examining previous repair areas for soundness.
- Checking the pier for plumb visually or using a plumb bob.
- Looking for vegetation growing inside of cracks or between the mortar and masonry unit. Plant roots can exert prying forces that further deteriorate these materials.
- Noting any abrasion of the masonry for pier walls located within a waterway.
- Examining the top surface (bearing seat) of pier walls for cracking and spalling. Deterioration in these areas may be caused by frozen expansion bearings that transmit lateral forces to the pier not intended for in the original design.
- Noting if any soil, rock or debris has been piled against the pier wall. This will cause lateral forces on the element not originally accounted for in the original design.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects.

However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Mortar Breakdown (1610)
- Split/Spall/Patched Area (1620)
- Masonry or Panel Displacement (1640)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.6.2 Masonry Abutment (Element 217)

Masonry abutments are solid, primary load-carrying members that support the superstructure. Similar to masonry wingwalls, masonry abutments also act as gravity retaining walls to contain the soil located under the approach. They rely purely upon their self-weight and any soil bearing on top of them to provide enough frictional resistance at their footings to prevent sliding. In addition, since unreinforced masonry cannot resist tensile stresses very well, they are built wide and heavy enough so that tensile stresses due to bending from lateral soil pressures are virtually eliminated.

Figure 2.5.6.2-1: Masonry Abutment

Element Level Inspection

On the inspection report form, masonry abutments are recorded in units of lineal feet. This measurement is taken as the length of the abutment along the skew (if present). The total quantity is the sum of all abutment lengths. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Level Inspection of masonry abutments should include the following items:

- Looking for cracked, split, spalled, loose or missing stone masonry units. This would suggest weathering due to freeze/thaw effects. Missing or crushed masonry units result in a loss of abutment cross sectional area, increasing the axial stresses.
- Checking the abutment surface for bulges. This defect suggests unstable soil, and the roadway above will also likely show signs of settlement.
- Looking for cracked, broken, deteriorated, loose or missing mortar. This would suggest weathering due to freeze/thaw effects. Missing or crushed mortar results in a loss of abutment cross sectional area, increasing the axial stresses.
- Looking for leaching. This indicates water is flowing through the mortar joints,
- Leaching out cementitious minerals. Long-term leaching will weaken the mortar.
- Checking areas exposed to drainage and roadway runoff. The runoff may cause scaling of the masonry units.
- Examining previous repair areas for soundness.

- Checking the abutment for plumb visually or using a plumb bob.
- Noting any abrasion of the masonry for abutments located within a waterway.
- Examining the top surface (bearing seat) of the abutment for cracking and spalling. Deterioration in these areas may be caused by frozen expansion bearings that transmit lateral forces to the pier not intended for in the original design.
- Checking to make sure weep holes in the abutment are functioning.
- Looking for vegetation growing inside of cracks or between the mortar and masonry unit. Plant roots can exert prying forces that further deteriorate these materials.
- Checking to make sure surface drains are functioning properly and not allowing water to penetrate the approach fill behind the abutment.
- Hammer tapping random and suspect areas to evaluate the masonry's soundness.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Mortar Breakdown (1610)
- Split/Spall/Patched Area (1620)
- Masonry or Panel Displacement (1640)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.5.6.2-2: Masonry Abutment with Moderate Mortar Deterioration – Condition State 2

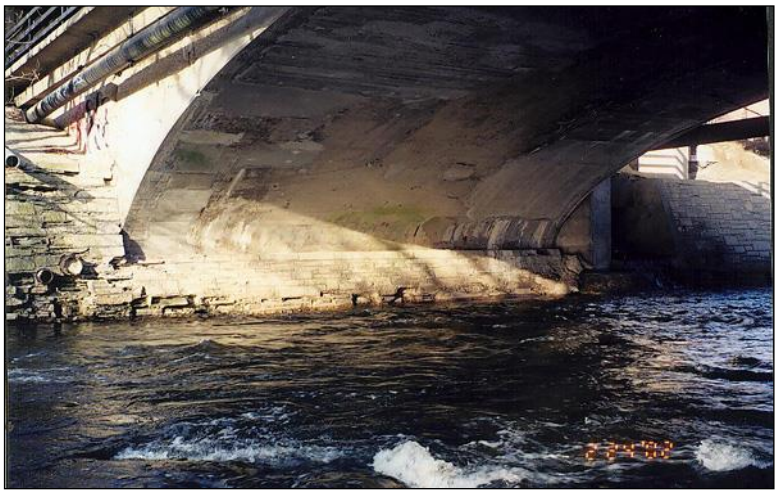

Figure 2.5.6.2-3: Masonry Abutment with Moderate Unit Deterioration – Condition State 3

Figure 2.5.6.2-4: Split and Spalled Masonry Units – Condition State 3

2.5.7 Other Material Substructure Elements

Other materials are intended to be used for members that are constructed of materials not otherwise defined.

2.5.7.1 Other Material Column (Element 203)

This element is for all other material columns regardless of protective system.

Element Level Inspection

Other columns are recorded in units of “each” on the inspection report form. Therefore, each element will be individually rated and assigned a Condition State. It is the inspector’s task to examine each column and reasonably assign the most severe Condition State to the entire element. This will quantify the element’s condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Areas (1080)
- Deterioration (1220)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.7.2 Other Material Pier Wall (Element 211)

This element defines those pier walls constructed of other materials. This is for all pier walls regardless of protective systems.

Element Level Inspection

On the inspection report form, other pier walls are recorded in units of lineal feet. This measurement is taken as the length of the wall from end to end along the skew (if present). The total quantity is the sum of all pier wall lengths. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)

- Delamination/Spall/Patched Areas (1080)
- Deterioration (1220)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.7.3 Other Material Abutment (Element 218)

This includes the sheet material retaining the embankment, and integral wingwalls and abutment extensions. This is for all abutments regardless of protective systems.

Element Level Inspection

On the inspection report form, other abutments are recorded in units of lineal feet. This measurement is taken as the length of the abutment along the skew (if present) and the lengths of monolithic wingwalls. The total quantity is the sum of all abutment and monolithic wingwall lengths. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering

indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Areas (1080)
- Deterioration (1220)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.7.4 Other Material Pile (Element 229)

This element defines other material piles that are visible for inspection. Piles exposed from erosion or scour and piles visible during an underwater inspection are included in this element. This element is for all other material piles regardless of protective system.

Element Level Inspection

Other piles are recorded in units of "each" on the inspection report form. Therefore, each element will be individually rated and assigned a Condition State. It is the inspector's task to examine each pile and reasonably assign the most severe Condition State to the entire element. This will quantify the element's condition and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Material Defects

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Areas (1080)
- Deterioration (1220)
- Distortion (1900)

Structural Defects

- Settlement (4000)
- Scour (6000)

Condition States Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow

- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.5.8 Substructure NBI Condition Ratings

Part of every Routine Inspection is rating the substructure according to the Federal Highway Administration (FHWA) General Condition Rating Guidelines. The numeric condition ratings of these guidelines describe existing bridge components as compared to their as-built condition. Ratings range from 9 to 0, with 9 describing components in excellent condition and 0 describing failed components.

Because only a single number is used to rate the substructure, the rating must characterize its overall general condition. The rating should not be used to describe local areas of deterioration, such as isolated heavy spalling. However, widespread heavy spalling would certainly influence the rating. A proper rating will therefore consider the severity of deterioration plus the extent to which it is distributed throughout the substructure.

National Bridge Inventory (NBI) ratings are used to evaluate the state of deterioration of the substructure material. Since material condition is independent of a bridge's load-carrying capacity, postings or original design capacities less than current legal loads will not influence the rating. Similarly, temporary substructure support does not change or improve the condition of the substructure material. Therefore, temporary strengthening methods will not influence the substructure rating.

The NBI general condition ratings found in the FHWA guidelines apply to decks, superstructures, and substructures. Ratings 9 to 6 apply to components built of any material, while ratings 5 to 0 mention specific defects or deterioration that can be applied to certain materials. Because the NBI general condition ratings apply to a wide range of components and materials, Wisconsin has developed supplemental rating guidelines. These supplemental rating guidelines are used to assist the inspector in properly assigning condition ratings to specific components constructed of the most commonly used materials. The general condition ratings, along with the Wisconsin supplemental rating guidelines for substructures, are as follows:

Code (Rating)	Description
N	**NOT APPLICABLE** **Wisconsin Supplemental Rating Guidelines:** Used only for culverts and spandrel arches where footings cannot be seen.
9	**EXCELLENT CONDITION** **Wisconsin Supplemental Rating Guidelines:** There are no noticeable or noteworthy deficiencies that affect the condition of the substructure. There may be insignificant scrape marks caused by drift or collision.

8 **VERY GOOD CONDITION –** no problems noted.

Wisconsin Supplemental Rating Guidelines:

Concrete Substructure – there may be shrinkage cracks, light scaling or insignificant spalling which does not expose reinforcing steel. There may be insignificant damage caused by drift or collision with no resulting misalignment. Corrective action is not required.

Steel Substructure – there may be insignificant damage caused by drift or collision with no resulting misalignment. Corrective action is not required.

Timber Substructure – there may be insignificant damage caused by drift or collision with no resulting misalignment. Corrective action is not required.

Masonry Substructure – there may be insignificant spalling of the masonry units. Damage caused by drift or collision may have occurred, but with no resulting misalignment. Corrective action is not required.

7 **GOOD CONDITION –** some minor problems.

Wisconsin Supplemental Rating Guidelines:

Concrete Substructure – there may be minor cracking with possible leaching or spalls with no detrimental effect on bearing area. Leakage of expansion devices may have caused minor cracking to start. Minor scouring may have occurred.

Steel Substructure – leakage of expansion devices may have started minor rusting without measurable section loss. Minor scouring may have occurred.

Timber Substructure – insignificant decay, cracking or splitting. Minor scouring may have occurred.

Masonry Substructure – there may be minor cracking or the mortar or spalls/cracking of the masonry units with no detrimental effect on bearing area. Minor scouring may have occurred.

6 **SATISFACTORY CONDITION –** structural elements show some minor deterioration.

Wisconsin Supplemental Rating Guidelines:

Concrete Substructure – minor deterioration or disintegration, spalls, cracking or leaching with little or no loss of bearing area. Shallow, local scouring may have occurred near the foundation.

Steel Substructure – corrosion, but no measurable section loss. Shallow, local scouring may have occurred near the foundation.

Timber Substructure – some initial decay, cracking or splitting. Fire damage is limited to surface scorching with no measurable section loss.

Masonry Substructure – minor deterioration or disintegration, spalls or cracking of the masonry units or mortar with little or no loss of bearing area. Shallow, local scouring may have occurred near the foundation.

5 **FAIR CONDITION –** all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.

Wisconsin Supplemental Rating Guidelines:

Concrete Substructure – measurable but minor section loss may exist, with possible exposed reinforcing steel. Scour may be progressive and/or is becoming more prominent with possible top of footing exposure. However, no misalignment or settlement is noted.

Steel Substructure – corrosion with measurable but minor section loss. Scour may be progressive and/or is becoming more prominent with possible top of footing exposure. However, no misalignment or settlement is noted.

Timber Substructure – moderate decay, cracking or splitting. A few secondary members may need replacement. Fire damage is limited to surface charring with minor, measurable section loss. There may be some exposure of piles as a result of erosion, reducing penetration.

Masonry Substructure – minor deterioration or disintegration, spalls or cracking of the masonry units and mortar with little or no loss of bearing area. Scour may be progressive and/or is becoming more prominent with possible top of footing exposure. However, no misalignment or settlement is noted.

4 **POOR CONDITION –** advanced section loss, deterioration, spalling or scour.

Wisconsin Supplemental Rating Guidelines:

Concrete Substructure – structural cracks and advanced deterioration. Additional backfilling is required. Extensive scouring or undermining of the footing is affecting the stability of the unit and requiring corrective action.

Steel Substructure – corrosion with extensive section loss. Additional cross bracing is required. Extensive scouring or undermining of the footing is affecting the stability of the unit and requiring corrective action.

Timber Substructure – substantial decay, cracking, splitting or crushing of primary members, requiring replacement. Fire damage with significant section loss that may reduce the load-carrying capacity of the member. Extensive exposure of piles as a result of erosion, thus reducing penetration and affecting the stability of the unit.

Masonry Substructure – structural cracks and advanced deterioration of the masonry units and mortar. Additional backfilling is required. Extensive scouring or undermining of the footing is affecting the stability of the unit and requiring corrective action.

3 **SERIOUS CONDITION –** loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.

Wisconsin Supplemental Rating Guidelines:

Concrete Substructure – severe disintegration. Reinforcing steel is exposed with advanced stages of corrosion. There is severe section loss in critical stress areas. Bearing areas are seriously deteriorated with considerable loss of bearing. Severe scouring or undermining of the footings affects the stability of the unit. Settlement may have occurred. Shoring is considered necessary (not just precautionary) to maintain the safety and alignment of the structure.

Steel Substructure – there is severe section loss in critical stress areas. Bearing areas are seriously deteriorated with considerable loss of bearing. Severe scouring or undermining of the footings affects the stability of the unit. Settlement may have occurred. Shoring is considered necessary (not just precautionary) to maintain the safety and alignment of the structure.

Timber Substructure – there is severe section loss in critical stress areas. Major fire damage is present that will substantially reduce the load-carrying capacity of the member. Bearing areas are seriously deteriorated with considerable loss of bearing. Settlement may have occurred. Shoring is considered necessary (not just precautionary) to maintain the safety and alignment of the structure.

Masonry Substructure – severe disintegration of the masonry units and mortar. There is severe section loss in critical stress areas. Bearing areas are seriously deteriorated with considerable loss of bearing. Severe scouring or undermining of the footings affect the stability of the unit. Settlement may have occurred. Shoring is considered necessary (not just precautionary) to maintain the safety and alignment of the structure.

2 **CRITICAL CONDITION –** advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.

Wisconsin Supplemental Rating Guidelines:

Concrete Substructure – concrete cap is soft and spalling with reinforcing steel exposed with no bond to concrete. The top of the cap is split or a column has undergone a shear failure. Scouring is sufficient that the substructure is near a state of collapse. Piers have settled.

Steel Substructure – members have critical section loss. Holes in the web and/or knife-edged flanges are typical. Scouring is sufficient that the substructure is near a state of collapse. Piers have settled.

Timber Substructure – the primary members are crushed or split and ineffective. Piers have settled.

Masonry Substructure – scouring is sufficient that the substructure is near a state of collapse. Piers have settled.

1 **"IMMINENT" FAILURE CONDITION –** major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structural stability. Bridge is closed to traffic but corrective action may put it back in light service.

Wisconsin Supplemental Rating Guidelines:

Bridge is closed. Corrective action may put it back in light service.

0 **FAILED CONDITION –** out of service, beyond corrective action.

Wisconsin Supplemental Rating Guidelines:

Bridge is closed. Replacement is necessary.

One suggested method for establishing a substructure rating is to identify phrases within the general condition/Wisconsin supplemental guideline language that describe a substructure condition more severe than what actually exists. The correct rating number will be one number higher than the one describing the more severe condition.

For example, suppose a reinforced concrete substructure has extensive delaminations, plus spalling with exposed reinforcing steel. The spalls occur on the tension side of the caps and

on random sides of the columns, but section loss of the reinforcing steel is minimal. Condition rating 4 indicates that there is advanced deterioration and spalling. Condition rating 3 indicates that deterioration and spalling have seriously affected the primary structural components, and that the reinforcing steel is in the advanced stages of corrosion. Using the method described above, Condition rating 3 describes a situation more severe than what actually exists on the substructure. Therefore, a rating of 4 would be appropriate.

Another method to help narrow down the substructure rating number is to group the numbers in more general categories. Ratings of 9 to 7 apply to substructures in good condition, 6 to 5 suggest fair condition, 4 to 3 suggest poor condition, 2 suggests poor/critical condition, and 1 to 0 suggest critical condition. It is also important to note that there is a significant change from a substructure in condition rating 5 (minor section loss, structural elements sound) to condition rating 4 (advanced section loss, advanced deterioration). A reduction in load-carrying capacity can be measured/calculated when a substructure enters condition rating 4.

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2.6 ROADWAY AND APPURTENANCES

2.6.1 Introduction

Bridge roadway and appurtenances refer to bridge related items that are not part of the deck, superstructure or substructure. The bridge will not collapse in the absence of these items. However, they are required for safety and serviceability. Roadways are the approach roadways located on either end of the bridge. Appurtenances are the bridge railings (parapets), expansion joints, protective coatings, and strengthening or repair systems.

2.6.2 Approaches

The primary function of the approach is to provide a smooth transition between the roadway pavement and the bridge deck. This smooth transition decreases the impact forces on the bridge superstructure and therefore increases bridge safety, as well as driver comfort. The length of approach is a judgment call, but in Wisconsin, the normal distance to rate would be from the abutment backwall to the nearest approach pavement joint. If the extent of the approach is unclear, inspect approximately 20 feet from the end of the bridge. The approach roadway width includes the shoulders if the paved shoulders are finished in concrete or bituminous pavement.

The pavement structure varies with the type of approach roadway. For bituminous approaches, the pavement structure consists of a bituminous wearing surface and either a concrete or bituminous sub base. For concrete approaches, it consists of a concrete slab with an aggregate sub base and a relief joint. The subgrade material for both types of approaches is the prepared and compacted soil or gravel immediately beneath the approach.

Vertical settlement is caused by the consolidation of the subgrade materials. Settlement is especially a problem near the abutment. Heave or uplift can also occur due to rotation of the abutment or expansion of frozen subgrade material. In addition, erosion can be a factor for failing approaches due to poor drainage along the edges of the roadway and bridge structure. Once erosion starts along the edges of the roadway, undermining can occur which will lead to further settlement of the approach slab. In certain instances, flood water can cause scour around the back side of the abutment. If evidence of flooding is apparent, special attention should be paid to ensure the backside of the abutment is fully protected.

The riding surface should be smooth, free of potholes, and properly sloped for drainage. Embankment slopes along the roadway shoulder should have adequate vegetation to provide erosion control. Roadway inlets located in the approach area should be in good condition and fully operational. Joints between the approach and the abutment backwall should be examined. Some of these joints are designed for thermal movement. During an inspection, a determination should be made whether or not there is adequate clearance to provide for thermal movement. If the joint was designed for a water seal, determine if the seal is adequate to prevent leakage.

A structural concrete approach is a concrete slab that bears on the abutment at one end. The opposite end bears on a concrete grade beam (a strip footing running the width of the approach). In Wisconsin, the structural approach slab is typically dowelled into the abutment creating a rigid connection. These types of concrete approaches are either reinforced or prestressed, and are designed to span between the abutment and grade beam. These types

of approaches are used because the approach fill is difficult to compact in the vicinity of an abutment and may settle over time. In a sense, this approach type is “a bridge to the bridge”.

Concrete structural approach slabs will always have a grade beam. Non-structural approach slabs will not have a grade beam, that is non-structural approach slabs rest on the paving notch at the abutment and on the approach fill adjacent to the bridge. Non-structural approach slabs may or may not be reinforced. If they are not reinforced or prestressed, extensive cracking may take place if the approach fill settles. The distinction between structural approach slabs and non-structural approach slabs is not always obvious and will need to be determined based off of plans prior to field activities As a general rule, more recently constructed or reconstructed state highway bridges carrying high traffic volumns will have a structural approach slab resting. Older state highway bridges and most non-state highway bridges will have non-structural approach slabs.

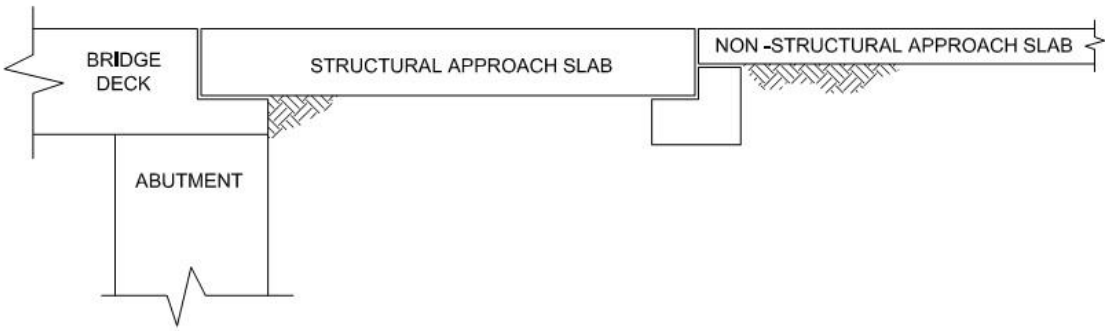

Figure 2.6.2.1-1 Structural Approach Slab Diagram

Concrete approaches on concrete roadways typically use a pavement relief joint between the approach and roadway pavement. The relief joint is a strip of asphalt or filler material that compresses as the roadway pavement expands or migrates towards the bridge. Compression of this relief joint minimizes the roadway pavement from pushing on the approach and abutment backwall.

A concrete approach may be completely overlaid with bituminous asphalt. Similar to bridge decks, the approach slabs are rated based on all exposed surfaces. In the instances of bituminous asphalt overlays, the condition of the bituminous asphalt should be inspected for signs of deterioration that would lead to the conclusion that the concrete approach slab has failed.

2.6.2.1 Prestressed Concrete Structural Approach Slab (Element 320)

Both Elements 320 and 321 are for structural approach slabs only. Non-structural approach slabs will be coded under the applicable Assessment. Structural approach slabs will have one end resting on the abutment paving notch, and the other end resting on a grade beam.

Element Level Inspection

On the inspection report form, a prestressed concrete structural approach slab is recorded in units of square feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the approach slabs condition and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of concrete approaches should include the following items:

- Looking for settlement or heaving of the approach roadway. If settlement of the slab or grade beam has occurred, there may be evidence of cracking.

- Examining the joint between the approach and the abutment backwall. Some of these joints are designed for thermal movement. A determination should be made whether or not there is adequate clearance to provide for this movement. If the joint was designed for a water seal, determine if the seal is adequate to prevent leakage.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Prestressing (1100)
- Cracking (PSC) (1110)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the

severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.2.2 Reinforced Concrete Structural Approach Slab (Element 321)

Both Elements 320 and 321 are for structural approach slabs only. Non-structural approach slabs will be coded under the applicable Assessment. Structural approach slabs will have one end resting on the abutment paving notch, and the other end resting on a grade beam.

Figure 2.6.2.2-1 Concrete Approach with Wide Cracks – Condition State 3

Element Level Inspection

On the inspection report form, a reinforced concrete structural approach slab is recorded in units of square feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the approach slabs condition and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of concrete approaches should include the following items:

- Looking for settlement or heaving of the approach roadway. If settlement of the slab or grade beam has occurred, there may be evidence of cracking.

- Examining the joint between the approach and the abutment backwall. Some of these joints are designed for thermal movement. A determination should be made whether or not there is adequate clearance to provide for this movement. If the joint was designed for a water seal, determine if the seal is adequate to prevent leakage.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delaminations/Spalls/Patched Areas (1080)
- Exposed Rebar (1090)
- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)
- Precast Concrete Connections (8906)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.3 Wearing Surfaces

Wearing surfaces are placed on top of the deck and protects the deck and provides a smooth riding surface. Wearing surfaces are incorporated in many new deck designs and are also a common repair or protective procedure for decks. The wearing surface elements extends to a depth of the bottom of the top mat of deck/slab reinforcement.

2.6.3.1 Wearing Surface – Other (Element 510)

This element is for all decks/slabs that have overlays other than the ones described below in this section. Examples would be FRP overlays, timber running planks, steel plates in wheel paths, etc.

Element Level Inspection

On the inspection report form, wearing surfaces are recorded in units of square feet. The square footage of a wearing surface does not need to equal the square footage of the deck. The area of the wearing surface will be limited to an equal or smaller area. Examples of smaller area occasions are timber decks with raised medians or raised sidewalks, which would not be included in the wearing surface element. In this case, only the section of wearing surface between the raised median/sidewalk would be counted as square feet under this element. In addition, bridge decks with concrete parapets would be measured similarly, between parapets and/or raised medians/sidewalks, i.e. only the driving surface. The area under the concrete parapet would not be counted towards the total area under this element.

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/Spall/Patched Area/Pothole – Wearing Surface (3210)
- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.3.2 Wearing Surfaces – Bare (Element 8000)

Decks, slabs, or top flanges without a wearing surface or overlay must have this element added. The top of the deck, slab, or top flange shall be evaluated under this element. This is done for simplicity by eliminating the necessity to overlap top and bottom material defects. By always having an assigned wearing surface to the top surface of a deck and slab element, the inspector and department have the added ability to provide specific maintenance recommendations for both the top and bottom of decks. This element shall be used on constructed decks with integral wearing surfaces placed on top of the deck (i.e. decks measuring approximately 1.5 inches thicker due to a sacrificial wearing surface). Integral wearing surfaces should not be confused with concrete overlays which are separate pours and not monolithic with the deck. Wearing Surface – Bare shall be used when a structural concrete overlay is placed on a deck. A structural overlay is thicker than a typical overlay and has a mat of steel reinforcement. These overlays are structurally tied into the deck making it essentially an extension of the older deck.

This element shall be used for all deck, slab or appropriate top flange elements regardless of construction material except Steel Deck – Open Grid (Element 28). Both the top and bottom condition of the Steel Deck – Open Grid element is easily inspected from either the top or bottom. All other deck and slab elements and those top flanges used as a driving surface without an overlay or wearing surface shall require this element for top surface evaluation.

When evaluating patches in wearing surfaces, the material of the patch will effect whether or not the patch can be considered sound. Patches of differing materials will not be considered sound (i.e. asphalt patch on a concrete overlay). The patch must be of the same material to be considered sound. If the patch and base materials are the same, then the patch can be inspected for delamination or other unsound indications. . If inspection shows a sound patch, the Condition State of the patched area may be raised to Condition State 2.

Element Level Inspection

On the inspection report form, wearing surfaces are recorded in units of square feet. The square footage of a wearing surface does not need to equal the square footage of the deck.

The area of the wearing surface will be limited to an equal or smaller area. Examples of smaller area occasions are timber decks with raised medians or raised sidewalks, which would not be included in the wearing surface element. In this case, only the section of wearing surface between the raised median/sidewalk would be counted as square feet under this element In addition, bridge decks with concrete parapets would be measured similarly, between parapets and/or raised medians/sidewalks, i.e. only the driving surface. The area under the concrete parapet would not be counted towards the total area under this element.

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/Spall/Patched Area/Pothole – Wearing Surface (3210)
- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)

When steel reinforcing or other reinforcing material becomes exposed through a wearing surface, the wearing surface in that area is no longer considered effective and shall be coded Condition State 4 for the appropriate measurement under defect Delaminiation/Spall/Patched Area/Pothole – Wearing Surface (3210).

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange

- Condition State 4 Severe Red

2.6.3.3 AC Overlay (Element 8511)

This element includes Chip seals.

Placing an Asphalt Concrete (AC) Overlay on top of a cracked or damaged bridge deck is a typical repair. An AC Overlay will typically be 1-3" in depth. AC Overlays will typically be used over distressed concrete or previous overlays to restore a smooth wearing surface. One disadvantage of AC Overlays is that expansion and contraction of the underlying pavement will typically cause cracks that are referred to as reflective cracks in the asphalt.

When evaluating patches in wearing surfaces, the material of the patch will effect whether or not the patch can be considered sound. Patches of differing materials will not be considered sound (i.e. asphalt patch on a concrete overlay). The patch must be of the same material to be considered sound. If the patch and base materials are the same, then the patch can be inspected for delamination or other unsound indications. If inspection shows a sound patch, the Condition State of the patched area may be raised to Condition State 2.

Element Level Inspection

On the inspection report form, wearing surfaces are recorded in units of square feet. The square footage of a wearing surface does not need to equal the square footage of the deck. The area of the wearing surface will be limited to an equal or smaller area. Examples of smaller area occasions are timber decks with raised medians or raised sidewalks, which would not be included in the wearing surface element. In this case, only the section of wearing surface between the raised median/sidewalk would be counted as square feet under this element. In addition, bridge decks with concrete parapets would be measured similarly, between parapets and/or raised medians/sidewalks, i.e. only the driving surface. The area under the concrete parapet would not be counted towards the total area under this element.

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/Spall/Patched Area/Pothole – Wearing Surface (3210)

- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.3.4 AC Overlay and Membrane (Element 8512)

This element includes Polymer Modified Asphalt (PMA) systems.

A membrane can be placed between an AC Overlay and a concrete deck to seal the concrete against salt and water infiltration which can corrode the reinforcing bars and lead to early deterioration of the bridge deck. Membranes are typically used on bridge deck rehabilitation projects. Since these membranes are not visible during the inspection, the inspector will need to review plans/maintenance rehabilitation history to verify the presence of a membrane.

When evaluating patches in wearing surfaces, the material of the patch will effect whether or not the patch can be considered sound. Patches of differing materials will not be considered sound (i.e. asphalt patch on a concrete overlay). The patch must be of the same material to be considered sound. If the patch and base materials are the same, then the patch can be inspected for delamination or other unsound indications. . If inspection shows a sound patch, the Condition State of the patched area may be raised to Condition State 2.

Element Level Inspection

On the inspection report form, wearing surfaces are recorded in units of square feet. The square footage of a wearing surface does not need to equal the square footage of the deck. The area of the wearing surface will be limited to an equal or smaller area. Examples of smaller area occasions are timber decks with raised medians or raised sidewalks, which would not be included in the wearing surface element. In this case, only the section of wearing surface between the raised median/sidewalk would be counted as square feet under this element. In addition, bridge decks with concrete parapets would be measured similarly, between parapets and/or raised medians/sidewalks, i.e. only the driving surface. The area under the concrete parapet would not be counted towards the total area under this element.

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/Spall/Patched Area/Pothole – Wearing Surface (3210)
- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.3.5 Thin Polymer Overlay (Element 8513)

Polymer overlays have strong adhesion to the concrete base underneath and elasticity so they are less susceptible to thermal movements and deflections from loading. These overlays also tend to have a rougher surface that can avert skidding and retain de-icing materials for a longer period of time.

When evaluating patches in wearing surfaces, the material of the patch will effect whether or not the patch can be considered sound. Patches of differing materials will not be considered sound (i.e. asphalt patch on a concrete overlay). The patch must be of the same material to

be considered sound. If the patch and base materials are the same, then the patch can be inspected for delamination or other unsound indications. . If inspection shows a sound patch, the Condition State of the patched area may be raised to Condition State 2.

Element Level Inspection

On the inspection report form, wearing surfaces are recorded in units of square feet. The square footage of a wearing surface does not need to equal the square footage of the deck. The area of the wearing surface will be limited to an equal or smaller area. Examples of smaller area occasions are timber decks with raised medians or raised sidewalks, which would not be included in the wearing surface element. In this case, only the section of wearing surface between the raised median/sidewalk would be counted as square feet under this element. In addition, bridge decks with concrete parapets would be measured similarly, between parapets and/or raised medians/sidewalks, i.e. only the driving surface. The area under the concrete parapet would not be counted towards the total area under this element.

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/Spall/Patched Area/Pothole– Wearing Surface (3210)
- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)

Condition State Commentary

Appendix ??A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green

- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.3.6 Concrete Overlay (Element 8514)

This type of overlay can be placed on any pavement type, including concrete, asphalt, and composite concrete and asphalt pavements. A concrete overlay can be bonded or unbounded. A bonded concrete overlay will be relatively thin (2-4") and can be used with surface distress exists but the overall structural integrity of the existing concrete still exists. An unbounded concrete overlay will be thicker (4-11") and will perform as a new pavement and the existing pavement will exist as a base. This will typically be used when the structural integrity of the existing pavement is in question.

When evaluating patches in wearing surfaces, the material of the patch will effect whether or not the patch can be considered sound. Patches of differing materials will not be considered sound (i.e. asphalt patch on a concrete overlay). The patch must be of the same material to be considered sound. If the patch and base materials are the same, then the patch can be inspected for delamination or other unsound indications. If inspection shows a sound patch, the Condition State of the patched area may be raised to Condition State 2.

Element Level Inspection

On the inspection report form, wearing surfaces are recorded in units of square feet. The square footage of a wearing surface does not need to equal the square footage of the deck. The area of the wearing surface will be limited to an equal or smaller area. Examples of smaller area occasions are timber decks with raised medians or raised sidewalks, which would not be included in the wearing surface element. In this case, only the section of wearing surface between the raised median/sidewalk would be counted as square feet under this element. In addition, bridge decks with concrete parapets would be measured similarly, between parapets and/or raised medians/sidewalks, i.e. only the driving surface. The area under the concrete parapet would not be counted towards the total area under this element.

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects.

However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/Spall/Patched Area/Pothole – Wearing Surface (3210)
- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)

When steel reinforcing or other reinforcing material becomes exposed through a wearing surface, the wearing surface in that area is no longer considered effective and shall be coded Condition State 4 for the appropriate measurement under defect Delaminiation/Spall/Patched Area/Pothole – Wearing Surface (3210).

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.3.7 Polyester Concrete Overlay (Element 8515)

When polyester resin is used in Portland Cement Concrete it has as rapid cure time of 2-4 hours, has greater flexural strength than conventional concrete, and is typically 1” thick.

When evaluating patches in wearing surfaces, the material of the patch will effect whether or not the patch can be considered sound. Patches of differing materials will not be considered sound (i.e. asphalt patch on a concrete overlay). The patch must be of the same material to be considered sound. If the patch and base materials are the same, then the patch can be inspected for delamination or other unsound indications. If inspection shows a sound patch, the Condition State of the patched area may be raised to Condition State 2.

Element Level Inspection

On the inspection report form, wearing surfaces are recorded in units of square feet. The square footage of a wearing surface does not need to equal the square footage of the deck. The area of the wearing surface will be limited to an equal or smaller area. Examples of smaller area occasions are timber decks with raised medians or raised sidewalks, which would not be included in the wearing surface element. In this case, only the section of

wearing surface between the raised median/sidewalk would be counted as square feet under this element. In addition, bridge decks with concrete parapets would be measured similarly, between parapets and/or raised medians/sidewalks, i.e. only the driving surface. The area under the concrete parapet would not be counted towards the total area under this element.

Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Delamination/Spall/Patched Area/Pothole/Pothole – Wearing Surface (3210)
- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.4 Median and Sidewalk Elements

These elements cover the area of all raised medians and sidewalks that are on top of the deck or slab element. These elements do not include non-raised (at grade) or painted medians or sidewalks.

2.6.4.1 Median (Element 8207)

This element is for medians measured on the top side by the SF of exposed horizontal surface. Exposed sides or bottom are not measured. Note any defects observed on the exposed top, bottom, and edge surfaces captured using the condition states. Record the controlling top, bottom, or edge defect condition state for each SF of the element.

Medians located within the bridge approach should not be rated as part of the approach element/assessment; record the condition and findings in the narrative under the structural approach slab (Element 321) or approach roadway (Assessment 9322, 9323 or 9324). A railing/parapet as part of the median with or without curb assessed as a railing element is not measured and assessed as Element 8207 Median. A median that is placed over fill is coded in the evaluation of the Roadway Over Structure assessment 9325.

2.6.4.2 Sidewalk (Element 8209)

This element is for sidewalks raised from the deck/slab and greater than 18 inches wide measured on the top side by the SF of exposed horizontal surface. Exposed sides or bottom are not measured. Note any defects observed on the exposed top, bottom, and edge surfaces captured using the condition states. Record the controlling top, bottom, or edge defect condition state for each SF of the element. For timber sidewalks, use timber defects identified in Chapter 3.D of the field manual.

Additional sidewalk or pedestrian path evaluations:

- Condition of a sidewalk or pedestrian path located on a bridge approach should not be rated with the approach element/assessment; record the condition and findings in the narrative under the structural approach slab (Element 321) or approach roadway (Assessment 9322, 9323 or 9324).

- Sidewalk or pedestrian paths that are not raised above the deck/slab are assessed as part of the wearing surface element.

- The condition of a sidewalk over a buried structure is to be documented in the narrative for assessment 9325 Roadway Over Structure. Do not consider the condition of the sidewalk over when rating assessment 9325 unless in the inspector's judgement, the sidewalk condition is having a detrimental effect on the structure condition.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Debonding/Delaminations/Spalls/Patched Area/Pothole – Wearing Surface (3210)
- Crack – Wearing Surface (3220)
- Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911)
- Deterioration/Distortion – Wearing Surface (8912)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.5 Joint Elements

Expansion joints provide for thermal expansion and contraction of the deck and superstructure. The clear opening of the joint should provide for adequate space for movement of the adjacent superstructure elements. Joints also fill the gap between deck and abutment backwall to provide a smooth ride for vehicles transitioning onto and off the bridge. They must also be durable enough to withstand the abuse from traffic loads, snow plow blades, road debris, sunlight, freeze/thaw, and deicing chemicals.

The primary components of a deck joint are the anchor system, the support system, and the joint material. The concrete on either side of the opening into which the anchorage systems are cast is often called the “header”.

The two major categories of deck joints are closed joints and open joints. Closed joints are designed so that water and debris do not pass through them. The five types of closed joints are strip seals, poured joint seals, compression seal/cellular seal, modular strip seal, and elastomeric expansion seal. Open joints allow water and debris to pass through the joints. The four types of open joints are open/formed joints, finger plate joints, sliding plate joints, and open expansion joints.

Construction joints and longitudinal bridge joints should not be coded under this element. In addition, joints between the deck and non-structural approach slabs shall not be inventoried

as a joint. It may be necessary to review bridge plans to distinguish between sealed construction joints and pourable joint seals. Joints will be inventoried when there is a discontinuity in deck reinforcement, superstructure, or if bearings support the adjacent superstructure section and provide room for expansion and contraction. In some cases, there is an expansion joint at the end of the structural approach slab, between the structural and non-structural approaches. If this joint provides room for expansion and contraction, then this joint shall be coded as an element.

2.6.5.1 Strip Seal Expansion Joint (Element 300)

Strip seal expansion joints are commonly used in Wisconsin today. Strip seals are "V" shaped neoprene glands used at expansion joints to prevent water from falling through the joint and corroding the girders, bearings, and other elements below. They are formed with bulbs along their edges that lock and are glued into steel extrusions on either side of the opening. The steel extrusions are anchored to either the deck or backwall.

Single strip seals typically have a normal expansion gap of 4 inches. With superstructure expansion and contraction, the seal folds or opens up while providing a continuous seal across the opening. If there is not enough slack in the gland, the gland may pull out of the anchorage and leak. The gland may also fill with debris and, when compressed, the gland may tear and leak.

Element Level Inspection

On the inspection report form, a strip seal expansion joint is recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of strip seal expansion joints should include the following items:

- Checking for debris that has filled the joint. Road debris may contain sharp objects that can puncture or rip the gland when run over by vehicles. It can also prevent proper folding of the gland during superstructure expansion.
- Removing debris from the joint to examine the gland for tears or punctures (if this removal can be done safely).
- Looking for areas where the strip seal has pulled out of the extrusion.
- Examining the backwall and bearing seat for dampness, water stains or debris accumulation that indicate a leak in the strip seal.
- Measuring the opening between steel anchorages and the temperature when the measurement was taken.

- Checking the vertical alignment across the joint. Impact on the bridge is increased when each side of the joint opening is at a different elevation.

- Noting any indiscriminate bituminous overlays that were placed over the joint. This prevents proper joint operation. Asphalt paving over an expansion joint is considered debris impaction.

- Checking to make sure the steel anchorages are secure. Loose anchorages could damage a vehicle.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Leakage, Seal Adhesion/Damage/Cracking (2310)

- Debris Impaction (2350)

- Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green

- Condition State 2 Fair Yellow

- Condition State 3 Poor Orange

- Condition State 4 Severe Red

2.6.5.2 Pourable Joint Seal (Element 301)

A poured joint seal is made from compressible filler, backer rod, and a poured sealant. The compressible filler is set 1 to 2 inches from the top of the deck. In the space above the filler is a backer rod for a bond breaker and an asphalt or polymer-based pourable sealant.

The poured joint seal can accommodate movement of about 1/4 inch, so it is usually found at fixed joints or at expansion joints for very short-span structures.

Element Level Inspection

On the inspection report form, a pourable joint seal is recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of pourable joint seals should include the following items:

- Checking the joint for deterioration due to normal weathering, wear, and tear.
- Looking along the joint for loss of adhesion.
- Looking for cracks in the joint material.
- Examining the backwall and bearing seat for dampness or water stains that indicate loss of adhesion to the deck or backwall.
- Checking the adjacent deck for spalling which is causing loss of adhesion.
- Looking for debris that has impacted the joint material.
- Checking the vertical alignment across the joint. Impact on the bridge is increased when each side of the joint opening is at a different elevation.
- Noting any indiscriminate bituminous overlays that were placed over the joint.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Leakage, Seal Adhesion/Damage/Cracking (2310)
- Debris Impaction (2350)
- Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.6.5.2-1 Poured Joint Seal – Condition State 3

2.6.5.3 Compression Joint Seal (Element 302)

A compression joint seal is a rectangular neoprene tube with an internal honeycomb cross-section. The seal is compressed as it is inserted (and sometimes glued) into a joint armored with steel angles. The honeycomb structure functions like a spring and allows the compression seal to expand against the sides of the joint as the joint moves.

Another type of compression seal is the cellular seal. The cellular seal is made of closed cell foam that allows the joint to move in different directions.

A large compression seal can accommodate expansion movements up to approximately 2 inches. However, they are most often used at fixed joints where limited movements are allowed. These types of joints tend to leak over time.

Element Level Inspection

On the inspection report form, a compression joint seal is recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of compression joint seals should include the following items:

- Checking the compression seal for deterioration due to normal weathering, wear, and tear.
- Looking along the joint for loss of adhesion or where the compression seal has pulled out or been driven down into the opening.
- Examining the backwall and bearing seat for dampness, water stains or debris accumulation that indicates a leak in the compression seal.
- Looking for debris that has impacted the compression seal.
- Checking for debris that has filled the joint, since these joint types are usually installed slightly recessed from the driving surface. Road debris may contain sharp objects that can puncture or rip the compression seal when run over by vehicle wheels.
- Removing debris from the joint to examine the seal for tears or punctures (if the removal can be safely done).
- Measuring the opening between steel anchorages or concrete headers. The temperature should be recorded as well.
- Checking the adjacent deck or backwall for spalls when steel angle armor is not used. When steel armor is used, check for spalls behind the steel armor, as well as debonding of the steel armor.
- Checking the vertical alignment across the joint. Impact on the bridge is increased when each side of the joint opening is at a different elevation.
- Noting any indiscriminate bituminous overlays that were placed over the joint. This prevents proper operation of the joint.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Leakage, Seal Adhesion/Damage/Cracking (2310)
- Debris Impaction (2350)
- Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.6.4.3-1: Compression Joint Seal Impacted with Debris and Pushed Down Into the Joint Opening (Far Lane)– Condition State 4.

Figure 2.6.4.3-2: Compression Seal Pushed Down Into the Joint Opening - Condition State 3.

2.6.5.4 Modular Joint (Element 303)

A modular joint is a series of strip seals connected by intermediate steel separation beams. The steel member anchored to the deck or backwall is called the edge beam. The beams are supported by a stringer system to help carry traffic loads across these wider joints. They are mainly used on long span structures with large expansion lengths, having a normal range of movement between 4 and 24 inches. However, these joints can be fabricated to accommodate movements up to 48 inches.

Element Level Inspection

On the inspection report form, a modular joint is recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of modular joints should include the following items:

- Checking for debris that has filled the joint. Road debris may contain sharp objects that can puncture or rip the compression seal when run over by vehicle wheels. It can also prevent proper folding of the gland during superstructure expansion.
- Removing debris from the joint to examine the glands for tears or punctures (if the removal can be safely done).
- Looking for areas where the strip seal has pulled out of the extrusion.
- Listening for any rattles or indications of component looseness as traffic drives over the joint.
- Looking for any broken separation beams.
- Examining the backwall and bearing seat for dampness, water stains or debris accumulation that indicate a leak in one of the strip seals.
- Measuring the opening between the steel edge beams (the intermediate steel separation beams will therefore be included in this measurement). The temperature should be recorded as well.
- Checking the vertical alignment across the joint. Impact on the bridge is increased when each side of the joint opening is at a different elevation.
- Noting any indiscriminate bituminous overlays that were placed over the joint. This prevents proper joint operation.
- Looking for surface damage to seals and separation beams.
- Examining the underside for evidence of leakage and fractured welds or bolts. Check for free movement of all segments.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Leakage, Seal Adhesion/Damage/Cracking (2310)
- Debris Impaction (2350)
- Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

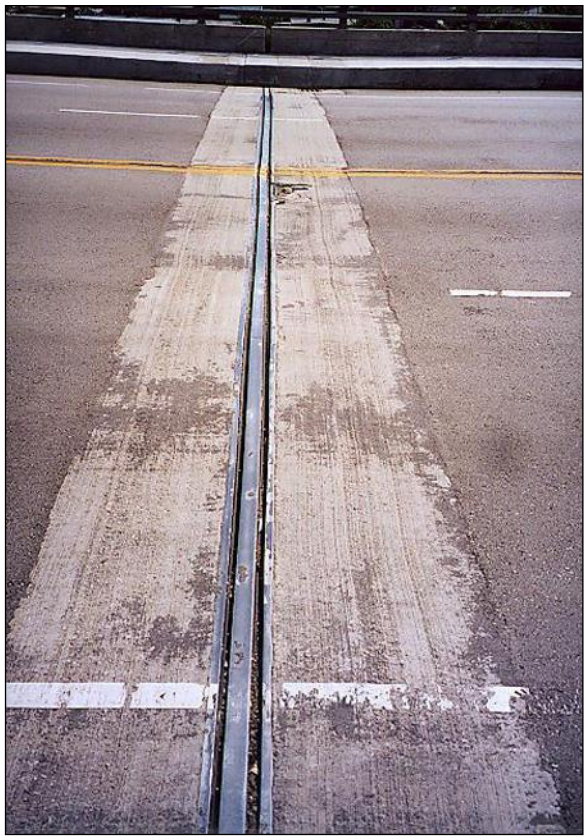

Figure 2.6.5.4-1 Modular Expansion Joint – Condition State 1

2.6.5.5 Open Expansion Joint (Element 304)

Open expansion joints, sometimes called formed joints, are little more than a gap between the bridge deck and the abutment backwall or between adjacent deck sections. Open joints allow water and debris to pass directly through the joint and directly onto the girders and

bearings below. Because this water and debris can quickly corrode steel and lock up expansion bearings, use of open expansion joints is highly discouraged in modern designs.

Open expansion joints are usually unprotected, but the deck slab and backwall can be armored with steel angles. Open joints are common on short span bridges with concrete decks.

Element Level Inspection

On the inspection report form, an open expansion joint is recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of open expansion joints should include the following items:

- Checking the corners of the deck and backwall for cracks and spalls.
- Examining the backwall and bearing seat for excessive corrosion and debris accumulation.
- Checking the adjacent deck for spalling or cracking which is causing loss of armor anchorage.
- Checking the vertical alignment across the joint. Impact on the bridge is increased when each side of the joint opening is at a different elevation.
- Measuring the opening. The temperature should be recorded as well.
- Noting any indiscriminate bituminous overlays that were placed over the joint. Overlay material wedged inside of the opening can prevent proper movement of the superstructure and introduce loads not intended for in the original design.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Leakage, Seal Adhesion/Damage/Cracking (2310)
- Debris Impaction (2350)
- Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.5.6 Assembly Joint Without Seal (Element 305)

Some examples of these joints are Finger Plate joints and Sliding Plate joints. Finger plate joints, also known as a tooth plate joints, consist of two steel plates with overlapping fingers to provide for a smooth riding surface. These joints are usually found on longer span bridges where greater expansion is required. They may allow water and road debris to pass through the opening directly onto the superstructure below or a flexible trough is placed underneath of the opening to direct water and debris away, as with current design practice. The two types of finger plate joints are supported and cantilever.

The supported finger plate joint is used on longer spans. The fingers on this joint have their own support system in the form of transverse beams under the joint. Some types of finger plate joints are segmental, allowing for maintenance and replacement if necessary. Cantilever joints have no support beams.

Sliding plate joints are designed to allow one plate to slide over the top of another. This joint is usually not watertight so an elastomeric sheet is sometimes used to seal the joint. Pourable joint fillers are also sometimes used to seal the openings, but these are usually only effective in the short term. The sliding plate joint can accommodate a maximum movement of 4 inches.

Element Level Inspection

On the inspection report form, a finger or sliding plate joint is recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of finger plate, sliding plate, and other expansion joints should include the following items:

- Looking for broken teeth on finger joints.
- Looking for debris lodged between the plates of a sliding plate joint
- Checking the horizontal and vertical alignment of the joint elements. In a finger plate joint, the individual fingers should mesh together without interference and should match the same plane as the deck surface. Vertical misalignments cause additional impact forces on the joint and bridge.
- Measuring the joint opening. On finger plate joints, this measurement should be taken from a fingertip of one plate to the cut-out root of the opposing plate. On sliding plate joints, this measurement is taken from the tip of the top plate to the joint armor vertical face on the opposite side of the opening. The temperature should be recorded as well.
- Listening for any rattles or indications of component looseness as traffic drives over the joint.
- Checking the support condition from below the deck. Look for broken welds and corrosion.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Leakage, Seal Adhesion/Damage/Cracking (2310)
- Debris Impaction (2350)
- Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow

- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.6.4.6-1: Sliding Plate Expansion Joint (Note Transverse Deck Crack on Left Side Suggesting a Failing Anchorage System) – Condition State 2.

Figure 2.6.4.6-2: Finger Plate Expansion Joint – Condition State 1.

2.6.5.7 Other Joint (Element 306)

Other joint will be a place holder for any type of joint that does not fit into one of the elements listed above.

Element Level Inspection

On the inspection report form, Other Joint is recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Leakage, Seal Adhesion/Damage/Cracking (2310)
- Debris Impaction (2350)
- Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.6 Strengthening or Repair Systems

2.6.6.1 FRP (Element 8800)

This defines Fiber-Reinforced Polymer (FRP) wrapped columns and pier caps as well as FRP used for strengthening slabs and beams. FRP wraps are typically used to provide structural capacity to columns, beams, or slabs that have impact damage, corrosion, or other types of deterioration. FRP will consist of all strengthening repairs made to various structural members.

Each FRP wrap is evaluated individually. Patched areas under FRP wraps shall be CS 2 or worse, depending on the element condition prior to FRP wrap installation. If an element is partially FRP wrapped, the FRP and concrete would be coded separately with the various associated units (SF, LF, EA).

Element Level Inspection

On the inspection report form, FRP wrap is recorded in units of "each". It is the inspector's task to examine each FRP wrap and reasonably assign the most severe defect Condition State to the entire element. This will quantify wraps' state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Deterioration (1220)
- Distortion (1900)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange

- Condition State 4 Severe Red

2.6.6.2 Jacketing (Element 8801)

This defines jacketing an element with steel or concrete to add structural capacity to the element. This is frequently used in areas with seismic activity to strengthen columns and beams. Jacketing will consist of all jackets found on substructure units, regardless of material type. Examples include concrete jacket on timber column, steel jacket on concrete column, steel jacket on timber column, concrete jacket on a concrete column etc.

Each jacket is evaluated individually. The underlying material to which the jacket is applied is considered to be repaired prior to the proper installation of the jacket. The condition state of the underlying concrete is increased to a condition state 1 due to this repair prior to jacket installation. If an element is partially jacketed, the jacket and concrete would be coded separately with the various associated units (SF, LF, EA). However, if a timer pile is jacketed, the timber will typically not be repaired prior to jacket installation. In this case, the timber pile will remain in the previous condition state. The jacket will then be coded by the exposed surfaces. Patched concrete not covered by a jacket is coded as condition state 2.

Element Level Inspection

On the inspection report form, Jacketing is recorded in units of “each”. It is the inspector’s task to examine each Jacketing and reasonably assign the most severe defect Condition State to the entire element. This will quantify wraps’ state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity..

- Corrosion (1000)
- Cracking (Steel) (1010)
- Connection (1020)
- Delaminations/Spalls/Patch Areas (1080)
- Exposed Rebar (1090)
- Cracking (PSC) (1110)
- Cracking (RC) (1130)

- Abrasion/Wear (PSC/RC) (1190)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.6.3 Culvert Liner (Element 8802)

This defines elements that are inserted into culverts to add structural stability due to excessive erosion, deterioration, corrosion, or other defect to the original culvert material. This system is used to strengthen and stabilize a deteriorated culvert. If a culvert has a culvert liner, the original culvert will typically not be repaired prior to liner installation. In this case, the original culvert material will remain in the previous condition state noted prior to liner installation. The liner will then be evaluated by the exposed surfaces.

Element Level Inspection

On the inspection report form, culvert liners are recorded in units of linear feet along the barrel of the culvert, typically perpendicular to the roadway. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Deterioration (1220)

- Distortion (1900)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.6.4 External Post Tensioning (Element 8803)

This element defines the system used to strengthen a beam, column, cap, or footing by means of external post tensioning. This is typically necessary when existing tendons are damaged, corroded, cut, broken, or have lost their tensioning force. External post tensioning will typically be attached to the existing structure and possibly encased in concrete for added protection. External Post Tensioning will consist of steel material. This element is not to be used with Steel Tension Rods/Post Tensioning Cables (Element 8165).

Element Level Inspection

On the inspection report form, External Post Tensioning is recorded in units of “each”. It is the inspector’s task to examine each repair and reasonably assign the most severe defect Condition State to the entire element. This will quantify the approach’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Corrosion (1000)
- Cracking (Steel) (1010)
- Connection (1020)

- Delaminations/Spalls/Patch Areas (1080)
- Exposed Rebar (1090)
- Cracking (PSC) (1110)
- Cracking (RC) (1130)
- Abrasion/Wear (PSC/RC) (1190)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.7 Protective Coatings

With increasing research, the uses of protective systems are increasing the life of bridge decks. Many bridge decks require repair before the components of the bridge structure. Protecting the bridge deck from contamination and deterioration is becoming more common practice in order to extend the life of a bridge.

2.6.7.1 Concrete Protective Coating (Element 521)

This element is a place holder and should not be used at this time. This element does not include any type of painting or other sealing on any other concrete surface. Painted concrete surfaces should be evaluated under Aesthetic Treatments (Assessment 9010).

Future uses for this element may include concrete elements that have a protective coating applied to them. These coatings include silane/siloxane water proofers, crack sealers such as High Molecular Weight Methacrylate (HMWM), or any top coat barrier that protects concrete from deterioration and reinforcing steel from corrosion.

Element Level Inspection

This element is a place holder and thus should not be on the inspection report form.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Effectiveness (3540)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.7.2 Steel Protective Coating - Other (Element 515)

This element is for all Steel Protective Coatings other than the ones described below in this section.

Protective coatings are to be evaluated separately from the primary material element, regardless of the condition of the element it protects. Steel Protective Coatings (Elements 515, 8516, 8517, 8518 and 8519) will be calculated for all steel superstructure elements, as well as primary steel substructure elements (pier caps, columns, towers, and abutments).

The presence of lead paint is possible and should be considered when performing inspections of steel protective coatings. Any feasible actions for protective coatings should also identify the presence lead paint.

Element Level Inspection

On the inspection report form, protective coatings are recorded in units of square feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

The sum of all of the reported condition states must equal the total quantity of all coated superstructure or substructure elements. All superstructure or substructure elements with a steel protective coating will be quantified and listed under the corresponding primary superstructure or substructure element. An example would be a four-girder bridge with a floor system would have the protective coating quantity for the girders, floor beams, stringers, gusset plates, bracing, etc. This quantity the total quantity for the all of the elements listed would then be combined and placed under the primary superstructure element (steel girder). Similarly, a steel bent with painted piles and painted cross bracing would have the total quantity for all elements/assessments placed under the primary substructure element (Steel Pile).

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Effectiveness (3440)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.7.3 Painted Steel (Element 8516)

Paint is a typical protective coating applied to steel after erection. Depending on the environment, paint can act as a barrier between the steel and corrosive elements for a significant amount of time before maintenance is required. If the paint is scratched or removed in any way the steel underneath will be exposed and can begin to corrode.

Protective coatings are to be evaluated separately from the primary material element, regardless of the condition of the element it protects. Steel Protective Coatings (Elements 515, 8516, 8517, 8518 and 8519) will be calculated for all steel superstructure elements, as well as primary steel substructure elements (pier caps, columns, towers, and abutments).

The presence of lead paint is possible and should be considered when performing inspections of steel protective coatings. Any feasible actions for protective coatings should also identify the presence lead paint.

Element Level Inspection

On the inspection report form, protective coatings are recorded in units of square feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

The sum of all of the reported condition states must equal the total quantity of all coated superstructure or substructure elements. All superstructure or substructure elements with a steel protective coating will be quantified and listed under the corresponding primary superstructure or substructure element. An example would be a four-girder bridge with a floor system would have the protective coating quantity for the girders, floor beams, stringers, gusset plates, bracing, etc. This quantity the total quantity for the all of the elements listed would then be combined and placed under the primary superstructure element (steel girder).

Similarly, a steel bent with painted piles and painted cross bracing would have the total quantity for all elements/assessments placed under the primary substructure element (Steel Pile).

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Effectiveness (3440)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.7.4 Weathering Steel (Element 8517)

Weathering steel was developed to eliminate the need for further protective coatings to be added after erection. The steel will develop a rust-like layer after being exposed to the atmosphere that will protect it from further corrosion. This rust-like layer is effectively acting as a protective coating and will be evaluated as such.

Weathering Steel is to be evaluated separately from the primary material element, regardless of the condition of the element it protects. Weathering Steel quantities shall be calculated for all steel superstructure elements comprised of weathering steel. Note that weathering steel blasted and painted does not constitute, two separate protective systems, nor a duplex system. In this case, the area painted would be evaluated under Painted Steel and the remaining portion of weathering steel with a protective patina would be evaluated under Weathering Steel.

Element Level Inspection

On the inspection report form, protective coatings are recorded in units of square feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

The sum of all of the reported condition states must equal the total quantity of all coated superstructure or substructure elements. All superstructure or substructure elements with a steel protective coating will be quantified and listed under the corresponding primary superstructure or substructure element. An example would be a four-girder bridge with a floor system would have the protective coating quantity for the girders, floor beams, stringers, gusset plates, bracing, etc. This quantity the total quantity for the all of the elements listed would then be combined and placed under the primary superstructure element (steel girder). Similarly, a steel bent with painted piles and painted cross bracing would have the total quantity for all elements/assessments placed under the primary substructure element (Steel Pile).

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Oxide Film Degradation Color/Texture Adherence – Weathering Steel (3430)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.7.5 Galvanization (Element 8518)

The process of applying a protective zinc coating to steel is referred to as galvanizing. This process will protect the steel from rusting through the ability of the zinc to act as a sacrificial anode rather than the steel. If the protective coating is scratched or removed the steel will still be protected from corrosion by the remaining zinc through self-repair, which a distinct advantage over paint and powder coating. The layer of zinc is effectively acting as a protective coating and will be coded as such.

Protective coatings are to be evaluated separately from the primary material element, regardless of the condition of the element it protects. Steel Protective Coatings (Elements 515, 8516, 8517, 8518 and 8519) will be calculated for all steel superstructure elements, as well as primary steel substructure elements (pier caps, columns, towers, and abutments).

Element Level Inspection

On the inspection report form, protective coatings are recorded in units of square feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

The sum of all of the reported condition states must equal the total quantity of all coated superstructure or substructure elements. All superstructure or substructure elements with a steel protective coating will be quantified and listed under the corresponding primary superstructure or substructure element. An example would be a four-girder bridge with a floor system would have the protective coating quantity for the girders, floor beams, stringers, gusset plates, bracing, etc. This quantity the total quantity for the all of the elements listed would then be combined and placed under the primary superstructure element (steel girder). Similarly, a steel bent with painted piles and painted cross bracing would have the total quantity for all elements/assessments placed under the primary substructure element (Steel Pile).

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects.

However, only the controlling defect will be counted in the total element condition state quantity.

- Effectiveness (3440)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.7.6 Duplex System (Element 8519)

A duplex protective system includes multiple systems protecting steel elements, specifically paint or power coating over hot-dip galvanized material. This is used to utilize maximum corrosion protection while remaining aesthetically pleasing. Combining the coatings delivers superior cathodic and barrier protection from the galvanizing and some barrier protection from the paint or powder. Once the paint or powder fails the galvanizing will still provide significant protection. The additional initial cost of both hot-dip galvanizing and painting or powder coating the steel will allow for added savings through minimal maintenance when compared to lesser protective coatings.

Protective coatings are to be evaluated separately from the primary material element, regardless of the condition of the element it protects. Steel Protective Coatings (Elements 515, 8516, 8517, 8518 and 8519) will be calculated for all steel superstructure elements, as well as primary steel substructure elements (pier caps, columns, towers, and abutments).

The presence of lead paint is possible and should be considered when performing inspections of steel protective coatings. Any feasible actions for protective coatings should also identify the presence lead paint.

Element Level Inspection

On the inspection report form, protective coatings are recorded in units of square feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify

the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

The sum of all of the reported condition states must equal the total quantity of all coated superstructure or substructure elements. All superstructure or substructure elements with a steel protective coating will be quantified and listed under the corresponding primary superstructure or substructure element. An example would be a four-girder bridge with a floor system would have the protective coating quantity for the girders, floor beams, stringers, gusset plates, bracing, etc. This quantity the total quantity for the all of the elements listed would then be combined and placed under the primary superstructure element (steel girder). Similarly, a steel bent with painted piles and painted cross bracing would have the total quantity for all elements/assessments placed under the primary substructure element (Steel Pile).

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the lowest numbered defect controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

- Effectiveness (3440)

When the paint system of a duplex protective system begins to show signs of failure, but the underlying galvanization remains intact and functional, the Condition State for defect Effectiveness shall be considered higher than 4. Only when the bare steel of the element becomes exposed should the protective system be considered completely compromised.

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.7.7 Concrete Reinforcing Steel Protective System (Element 520)

Conventional steel reinforcement corrosion causes detrimental effects on concrete decks. Epoxy coating is often used on all conventional steel deck reinforcement to prevent corrosion. The epoxy coating is resistant to chemicals, water, and atmospheric moisture. Epoxies utilize an epoxy polymer binder that forms a tough, resilient film upon drying and curing. Drying is by solvent evaporation, while curing entails a chemical reaction between the coating components.

Element 520 refers to visible cathodic protection systems only. Other coatings not visible (i.e. epoxy coated rebar) cannot be coded. Reinforcing protective system are denoted as part of the element used for decks and slabs. Select “set rebar” under the appropriately selected deck or slab element:

Black steel refers historically used uncoated bar steel reinforcement

Coated reinforcing steel is typically coated in epoxy in a factory to protect against corrosion.

Stainless steel has been utilized on high cost structures with very high traffic volumes. The corrosion process is negligible when stainless steel reinforcement is used. Solid stainless steel reinforcement bars can be used due to its corrosion resistance being greater than conventional reinforcement and an estimated service life of 100 years. Stainless steel coating / cladding can be used on conventional reinforcement to which will protect the reinforcement from water and air and quickly reform if the surface is scratched.

Non-metallic systems including Fiber-Reinforced Polymer (FRP), Glass Fiber Reinforced Plastic (GFRP), etc.

Reinforcing protective systems will have same area as the deck or slab. On initial inspections, existing plans should be verified to conclude reinforcing protective system used (e.g. epoxy coated).

2.6.8 Traffic Safety Features

Traffic safety features on a bridge include concrete barriers (parapets), railings, sidewalks, and curbs. Barriers (parapets) and railings prevent errant vehicular traffic from driving off the bridge or into opposing traffic. Sidewalks provide a safe path away from vehicular traffic for pedestrians and bicycles. Curbs, either as part of the sidewalk or base of a railing, provide a means to keep the wheels of vehicles (and therefore the vehicles themselves) from hitting pedestrians or the railing.

Bridge Railings

Bridge railings are constructed of concrete, steel, aluminum, timber or a combination of these. Railings built of solid concrete are often referred to as parapets. The primary functions of the bridge railing are to keep errant vehicles from driving off the bridge and to smoothly redirect these vehicles in such a manner that they do not overturn. Another function is to protect the main load-carrying elements of certain superstructure types (such as through trusses and arches) from damage due to vehicular impacts.

The bridge railing evaluation includes the curb. The curb will be evaluated using the material defects of which the curb is constructed, regardless if constructed of a different material from the bridge railing. The height of a barrier will determine if it is considered the railing or a curb.

- If the height of a bridge barrier is greater than or equal to 12”, then the barrier is considered a railing. Any additional railing topping the barrier should be coded under Decorative Rail (Assessment 9335) or Protective Screening (Assessment 9337).

- If the height of a bridge barrier is less than 12”, then the barrier is considered a curb and should be evaluated under the appropriate bridge railing located above the curb.

- The inspector should use engineering judgment to consider what the primary stopping force is when deciding if a barrier is a curb or railing.

When sidewalks are present on the bridge, railings are placed at the outermost edge of sidewalk. It is used to prevent pedestrians and bicycles from straying off the bridge and to give a minimum level of comfort while crossing. Sidewalks and medians are not to be coded under the bridge railings. These shall be coded under the appropriate assessment.

On the inspection report form, all railings are recorded in units of linear feet. When wingwalls are not parallel to the roadway, railing quantities used for Element Level Inspections are measured from end of abutment to end of abutment. On bridges with wingwalls parallel to the roadway, railing quantities are measured from wing tip to wing tip, and these points will often coincide with the point of their beam guardrail attachment. When a combination of railings exist along one edge of the roadway (*e.g.,* metal stacked on concrete), the length is measured along the length of the roadway and should not be doubled for that one side.

The Condition States for rail elements are concerned with material deterioration severity. Damage that is not associated with material defects, such as traffic impact, needs to be recorded on the inspection form under the most appropriate defect. The inspector shall note the defect caused by traffic damage under the bridge railing element notes.

It is the inspector's task to examine the rails and reasonably assign the most severe Condition State to the entire element or portions thereof. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

When a bridge railing is comprised of several different material types, for example a thrie beam attached to the front face of a concrete parapet with a timber curb, and a unit of measure exhibits overlapping defects in all materials, the most severe defect Condition State will control. However, if the Condition States of the overlapping defects are the same, the defect of the bridge railing element shall be recorded above all other railing component defects, regardless if the other defect has a lower defect number. This is to ensure that the condition of the bridge railing element itself takes priority.

2.6.8.1 Metal Rail (Element 330)

Uncoated metal railings are constructed of aluminum or stainless steel, although uncoated steel rails are sometimes but rarely found. Uncoated metal rails rely on the material's inherent corrosion resistance to protect themselves from salt spray and atmospheric moisture. It is unusual to find an uncoated metal railing in Wisconsin.

On metal rails, the vertical members anchored to the deck are referred to as posts, while the horizontal members attached to the posts are referred to as rail beams.

Metal rails may have multiple different materials associated with it. In other words, the metal rail would be the primary rail; however, it may have a concrete curb, steel posts, and timber blocking between the posts and railing associated with it. Due to this fact, it may be necessary to code defects of other material types under Metal Rail. The inspector shall report the material defects on the primary rail material when coding overlapping defects of similar conditions states.

Figure 2.6.8.1-1 Metal Railing with Masonry Posts – Condition State 1

Element Level Inspection

On the inspection report form, bridge railings are recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of uncoated metal rails should include the following items:

- Examining the posts for section loss due to corrosion, spalling, or decay especially near its base. The largest bending moment of a post occurs at its base, which is unfortunately where most section loss will occur due to moisture and deicing chemicals.
- Looking for damage caused by vehicular collisions. Note the condition of the rail beam or post, as well as its remaining ability to withstand a second impact.
- Noting the extent of corrosion.
- Looking for solid anchorage of the post to the deck. Report any loose connections or anchorage losses.
- Examining the rail beam for loose connections to the posts.
- Checking the rail beam's horizontal and vertical alignment. Permanent deflections affect the railing's ability to properly redirect an errant vehicle.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the defect associated with the bridge railing material controls.

Metal railings may use multiple material defects for each rail. When deciding which type of material railing detailed in this chapter, the inspector is to consider the primary material of the railing. Other elements of the railing which may be made of other materials will be coded using each specific material defect, but quantified under the main railing material railing type. An example of this would be a metal railing with timber blocking. This railing can be coded using both material defects from steel and timber. The defects listed below are only to be used for the metal railing portion. The inspector is responsible to determine the necessary defects of the remaining components not constructed of metal.

- Corrosion (1000)
- Cracking (1010)

- Connection (1020)
- Distortion (1900)

Condition States

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.8.2 Reinforced Concrete Bridge Rail (Element 331)

Concrete railings (parapets) are very common on highway bridges. New Jersey-type concrete parapets have a proven track record for strength and serviceability. They are often able to absorb vehicle impacts without failing and are able to redirect, minimize rollovers, and not over-decelerate these vehicles during a crash.

Other concrete parapets have been recently developed for architectural effects. The most common is the Texas rail, which still satisfies current safety criteria while featuring decorative cutouts between the deck and top horizontal rail.

In order to use this element the entire rail must be made of concrete.

Concrete rails may have different materials associated with the curb. Due to this fact, it may be necessary to code defects of other material types under Reinforced Concrete Bridge Rail. The inspector shall report the material defects on the primary rail material when coding overlapping defects of similar conditions states.

Element Level Inspection

On the inspection report form, bridge railings are recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of concrete rails should include the following items:

- Looking for signs of impact damage, such as spalls, delamination and cracking. The location, severity, and size of the damage should be documented. Check the backside of the railing and deck edges for related cracking, delamination, and spalls.

- Checking the parapet for Delaminations, spalls, and exposed reinforcing steel. Large spalls can pose the additional hazard of snagging an errant vehicle.

- Inspecting the member for both vertical and transverse cracks.

- Checking the entire member for signs of corroding reinforcing steel, as indicated by rust stains or exposed reinforcement. Section loss associated with reinforcing steel corrosion can reduce the parapet's strength.

- Looking for leaching, and noting if it is stained with rust since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as Delaminations and spalls.

- Checking previously repaired areas for soundness by hammer tapping.

- Looking for areas of scaling.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the defect associated with the bridge railing material controls. The defects listed below are only to be used for the reinforced concrete railing portion. The inspector is responsible to determine the necessary defects of the remaining components not constructed of reinforced concrete.

- Delaminations/Spalls/Patched Areas (1080)

- Exposed Rebar (1090)

- Cracking (RC) (1130)

- Abrasion/Wear (PSC/RC) (1190)

Condition States

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green

- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.6.8.2-1 Concrete Bridge Railing - Condition State 1

Figure 2.6.8.2-2 Concrete Bridge Rail – Most in Condition State 1 - Two Areas Condition State 3.

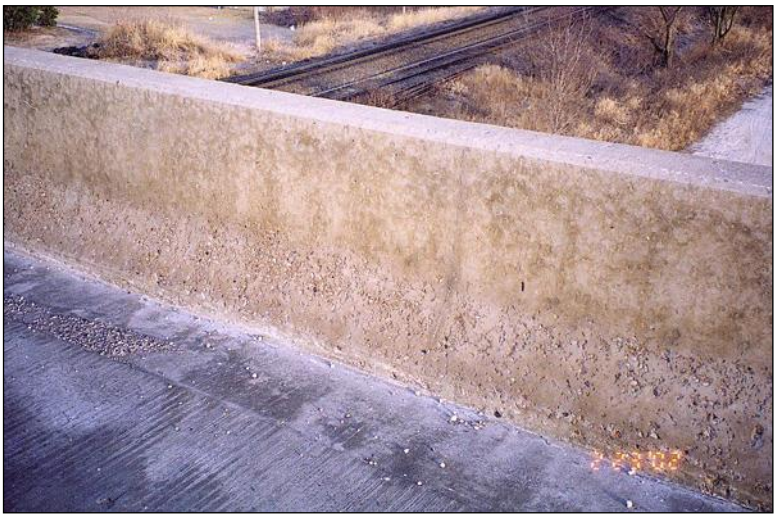

Figure 2.6.8.2-3 Scaling on the Lower Half of a Concrete Bridge Rail - Condition State 2

Figure 2.6.8.2-4 Ornamental Concrete Bridge Rail – Condition State 3

Figure 2.6.8.2-5 Concrete Bridge Railing - Condition State 4

Figure 2.6.8.2-6 Concrete Railing with Decorative Rail Assessment – Condition State 3

2.6.8.3 Timber Bridge Rail (Element 332)

Although timber is an acceptable material to the American Association of State Transportation and Highway Officials (AASHTO) for railing construction, timber rails are not commonly used for new designs unless aesthetics is important. Timber rails are more commonly found on older or rural bridges. Many older timber rails do not meet current safety or strength criteria. Timber rails are normally constructed using vertical posts anchored to the deck and horizontal rail beams connected to the posts.

Timber rails may have multiple different materials associated with it. In other words, the timber rail would be the primary rail; however, it may have a concrete curb, steel posts, and FRP blocking between the posts and railing associated with it. Due to this fact, it may be necessary to code defects of other material types under Timber Bridge Rail. The inspector shall report the material defects on the primary rail material when coding overlapping defects of similar conditions states.

Element Level Inspection

On the inspection report form, bridge railings are recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of timber rails should include the following items:

- Examining the posts and rail beams for splits, checks, and decay.
- Looking for damage caused by vehicular collisions, such as cracked, split or crushed rail beams, cracked posts, and post bases that have ripped away from the deck. Note the condition of the rail beam or post, as well as its remaining ability to withstand additional impact.
- Looking for solid anchorage of the post to the deck. Report any loose connections or anchorage losses.
- Examining the rail beam for loose connections to the posts.
- Checking the rail beam's horizontal and vertical alignment. Permanent deflections affect the railing's ability to properly redirect an errant vehicle.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the defect associated with the bridge railing material controls. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity..

Timber railings may use multiple material defects for each rail. When deciding which type of material railing detailed in this chapter, the inspector is to consider the primary material of the railing. Other elements of the railing which may be made of other materials will be coded using each specific material defect, but quantified under the main railing material railing type. An example of this would be a timber railing with a concrete curb. This railing can be coded using both material defects from timber and concrete. The defects listed below are only to

be used for the timber railing portion. The inspector is responsible to determine the necessary defects of the remaining components not constructed of timber.

- Connection (1020)
- Decay/Section Loss (1140)
- Checks/Shakes/Cracks/Splits/Delaminations (1150)
- Abrasion/Wear (1180)
- Distortion (1900)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

Figure 2.6.8.3-1 Timber Bridge Railing - Condition State 1

Figure 2.6.8.3-2 Timber Bridge Railing – Condition State 2

Figure 2.6.8.3-3 Missing Timber Bridge Rail Beam – Condition State 3

2.6.8.4 Masonry Bridge Railing (Element 334)

This element defines all types and shapes of masonry block or stone bridge railing. All elements of the railing must be masonry block or stone. Faux masonry elements (i.e. precast concrete blocks, reinforced concrete blocks, etc.) will not be rated under this set of elements. These types of material will be rated under “Other Materials”.

In order to use this element, the entire railing must be masonry.

Element Level Inspection

On the inspection report form, bridge railings are recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the defect associated with the bridge railing material controls. The defects listed below are only to be used for the masonry railing material. The inspector is responsible to determine the necessary defects of the remaining components not constructed of masonry.

- Mortar Breakdown (1610)
- Splits/Spall/Patched Area (1620)
- Masonry or Panel Displacement (1640)

Condition State Commentary

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

2.6.8.5 Other Material Bridge Railing (Element 333)

This element describes railings which are composed of other materials besides concrete, metal, timber, or masonry.

Element Level Inspection

On the inspection report form, bridge railings are recorded in units of linear feet. Where multiple condition states exist within a unit of measure only the predominant defect in severity and extent is recorded. The other defects located within the unit of measure shall be captured by the inspector under the element or appropriate defect notes. The sum of all of the reported condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of combination rails should include the following items:

- Looking for signs of impact damage. On reinforced concrete components, evidence includes spalls and localized heavy cracking. The location, severity, and size of the damage should be documented. On metal components, evidence includes dents, bends, tears, and fractures. On timber components, evidence includes cracked, chipped or crushed rail beams, cracked posts, and post bases that have ripped away from the deck. Note the condition of the rail beam or post, as well as its remaining ability to withstand additional impact.

- Checking reinforced concrete components for Delaminations, spalls, and exposed reinforcing steel. Large spalls can pose the additional hazard of snagging an errant vehicle.

- On reinforced concrete components, reinforcing steel corrosion is indicated by rust stains or exposed reinforcement. Section loss associated with reinforcing steel corrosion can reduce the rail strength. On steel components, examine the posts for section loss due to corrosion, especially near their base. The largest bending moment of a post occurs at its base, which is unfortunately where most section loss due to moisture and deicing chemicals will occur. Note the paint/galvanized coating condition and extent of corrosion on coated steel. Check connection areas for dissimilar metal corrosion (aluminum rails with steel hardware).

- Examining the components for long-term deterioration. On timber components, check the posts and rail beams for splits, checks, and decay. On reinforced concrete components, look for both vertical and transverse cracks. Look also for leaching on concrete, and note if it is stained with rust since this condition suggests reinforcing steel corrosion. These defects can grow into larger problems such as Delaminations and spalls.

- Checking previously repaired areas for soundness and functionality.

- Checking horizontal and vertical alignment. Permanent deflections affect the railing's ability to properly redirect an errant vehicle.

- Looking for solid anchorage to the deck. Report any loose connections or anchorage losses.

- Examining rail beams for loose connections to the posts.

Element Defects

Refer to Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.). The order of the defect numbering indicates the controlling defect. Given multiple defects of the same condition state within a unit of measure, the defect associated with the bridge railing material controls. The defects listed below are only to be used for the other material railing portion. The inspector is responsible to determine the necessary defects of the remaining components not constructed of "other" material.

- Corrosion (1000)
- Cracking (1010)
- Connection (1020)
- Delamination/Spall/Patched Area (1080)
- Deterioration (1220)
- Distortion (1900)

Condition States

Appendix A defines the Condition States for each individual defect. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The WisDOT Field Manual tabulates the element defects listed above and bases the Condition States on the progression of severity for each defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

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2.7 ASSESSMENTS

There are several components found on a bridge structure that are not primary structural members but are items that the department wants to track. As they were not structural members, these items did not need to be elements, as this would require the creation and tracking of element specific defects. On the WisDOT bridge inspection form, the inspector will find an area for assessments. Many of the elements from WisDOT's CoRe inspections (the previous version of element level inspection prior to 2014) are now identified as assessments. Assessments are not elements and do not have defects associated with them. Rather, similar to condition states, assessments have four levels of deterioration or Assessment States.

Bridge inspections will typically have several assessments that must be inspected and recorded. It is the inspector's responsibility to determine the assessments that are associated with each bridge and evaluate and record them. Assessments are not optional items. The inspector must capture all assessments during a bridge inspection.

2.7.1 Assessment Items

On the inspection report all assessments are recorded in units of "each". It is the inspector's task to examine each assessment and reasonably assign the most severe assessment condition to each assessment. Similar to elements and defects, the inspector has the ability to describe and elaborate on assessments on the inspection report in the associated box under each assessment. It is recommended the inspector treat assessments with the same attention to detail as he/she would elements/defects; incorporating sketches, photographs and detailed notes. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Each assessment will be coded on an assessment state scale: Good, Fair, Poor, and Severe. Each assessment is described below, along with a description of the coding criteria.

2.7.1.1 Drainage – Approach (Assessment 9001)

This assessment defines drainage systems for the approach sections of the bridge. It is present on all bridges regardless if drainage system is present with the exception of unit bridges. Unit bridges (larger bridges that are comprised of several adjacent bridges, each with a separate bridge number) will not have this assessment on any interior spans but will on the bridges with the end spans. This assessment is used to monitor the condition of any erosion off the ends of the structure or undermining of approach.

On the inspection report form approach drainage is recorded in units of "each". All quadrants of a bridge span will count as an individual "each". Therefore, smaller bridges will have a quantity of four. Split bridges with a grass median may have a quantity of eight. The inspector will capture this assessment even if the approach is on the high side of a super-elevated bridge, or a bridge with curb and gutter along the approaches.

The assessment Drainage - Approach is limited to 20 feet from the wingtip at each quadrant of a bridge. This may include storm sewer inlets. Degrading storm sewer systems in the vicinity of the structure may have an adverse impact on the structure capacity of the abutment or approach slabs.

It is the inspector's task to examine each approach drainage assessment and reasonably assign the most severe assessment state to the "each" quantity. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-1: Assessment 9001 – Approach Drainage. Note concrete flume.

Assessment Condition

Good: Drainage systems are functioning properly. No slope erosion is evident off the ends of the bridge or in the associated ditches.

Fair: Minor erosion of slopes around the bridge. Drainage systems are plugged or have minor deterioration.

Poor: Moderate erosion of slopes around the bridge. Drainage systems are plugged or have moderate deterioration.

Severe: Major erosion of bridge slopes not related to slope protection. Drainage systems are plugged and have major deterioration.

2.7.1.2 Drainage – Deck (Assessment 9004)

This assessment defines drainage systems on the deck of the bridge. This assessment includes any scuppers and inlets that are located on the deck. The inlets located within the approach roadway of the bridge are not included in this assessment.

On the inspection report form, deck drainage is recorded in units of "each". Each scupper, inlet, or drain pipe counts as one "each" regardless if combined or not.

It is the inspector's task to examine all deck drainage and reasonably assign the most severe assessment condition to each drain found within the limits of the bridge deck. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-2: Assessment 9004 – Deck Drainage.

Assessment Condition

Good: Drainage systems are functioning properly. No slope erosion is evident off the ends of the bridge or in the associated ditches.

Fair: Drainage systems are plugged or have minor deterioration.

Poor: Drainage systems are plugged or have moderate deterioration.

Severe: Drainage systems are plugged and have major deterioration.

2.7.1.3 Aesthetic Treatment (Assessment 9010)

This assessment evaluates the aesthetic coatings or treatments on a bridge structure. Examples of aesthetic treatments include concrete staining, form liners, obelisks, as well as many others. The inspector is to note the type of aesthetic treatment on the inspection report.

On the inspection report form aesthetic treatments are recorded in units of “each” for the entire bridge. That is, regardless of the number of aesthetic treatments found on a bridge, the quantity reported shall be 1 Each. The inspector must note the type(s) of aesthetic treatments used on the structure under the assessment notes for Aesthetic Treatment.

It is the inspector’s task to examine each aesthetic treatment assessment and reasonably assign the most severe assessment condition to the whole assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

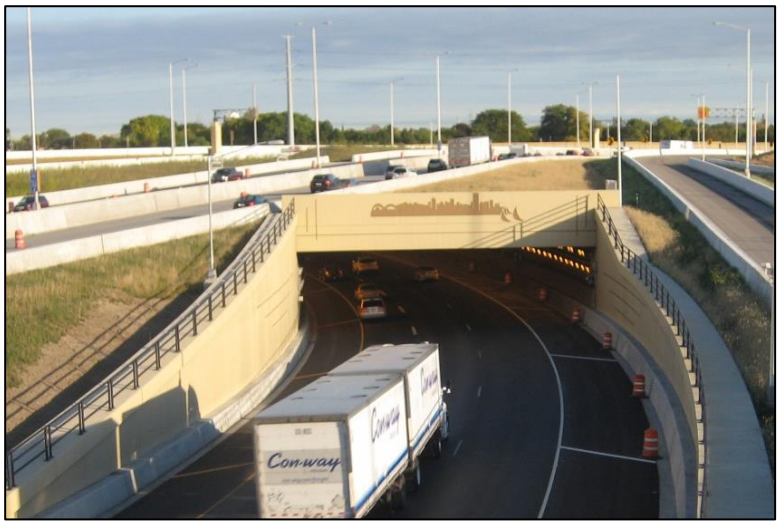

Figure 2.7.1-3: Assessment 9010 – Aesthetic Treatment.

Assessment Condition

Good: System is in good condition, with no notable issues.

Fair: Aesthetic system is in fair condition, with some fading or discoloration. Minor issues.

Poor: Aesthetic system is in poor condition, with fading or discoloration.

Severe: Aesthetic system is in severe condition and is not functioning as intended.

2.7.1.4 Utilities (Assessment 9011)

This assessment evaluates utilities that are attached to bridge structures. This can include lighting, sewer lines, water mains, gas lines, electrical lines, as well as multiple others.

On the inspection report form, utilities are recorded in units of “each” per each conduit run. Therefore, if four conduits are hung in a bay between two beams, the quantity would be four. Similarly, three conduits hung from the exterior of a concrete parapet would have a quantity of three. It will be at the inspector’s discretion if several conduits are hung by one attachment point as whether the quantity will be one, or the number of conduits in that package. Clarification should be made in the notes.

It is the inspector’s task to examine each utility and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-4: Assessment 9011 – Utilities.

Assessment Condition

Good: Utility is in excellent condition, no problems noted.

Fair: Utility is in fair condition. Some minor problems are noted, but they do not affect the serviceability of the utility

Poor: Utility is in poor condition and local failures are possible. Utility hangers may be disconnected from the bridge structure at various locations.

Severe: Utility is in severe condition. Failures have occurred.

2.7.1.5 Movable Bridge – Counterweight (Assessment 9020)

This assessment evaluates the counterweight system used for movable bridges.

On the inspection report form, counterweights are recorded in units of “each” with each counterweight assessed separately.

It is the inspector’s task to examine each counterweight and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-5: Assessment 9020 – Movable Bridge Counterweight.

Assessment Condition

Good: Counterweight is in good condition, no problems noted.

Fair: Counterweight is in fair condition. Some minor problems are noted, but they do not affect the serviceability of the bridge.

Poor: Counterweight is in poor condition and local failures are possible. Serviceability of the bridge could be affected by the item's continued deterioration.

Severe: Counterweight is in severe condition and is not functioning properly.

2.7.1.6 Movable Bridge Cables (Assessment 9021)

This assessment evaluates the cables on a movable bridge. These cables should not be captured under the National Bridge Elements 147 or 148, as these cables are used for suspension, cable-stayed, and tied arch bridges.

On the inspection report form, bridge cables are recorded in units of "each" where each grouping of cables is considered one assessment. The inspector should clarify quantities in the assessment notes. The inspector should include the condition of the cable attachments within the cable assessment. Sheave components which support the cables, will be evaluated in the Moveable Bridge Mechanical Inspection Report.

It is the inspector's task to examine each cable or cable group and reasonably assign the most severe assessment condition to the each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-6: Assessment 9021 – Movable Bridge Cables.

Assessment Condition

Good: Cables are in good condition and are properly functioning

Fair: Cables are in fair condition and are properly functioning.

Poor: Cables are in poor condition.

Severe: Cables are in severe condition, resulting in the lifting mechanism not working properly.

2.7.1.7 Signs – Object Markers (Assessment 9030)

This assessment evaluates the condition of bridge object markers at the location of the bridge structure. This assessment will also include the culvert delineators found at culverts.

On the inspection report form, signs are recorded in units of “each” where each individual sign is considered its own assessment. Typically, bridges will have an object marker (tiger board) at each corner of the bridge. The quantity in this case would be four. In some instances, typically on shorter span structures, one sign post will be located on one side of a bridge approach and another post on the opposite side on the other approach. These posts may have an object marker on each side of the post. The quantity in this case would be four. Similarly, each culvert delineator will be an individual assessment.

It is the inspector’s task to examine each sign and reasonably assign the most severe assessment condition. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-7: Assessment 9030 – Signs – Object Markers.

Assessment Condition

Good: Sign is present and is in good condition (there may be superficial damage or deterioration).

Fair: Sign is present - sign may have some damage or deterioration (slightly bent or fading), but remains readable.

Poor: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

Severe: Sign is absent, or incorrect, or existing sign is damaged or deteriorated to the extent that repair or replacement is required as soon as possible.

2.7.1.8 Signs – Narrow Bridge (Assessment 9031)

This assessment evaluates narrow bridge signs both at the bridge/culvert, as well as the advanced warning signs for the bridge/culvert.

On the inspection report form, signs are recorded in units of “each” where each individual sign is considered its own assessment. Typically each direction will have one advanced warning sign leading up to the bridge, as well as at the bridge. Each one of these signs will be an assessment.

It is the inspector’s task to examine each sign and reasonably assign the most severe assessment condition. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Sign is present and is in good condition (there may be superficial damage or deterioration).

Fair: Sign is present - sign may have some damage or deterioration (slightly bent or fading), but remains readable.

Poor: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

Severe: Sign is absent, or incorrect, or existing sign is damaged or deteriorated to the extent that repair or replacement is required as soon as possible.

2.7.1.9 Signs – One Lane Bridge (Assessment 9032)

This assessment defines the condition of the one lane bridge signs both at the bridge/culvert, as well as the advanced warning signs for the bridge/culvert.

On the inspection report form, signs are recorded in units of “each” where each individual sign is considered its own assessment. Typically each direction will have one advanced warning sign leading up to the bridge, as well as a sign at the bridge. Each one of these signs will be an assessment.

It is the inspector’s task to examine each sign and reasonably assign the most severe assessment condition. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Sign is present and is in good condition (there may be superficial damage or deterioration).

Fair: Sign is present - sign may have some damage or deterioration (slightly bent or fading), but remains readable.

Poor: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

Severe: Sign is absent, or incorrect, or existing sign is damaged or deteriorated to the extent that repair or replacement is required as soon as possible.

2.7.1.10 Signs – Vertical Clearance (Assessment 9033)

This assessment evaluates the condition of the vertical clearance signs at the bridge. Typically, these signs will be attached to the bridge deck/girder to inform the travelling public under the bridge as to the clearance restrictions of the bridge under-clearance.

On the inspection report form, signs are recorded in units of “each” where each individual sign is considered its own assessment. Most bridges with restricted clearances will have a sign on

both sides of the bridge to inform both directions of traffic. Each one of these signs will be an assessment.

It is the inspector's task to examine each sign and reasonably assign the most severe assessment condition. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-8: Assessment 9033 – Signs – Vertical Clearance.

Assessment Condition

Good: Sign is present and is in good condition (there may be superficial damage or deterioration).

Fair: Sign is present - sign may have some damage or deterioration (slightly bent or fading), but remains readable.

Poor: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

Severe: Sign is absent, or incorrect, or existing sign is damaged or deteriorated to the extent that repair or replacement is required as soon as possible.

2.7.1.11 Signs – Weight Limit Posting (Assessment 9034)

This assessment evaluates the weight limit posting signs both at the bridge/culvert, as well as the advanced warning signs further away from the bridge/culvert.

On the inspection report form, signs are recorded in units of "each" where each individual sign is considered its own assessment. Typically each direction will have one advanced warning

sign leading up to the bridge, as well as at the bridge. Each one of these signs will be an assessment.

It is the inspector's task to examine each sign and reasonably assign the most severe assessment condition. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-9: Assessment 9034 – Signs – Weight Limit Posting.

Assessment Condition

Good: Sign is present and is in good condition (there may be superficial damage or deterioration).

Fair: Sign is present - sign may have some damage or deterioration (slightly bent or fading), but remains readable.

Poor: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

Severe: Sign is absent, or incorrect, or existing sign is damaged or deteriorated to the extent that repair or replacement is required as soon as possible.

2.7.1.12 Signs – Others (Assessment 9035)

This assessment evaluates any additional signs both at the bridge/culvert, as well as the advanced warning signs further away from the bridge/culvert that are not captured under Assessments 9030, 9031, 9032, 9033, and 9034.

On the inspection report form, signs are recorded in units of "each" where each individual sign is considered its own assessment.

It is the inspector's task to examine each sign and reasonably assign the most severe assessment condition. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Sign is present and is in good condition (there may be superficial damage or deterioration).

Fair: Sign is present - sign may have some damage or deterioration (slightly bent or fading), but remains readable.

Poor: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

Severe: Sign is absent, or incorrect, or existing sign is damaged or deteriorated to the extent that repair or replacement is required as soon as possible.

2.7.1.13 Slope Protection – Asphalt (Assessment 9040)

This assessment evaluates slopes protected by asphaltic systems, similar to asphalt roadway paving. This should not be confused with crushed aggregate sprayed with bituminous. This is a separate assessment and will be covered in Section 2.7.1.18.

On the inspection report form, slope protection systems are recorded in units of "each" where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number).

It is the inspector's task to examine each slope protection system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Slope protection is sound and protecting the embankments adjacent to the abutments.

Fair: Minor deterioration. Minor settlement, cracking or spalling may exist.

Poor: Moderate deterioration. Moderate settlement, cracking or spalling may exist.

Severe: Major deterioration of slope paving. Serious settlement, undermining, cracking, buckling or spalling may exist.

2.7.1.14 Bridge Closure System (Assessment 9036)

This assessment evaluates the signage, barricades, fencing, etc. that is associated with the closure or partial closure of a bridge. This assessment does not include the signage pertaining

to vertical clearance, advanced warning signs (unless explicitly noting a detour or lane closure), object markers, etc.

On the inspection report form, bridge closure systems are recorded in units of “each” where each side of the bridge is considered its own assessment. Each unit will include the condition evaluation of all components.

It is the inspector’s task to examine each bridge closure system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-10: Assessment 9036 – Bridge Closure System. Note assessment used for sidewalk closure.

Assessment Condition

Good: All barricades, signs, fencing, etc. for the closure system are secured and installed properly to permit safe passage through or around the bridge.

Fair: Detour or passage at the bridge is safe. Barricades, signs, fencing, etc. are installed; however, slightly misaligned/spaced and/or reflective materials are fading or dull.

Poor: Bridge closure system is improperly installed and/or inadequate for safe passage through or around bridge.

Severe: Bridge closure system is missing or damaged. Traversing bridge without system in place poses eminent endanger travelling public.

2.7.1.15 Slope Protection – Bare (Assessment 9041)

This assessment evaluates slopes that have no protection systems. Care should be taken not to confuse this slope protection with overgrown slope protection of a different material, or a material that has washed away.

On the inspection report form, slope protection systems are recorded in units of "each" where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number).

It is the inspector's task to examine each slope protection system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Bare Slope is sound with no erosion present.

Fair: Minor erosion present.

Poor: Moderate erosion present.

Severe: Severe erosion is present.

2.7.1.16 Slope Protection – Concrete (Assessment 9042)

This assessment evaluates slopes protected by concrete slope paving. Riprap infilled with concrete or grout will be captured under Assessment 9045 – Slope Protection - Riprap.

On the inspection report form, slope protection systems are recorded in units of "each" where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number).

It is the inspector's task to examine each slope protection system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Concrete slope protection is sound and protecting the embankments adjacent to the abutments.

Fair: Minor deterioration. Minor settlement, cracking or spalling may exist.

Poor: Moderate deterioration. Moderate settlement, cracking or spalling may exist.

Severe: Major deterioration of slope paving. Serious settlement, undermining, cracking, buckling or spalling may exist.

2.7.1.17 Slope Protection – Crushed Aggregate Sprayed with Bituminous(Assessment 9043)

This assessment evaluates slopes protected by crushed aggregate sprayed w/ bituminous asphalt coating. This should not be confused with asphaltic slope paving. This is a separate assessment, refer to Section 2.7.1.15.

On the inspection report form, slope protection systems are recorded in units of “each” where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number).

It is the inspector’s task to examine each slope protection system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

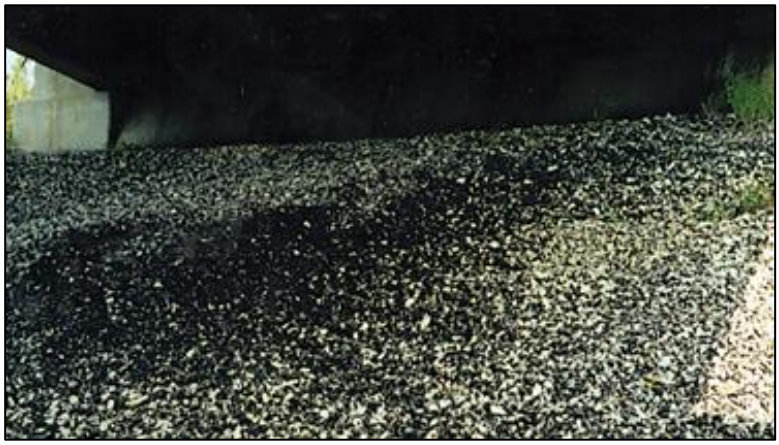

Figure 2.7.1-11: Assessment 9043 – Slope Protection – Crushed Aggregate Sprayed with Bituminous.

Assessment Condition

Good: Crushed aggregate is sound and protecting the embankment adjacent to the abutments.

Fair: Minor deterioration. Slope may have settled. Loose surface aggregate due to bleaching. Portions may be missing.

Poor: Moderate deterioration. Slope has settled. Loose surface aggregate due to bleaching. Portions may be missing.

Severe: Major deterioration. Major settlement and/or buckling are evident.

2.7.1.18 Slope Protection – Epoxy Coated Crushed Aggregate (Assessment 9044)

This assessment evaluates slopes that are protected by epoxy coated aggregate sloped paving.

On the inspection report form, slope protection systems are recorded in units of “each” where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number).

It is the inspector’s task to examine each slope protection system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Crushed stone is sound and protecting the embankment adjacent to the abutments.

Fair: Minor deterioration. Slope may have settled. Loose surface stone due to bleaching. Portions may be missing.

Poor: Moderate deterioration. Slope has settled. Loose surface stone due to bleaching. Portions may be missing.

Severe: Major deterioration. Major settlement is evident.

2.7.1.19 Slope Protection – Riprap (Assessment 9045)

This assessment evaluates slopes protected by riprap. Riprap with concrete infill grouting would also be captured under this assessment.

On the inspection report form, slope protection systems are recorded in units of “each” where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number).

It is the inspector’s task to examine each slope protection system and reasonably assign the most severe assessment condition to the whole assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-12: Assessment 9045 – Slope Protection – Riprap.

Assessment Condition

Good: Riprap is adequate, sound and protecting the embankments adjacent to the abutments.

Fair: Minor deterioration. Slope may have settled and portions may be missing. Riprap provides some embankment protection.

Poor: Moderate deterioration. Slope may have settled and portions may be missing. Riprap provides little embankment protection.

Severe: Major deterioration. Slope has settled and portions are missing. Riprap provides no embankment protection.

2.7.1.20 Slope Protection – Select Crushed Material (Assessment 9046)

This assessment evaluates slopes protected by select crushed material. This assessment will only capture slope protections without a stabilizer sprayed on top. If the crushed material/rock has a spray applied to it to stabilize the slope, then the inspector should use the appropriate assessment.

On the inspection report form, slope protection systems are recorded in units of “each” where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of a segments, each with a separate bridge number).

It is the inspector’s task to examine each slope protection system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Crushed aggregate is sound and protecting the embankment adjacent to the abutments.

Fair: Minor deterioration. Slope may have settled. Portions may be missing.

Poor: Moderate deterioration. Slope has settled. Portions may be missing.

Severe: Major deterioration. Major settlement is evident.

2.7.1.21 Slope Protection – Other (Assessment 9047)

This assessment evaluates other slopes not included in the specific slope protection assessment definitions.

On the inspection report form, slope protection systems are recorded in units of "each" where each side of the bridge is considered its own assessment. Most bridges will have two assessments for slope protection, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number).

It is the inspector's task to examine each slope protection system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Slope is sound with no erosion present.

Fair: Minor deterioration, but functioning effectively.

Poor: Moderate deterioration and/or erosion.

Severe: Slope protection system has failed.

2.7.1.22 Steel Diaphragm (Assessment 9167)

This assessment evaluates steel diaphragm members between girders. The steel protective coating is excluded from the assessment of this component and included in the primary painted superstructure element. The steel protective coating should not be included if the steel diaphragms are attached to concrete beams or are of a different protective coating than that of the superstructure. This assessment is not to be used on curved bridges as the diaphragms are primary load carrying members on those bridges. Steel diaphragms on curved bridges shall be coded under Other Primary Structural Members (Element 8170).

On the inspection report form, diaphragms are recorded in units of "each" for every individual diaphragm including connection points.

It is the inspector's task to examine each diaphragm and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-13: Assessment 9167 – Steel Diaphragm.

Assessment Condition

Good: Good condition. Connections are in place and functioning. No distortion.

Fair: Freckled Rust. Corrosion of the steel has initiated. Loose fasteners or pack rust without distortion is present but the connection is functioning.

Poor: Section loss is evident or pack rust is present. Missing bolts, rivets, broken welds, or fasteners.

Severe: The system no longer functions as intended.

2.7.1.23 Concrete Diaphragm (Assessment 9168)

This assessment evaluates concrete diaphragm members between girders that are above piers and at intermediate locations. To be considered a diaphragm, the concrete component must be greater than half the height of beam. Concrete that measures less than half the height of the beam will be considered extensions of the deck and evaluated as part of the deck element. In addition, diaphragms at abutments will be evaluated under this assessment if they do not retain fill (i.e. there is a backwall present behind the diaphragms). If the diaphragm retains fill, then the diaphragm will be considered part of the abutment and evaluated as part of the abutment element.

On the inspection report form, diaphragms are recorded in units of “each” for every applicable individual diaphragm throughout the superstructure.

It is the inspector’s task to examine each diaphragm and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-14: Assessment 9168 – Concrete Diaphragm.

Assessment Condition

Good: No deterioration. Possible discoloration, efflorescence, or superficial cracking but doesn't affect strength or serviceability.

Fair: Minor cracks and spalls may be present but there is no exposed reinforcing or surface evidence of rebar corrosion.

Poor: Some delaminations &/or spalls may be present and some reinforcing may be exposed. Possible rebar corrosion but section loss is incidental and doesn’t significantly affect strength or serviceability.

Severe: Advanced deterioration. Corrosion of reinforcement &/or loss of concrete section is sufficient to warrant analysis to ascertain the impact on the strength &/or serviceability of either the element or the bridge.

2.7.1.24 Lateral Bracing (Assessment 9169)

This assessment evaluates lateral bracing systems below the decks of trusses and through arches, the horizontal bracing inside trapezoidal box girders, and horizontal/wind bracing between I-girders.

Bracing above the roadway in both trusses and arches are evaluated under assessment Truss or Tied Arch Portal Bracing (9170). The steel protective coating is excluded from this assessment and should be evaluated and quantified under the primary painted superstructure element.

For deck arch structures, the lateral bracing between the arch ribs is evaluated under the respective arch element.

On the inspection report form, lateral bracing systems are recorded in units of “each” per span of the bridge regardless of the number of trusses, bays, or girder lines. It is the inspector’s task to examine the lateral bracing system in each span and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-15: Assessment 9169 – Lateral Bracing.

Assessment Condition

Good: Good condition. Connections are in place and functioning. No distortion.

Fair: Freckled Rust. Corrosion of the steel has initiated. Loose fasteners or pack rust without distortion is present but the connection is functioning.

Poor: Section loss is evident or pack rust is present. Missing bolts, rivets, broken welds, or fasteners.

Severe: The system no longer functions as intended.

2.7.1.25 Truss or Tied Arch Portal Bracing (Assessment 9170)

This assessment evaluates all bracing above the roadway and includes all secondary gusset plates (i.e. those plates whose plane lies perpendicular to the centerline of the roadway or not in the plane of the truss/arch). Vertical and lateral bracing, regardless of plane, is included within this assessment. The steel protective coating is excluded from this assessment and should be evaluated and quantified under the primary painted superstructure element.

On the inspection report form, Truss or Tied Arch Portal Bracing is recorded in units of "each" for each span, i.e. from substructure unit to substructure unit.

It is the inspector's task to examine each upper truss or arch bracing system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Good condition. Connections are in place and functioning. No distortion.

Fair: Freckled Rust. Corrosion of the steel has initiated. Loose fasteners may be present but the connection is functioning. No impact damage noted.

Poor: Section loss is evident or pack rust is present. Missing bolts, rivets, broken welds, or fasteners. Impact damage may be present, but has not distorted primary truss members.

Severe: The system no longer functions as intended. Impact damage has distorted primary truss members.

2.7.1.26 Timber Diaphragms (Assessment 9171)

This assessment evaluates timber diaphragm members between girders that are above substructure and at intermediate locations. Note that this assessment will typically only be used on timber superstructures.

Timber diaphragms may be either in an X formation or solid dimensional lumber cut to fit between the girder. Diaphragms comprised of glue-laminated timber shall fall under this assessment. The inspector shall pay close attention to loose, corroded or missing fasteners. X blocking, typically observed in simple span timber structures, will commonly have the fasteners pulled through the toe of one of the legs limiting the effectiveness of the diaphragm.

On the inspection report form, diaphragms are recorded in units of "each" for every applicable individual diaphragm throughout the superstructure.

It is the inspector's task to examine each diaphragm and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Assessment Condition

Good: Good condition. Connections are in place and functioning. No checking or splitting of the timber components.

Fair: Minor checking or splitting present in timber components. Loose or corroded fasteners is present but the connection is functioning. Minor decay may be present however not yet effecting function.

Poor: Missing fasteners. Timber components in place, however exhibiting full depth checking or splitting or moderate section loss due to decay.

Severe: The diaphragm no longer functions as intended.

2.7.1.27 Culvert End Treatment (Assessment 9248)

This assessment applies to culverts that have end treatments that include headwalls and aprons. Depending on the configuration of the culvert, only certain items of this assessment may be found (i.e., headwall without an apron). As long as a culvert structure has some component of the culvert end treatment, this assessment shall be included on the inspection report. Steel pipe culverts protruding from an earth embankment without any components (headwall, wingwall, or apron) would not have an end treatment associated with it.

On the inspection report form, culvert end treatments are recorded in units of "each" where each end counts as its own assessment. On a typical culvert, the quantity will be two (one for each end).

Wingwalls at culvert ends are coded and their condition assessed under element 8400, regardless of whether a joint exists between the culvert and the wingwall or the culvert and wing are monolithic.

It is the inspector's task to examine each culvert end treatment and reasonably assign the most severe assessment condition to the each assessment. Quantifying these amounts helps to generate quantity/cost estimates for future remedial work.

Figure 2.7.1-16: Assessment 9248 – Culvert End Treatment.

Assessment Condition

Good: Culvert end treatment has little or no deterioration. Timber may have minor splitting. Steel may have minor surface corrosion. Masonry may have minor weathering (mortar joints are sound). Concrete may have minor cracking or scale.

Fair: Culvert end treatment has minor to moderate deterioration. Timber may have moderate splitting (minor decay or fire damage). Steel may have moderate surface corrosion (minor section loss). Masonry may have moderate weathering (mortar joints may have minor deterioration). Concrete may have moderate cracking or scaling (there may be minor delamination or spalling). End treatment may have slight undermining, settlement, misalignment, or separation.

Poor: Culvert end treatment has extensive deterioration. Timber may have extensive splitting - there may be significant decay or fire damage (slight sagging or crushing). Steel may have extensive corrosion (measurable section loss). Masonry may have extensive weathering (mortar joints may have significant deterioration). Concrete may have extensive cracking or scaling (delamination or spalling may be prevalent). End treatment may have significant undermining, settlement, misalignment, or separation.

Severe: Culvert end treatment has severe deterioration, the function or structural capacity of the culvert has been severely impacted - immediate repairs or structural analysis may be required. Timber may have severe splitting or advanced decay (severe sagging or crushing). Steel may have advanced corrosion (severe section loss). Masonry may have severe weathering (mortar joints may have failed). Concrete may have severe cracking, scaling, delamination, or spalling. End treatment may have severe undermining, settlement, misalignment, or separation.

2.7.1.28 Crash Walls/Web Walls/Cross Bracing or Struts (Assessment 9250)

This assessment evaluates all types of cross bracing systems not defined by other elements. This assessment can only be used on substructures. Examples of this assessment include cross bracing between columns, crash walls, and web walls.

On the inspection report form, crash walls/web walls/cross bracing or struts are recorded in units of "each" for each substructure unit. If a single substructure unit has several braces on it, the quantity will still be one each for the entire substructure unit. In addition, if bracing joins two adjacent substructure units, then this brace will be quantified under this assessment, one for each side of the substructure units.

It is the inspector's task to examine each system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-17: Assessment 9250 – Crash Walls/Web Walls/Cross Bracing or Struts.

Assessment Condition

Good: No deterioration.

Fair: Minor deterioration is present but does not affect serviceability of the element.

Poor: Moderate deterioration is present but not of sufficient magnitude to affect serviceability of the bridge.

Severe: The system no longer functions as intended.

2.7.1.29 Dolphin or Fender System (Assessment 9290)

This assessment evaluates systems used to protect bridge substructure units from vessel collisions. These systems can be either free standing or attached to the substructure.

On the inspection report form, dolphin or fender systems are recorded in units of “each” for each system. If there are several free standing dolphins in the vicinity of the bridge structure, each one of these dolphins will be captured as an individual assessment. Similarly, each substructure unit found in the water with a fender system associated with it will be captured as an individual assessment.

It is the inspector’s task to examine each system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-18: Assessment 9290 – Dolphin or Fender System.

Assessment Condition

Good: The dolphin or fender system has little to no deterioration. Minor wear and deterioration may be present but the system is functioning as intended.

Fair: The dolphin or fender system shows signs of deterioration or minor collision damage but the protection of the bridge has not been compromised.

Poor: The dolphin or fender system has advanced deterioration or significant collision damage that compromises its effectiveness in protecting the bridge.

Severe: The dolphin or fender system has failed and provides little to no protection for the bridge.

2.7.1.30 Approach Roadway – Concrete (non-structural) (Assessment 9322)

This assessment evaluates approach roadway sections that are concrete but are not designed as a structural approach roadway. Structural approaches are elements and shall be coded under Elements 320 and 321.

On the inspection report form, non-structural approach roadways are recorded in units of "each" where each approach counts as its own assessment. Most bridges will have two assessments for approach roadway, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number). In addition, bridges with a grass median may have four approach roadway assessments. If a known approach has been recently paved over, the inspector should keep the existing assessment and make note of the change in the assessment note. If the approach has been replaced with a different type of approach, the inspection report should reflect the change by the removal of the old assessment and addition of the new approach type.

It is the inspector's task to examine each system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-19: Assessment 9322 – Concrete Approach Roadway.

Assessment Condition

Good: No deterioration or horizontal or vertical movement other than superficial surface cracks.

Fair: Minor cracks & spalls. There may be some settlement or heaving which increases traffic impact on bridge. <1/2" settlement.

Poor: Cracks may extend through slab, but doesn't act as if is it is broken. Major spalls, but they do not affect the structural integrity of the slab. <1" settlement.

Severe: Slab is broken or rocks under traffic loads. Settlement >1" and cannot be corrected without increasing the size of the slab. Deterioration is excessive which no longer allows for mudjacking.

2.7.1.31 Approach Roadway – Asphalt (Assessment 9323)

This assessment evaluates approach roadway sections that are composed of asphalt.

On the inspection report form, approach roadways are recorded in units of "each" where each approach counts as its own assessment. On the inspection report form, approach roadways are recorded in units of "each" where each approach counts as its own assessment. Most bridges will have two assessments for approach roadway, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number). In addition, bridges with a grass median may have four approach roadway assessments. If a known approach has been recently paved over, the inspector should keep the existing assessment and make note of the change in the assessment note. If the approach has been replaced with a different type of approach, the inspection report should reflect the change by the removal of the old assessment and addition of the new approach type.

It is the inspector's task to examine each system and reasonably assign the most severe assessment condition to each assessment. Quantifying these amounts helps to generate quantity/cost estimates for future remedial work.

Figure 2.7.1-20: Assessment 9323 – Asphalt Approach Roadway.

Assessment Condition

Good: No deterioration or settlement other than superficial cracks.

Fair: Minor cracks. May be minor settlement which increases traffic impact on bridge. <1/2" settlement.

Poor: Alligator cracks and possible rutting is evident. Settlement may be occurring which increases traffic impact on bridge. <1" settlement.

Severe: Major rutting and cracks are evident. >1" settlement which increases traffic impact on the bridge.

2.7.1.32 Approach Roadway – Gravel (Assessment 9324)

This assessment evaluates approach roadway sections that are composed of gravel.

On the inspection report form, approach roadways are recorded in units of "each" where each approach counts as its own assessment. On the inspection report form, approach roadways are recorded in units of "each" where each approach counts as its own assessment. Most bridges will have two assessments for approach roadway, with the exception of the spans of unit bridges (larger bridges that are comprised of adjacent bridge segments, each with a separate bridge number). In addition, bridges with a grass median may have four approach roadway assessments. If a known approach has been recently paved over, the inspector should keep the existing assessment and make note of the change in the assessment note. If the approach has been replaced with a different type of approach, the inspection report should reflect the change by the removal of the old assessment and addition of the new approach type.

It is the inspector's task to examine each system and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-21: Assessment 9324 – Gravel Approach Roadway.

Assessment Condition

Good: No potholes or depressions near edge of bridge deck. Minimal gravel may have been pushed onto deck. Approaches provide smooth transition to bridge.

Fair: Minor potholes or depressions near edge of bridge deck or minor amounts of gravel pushed up on deck. Minor problems that are sufficient to create a noticeable traffic bump.

Poor: Moderate potholes or depressions less than 1 inch deep near edge of bridge deck or moderate amounts of gravel pushed up on deck. Minor problems that are sufficient to create a noticeable traffic bump.

Severe: Significant potholes or depressions greater than 1 inch deep near edge of bridge deck or significant amounts of gravel pushed onto deck. Problems that are substantial enough to launch vehicular traffic so that vehicles bounce on the bridge creating possible impact damage.

2.7.1.33 Roadway Over Structure (Assessment 9325)

This assessment evaluates the roadway over a buried bridge/culvert where there is more than 9" of fill at any point above the structure.

On the inspection report form, roadways over structures are recorded in units of “each” where a bridge/culvert will have one roadway over structure assessment. For roadways on fill over structures, when medians or sidewalks are observed on the roadway, these components shall be evaluated under the Roadway Over Structure assessment.

It is the inspector’s task to examine each roadway and reasonably assign the most severe assessment condition to each assessment. Quantifying these amounts helps to generate quantity/cost estimates for future remedial work.

Figure 2.7.1-22: Assessment 9325 – Roadway over Structure.

Assessment Condition

Good: The roadway over structure is smooth and shows no sign of settlement.

Fair: The roadway over structure has minor settlement and roadway may be cracked and deteriorated.

Poor: The roadway over structure has moderate settlement and roadway may be cracked and deteriorated.

Severe: Roadway over structure has significant deterioration and settlement.

2.7.1.34 Decorative Rail (Assessment 9335)

This assessment includes railing on top of concrete parapet, chain link fence stub between a sidewalk and a travelled way, and other non-screening rail on the exterior of the bridge fascia. These rails can be on the outside fascia of the bridge, or between the roadway and a sidewalk. Also included will be the aluminum/steel tube railing attached to a concrete railing to bring the height of the railing up to standard.

On the inspection report form decorative rail is recorded in units of “each” for one continuous span along a bridge. Therefore, each side of the bridge would be considered two rails if present. Additional railings may be added if barriers are present between sidewalks and traffic.

It is the inspector’s task to examine each decorative rail assessment and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-23: Assessment 9335 – Decorative Rail.

Assessment Condition

Good: Rail has little or no deterioration. Galvanizing or protective coating is sound.

Fair: Rail has minor deterioration. Coating may have minor failures and surface corrosion may be present.

Poor: Rail has moderate deterioration. Coatings may have moderate failure and surface corrosion may be prevalent. Components may be bent or misaligned and connections may be loose.

Severe: Rail has extensive deterioration. Coating may have extensive failure and there may be section loss in exposed areas. Components may be bent or misaligned and connections may be loose.

2.7.1.35 Luminaire Bases (Assessment 9336)

This assessment evaluates the concrete base unit of luminaire supports typically located on the bridge railing along the side of the bridge. This does not take into account the condition of the actual light pole associated with the concrete base. This assessment does not include the base of sign structures, which will be evaluated as the foundation during sign structure inspections.

On the inspection report form, luminaire bases are recorded in units of “each” for every bump out.

It is the inspector’s task to examine each luminaire base and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment’s state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-24: Assessment 9336 – Luminaire Bases.

Assessment Condition

Good: Good condition, with no problems noted.

Fair: Fair condition, with superficial spalls, small areas of delamination, and/or hairline cracking. No steel reinforcement is exposed.

Poor: Moderate deterioration, with cracking and spalls. Steel reinforcement may be exposed with surface corrosion present.

Severe: Base has failed. Major deterioration noted. Major spalls exist and steel reinforcement is exposed with section loss.

2.7.1.36 Protective Screening (Assessment 9337)

This assessment evaluates protective screening or fencing on the exterior of a bridge structure that prevents debris from falling on traffic below. An example of protective screening is chain link fencing. Also included would be railing with protective screening (mesh fence) embedded into it.

On the inspection report form protective screening is recorded in units of "each" for one continuous span along a bridge. Therefore, for fencing on each side of a bridge, the quantity would be two protective screens.

It is the inspector's task to examine each protective screening assessment and reasonably assign the most severe assessment condition to each assessment. This will quantify the assessment's state of deterioration and help generate quantity/cost estimates for future remedial work.

Figure 2.7.1-25: Assessment 9337 – Protective Screening

Assessment Condition

Good: Chain link fence or screening has little or no deterioration. Galvanizing or vinyl coating is sound.

Fair: Chain link fence or screening has minor deterioration. Coating may have minor failure - surface rust may be present. Fence components are properly aligned (all connections are sound).

Poor: Chain link fence or screening has moderate deterioration. Coating may have moderate failure - surface rust may be prevalent. Components may be slightly bent or misaligned - connections may be slightly loose. Fabric may have snags or holes (areas may be slightly stretched or deformed).

Severe: Chain link fence or screening has extensive deterioration. Coating may have extensive failure - there may be section loss. Components may be bent or

misaligned - connections may be loose. Fabric may have numerous snags or holes (areas may be stretched or deformed).

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2.8 WATERWAYS

2.8.1 Introduction

For any structure crossing a waterway, scour, flooding, and debris buildup can affect the stability. Scour is defined as the removal of streambed material due to stream flow. Scour can cause pier/abutment instability and bridge failures. Flooding can cause failures due to erosion of the embankment or lateral water flow pressures on the bridge. Debris buildup/aggradation can cause loading on substructure elements for which they were not designed, as well as create conditions causing scour and flooding.

The most common cause of bridge failures is water flow. For most of these failures, scour of the substructure is the failure mechanism. Part of every Routine Inspection is rating the channel and channel protection according to the Federal Highway Administration (FHWA) General Appraisal Rating Guidelines. In addition, a waterway adequacy appraisal is also required.

2.8.2 Channel and Channel Protection, NBI Condition Rating

A complete discussion of channel types, waterway changes, and scour is beyond the scope of this manual. However, main points will be addressed. Complete treatment of these topics may be found in *the Manual for Bridge Evaluation* and in the *Safety Inspection of In-Service Bridges Participant Workbook*.

The channel condition does not influence the physical condition of any bridge component. However, there are three reasons to perform waterway inspections. The first is to identify critical damage to the bridge caused by the waterway. Damage is usually settlement caused by scour and exposure/deterioration of substructure elements. The second reason is to record and monitor the channel conditions for any changes caused by natural or manmade circumstances. The final reason to perform waterway inspections is to establish the scour potential of a particular bridge site.

2.8.2.1 Scour and Channel Protection

Scour is the removal of material from the streambed or embankment as a result of the erosive action of stream flow. It is important for the inspector to recognize that there are four types of scour that can affect a bridge and channel. These are:

General scour – occurs whether there is a bridge crossing or not. It includes natural streambed movements and natural river lateral movements over time.

Contraction scour – caused by an increase in water flow velocity as a result of channel constriction. Building a bridge over a waterway will often cause this. Contraction scour will cause a lowering of the streambed across the entire channel width.

Local scour – occurs around an obstruction within the waterway, such as a bridge pier or abutment. The obstructions cause water turbulence, which stirs up the streambed material and allows it to be removed. Local scour is more severe when

substructures do not line up with the flow of the stream or when blunt shapes (squared off pier ends) are used instead of streamlined shapes (round or tapered pier ends).

Lateral Stream Migration – caused by stream meandering, channel widening or man-made channel changes. It affects abutments, wingwalls, and approach embankments.

Several factors affect a waterway's potential for scouring. Conditions which are more likely to lead to scour include high stream flow velocities, loose or granular streambed materials, a stream flow direction that is skewed relative to the bridge substructure units, and wide floodplains constricted to a narrow hydraulic opening at the bridge site.

To guard against the erosive effects of scour, the channel banks must be protected and the stream must be controlled. Channel protection and hydraulic control structures cover both natural and man-made features. Natural channel protection includes banks with well established vegetation, and streambeds consisting of bedrock, boulders, etc. Man-made control structures and channel protection include the following:

Riprap – large stones/boulders placed along the bank or substructure units that rely upon their mass for stability against the flow of water. Riprap may be natural or man-made. It is usually placed on a geotextile fabric to prevent erosion of the soil underneath. The size of the rip rap is typically determined by design calculations.

Channel lining – a layer of concrete across part of or the entire channel to virtually eliminate scour risk. Channel linings also increase the flow rate of the stream to drain the upstream region more quickly.

Gabions – large baskets of stone contained by a wire mesh. The baskets are normally tied together and anchored to the bank. They may be placed on steeper slopes.

Slope stabilization – treatment of the existing bank with plantings, geotextile or wire mesh to prevent erosion.

Footing aprons – a protective concrete layer that surrounds the structural footing of a pier, abutment or wingwall.

Guide banks/wing dams – devices that direct/control natural stream flows to prevent meandering.

Spur dikes – devices constructed to redirect flood flows smoothly through the bridge waterway opening protecting the embankments.

Figure 2.8.2.1-1: Well-Vegetated, Stable Natural Channel.

Figure 2.8.2.1-2: Riprap Channel Protection.

Figure 2.8.2.1-3: Concrete Channel Lining Protection.

2.8.2.2 Inspection

A channel condition rating covers the areas upstream, downstream, and directly under the bridge. The type of waterway has a significant influence on the condition of a channel. Fast-flowing streams and rivers may result in channel changes and a greater likelihood of scour. Waterways can be very difficult to control with man-made devices. Slow-flowing streams and rivers may result in channel obstructions due to debris and ice build-ups and slowly changing channel locations. Lake channels are subject to location changes due to aggradation and can create ice pressures on the substructure units.

Channel inspection can be broken down into three parts: channel condition upstream, channel condition at the bridge, and channel condition downstream. Channel inspection upstream of the bridge should include the following items:

- Looking for stable banks. Stable banks will be gradually sloped and well vegetated.
- Looking for evidence of bank instability, such as sloughing due to scour and lateral steam movements.
- Looking for debris or aggradation that could redirect the stream flow.
- Looking for signs of streambed degradation.
- Noting any signs of lateral stream movement. General waterway alignment should normally be centered under the structure. Sketches or photographs should be taken during each inspection to monitor upstream alignment.
- Checking for other unusual obstructions to stream flow such as cattle guards and fences. These can trap debris, resulting in possible sediment buildup and redirection of the steam flow.
- Checking for evidence of stream flow within the floodplain.

- Noting changes within the floodplain, such as trees, houses, newly paved areas, etc. These affect main channel stream flow during floods.

Channel inspection under the bridge and at the bridge should include the following items:

- Checking for evidence of structure settlement, such as substructure tipping or superstructure vertical misalignments. A plumb bob or sighting along the bridge railing will often help in making this determination.

- Probing around the substructure units with a rod to check for areas of local scour.

- Checking if stream flow is impinging behind protective devices. For example, stream flow should not be pooling on the upstream side of a wingwall.

- Looking for evidence of overtopping by floodwaters. This evidence may include damaged girders or truss bottom chords; debris lodged between girders, cross frames, bearing areas, etc.; scrape or water marks on surrounding trees or bearings exhibiting transverse displacements.

- Noting if the superstructure or approaches are located within the flood plain.

- Looking for debris or sediment buildup that could redirect the stream flow.

- Looking for debris caught on the upstream end of piers or between pier columns. This can create localized increased water velocities.

- Looking for signs of degradation of the streambed.

- Noting if the hydraulic opening is sufficient to allow flood waters to pass without constriction, scour or damage to the superstructure.

Channel inspection downstream of the bridge should include the following items:

- Looking for stable banks. Stable banks will be gradually sloped and well vegetated.

- Looking for evidence of bank instability, such as sloughing due to scour and lateral steam movements.

- Looking for debris or sediment buildup that could redirect the stream flow and affect “getaway” flow conditions.

- Looking for signs of streambed degradation.

- Noting any signs of lateral stream movement. General waterway alignment should normally be centered under the structure. Sketches or photographs should be taken during each inspection to monitor downstream alignment.

- Checking for other unusual obstructions to stream flow such as cattle guards and fences. These can trap debris, resulting in possible sediment buildup and increasing tail water depths.

- Noting obstructions within the floodplain, such as trees, houses, etc.

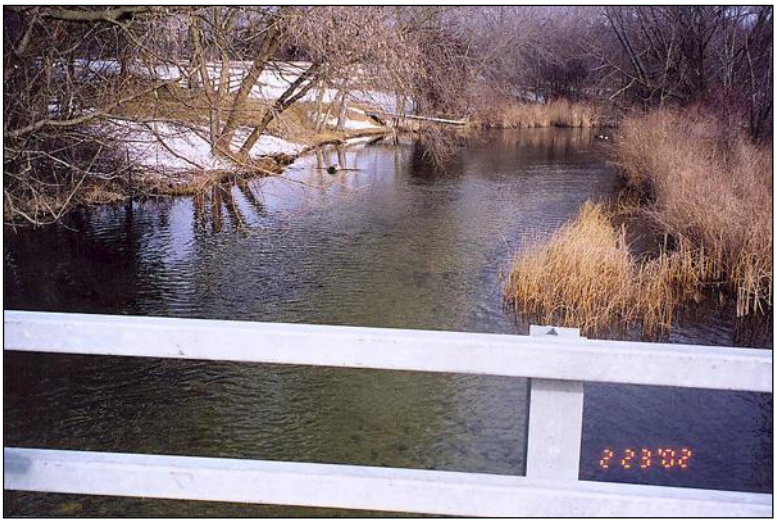

Figure 2.8.2.2-1: Stable Upstream Channel.

Figure 2.8.2.2-2: Aggradation in Front of Abutment and Debris Buildup in Front of Pier.

Figure 2.8.2.2-3: Heavily Vegetated Downstream Channel.

2.8.2.3 Rating

Channel condition ratings are concerned with water flow, channel protection, channel damage due to flow, and waterway stability. Damage to the substructure, superstructure or culvert should be reflected in their appropriate National Bridge Inventory (NBI) condition ratings.

Rating the channel is done according to the Federal Highway Administration (FHWA) General Condition Rating Guidelines. The numeric condition ratings of these guidelines describe existing channel and channel protection devices compared to their as-built condition. Ratings range from 9 to 0, with 9 describing components in excellent condition and 0 describing failed devices.

Because only a single number is used to rate the channel and its protection, the rating must characterize its overall general condition. The rating should not be used to describe local areas of deterioration, such as an isolated area of washed-out riprap or an old tire discarded in the river upstream. However, widespread riprap washouts or large quantities of trash/debris located within the channel would certainly influence the rating. A proper rating will therefore consider deterioration severity plus the extent to which it affects the channel condition and performance. The FHWA channel and channel protection general condition ratings are as follows:

Code (Rating)	Description
N	**NOT APPLICABLE –** use only when bridge is not over a waterway (channel).
9	**EXCELLENT CONDITION –** there are no noticeable or noteworthy deficiencies which affect the condition of the channel.

8 **VERY GOOD CONDITION –** banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable state.

7 **GOOD CONDITION –** bank protection is in need of minor repairs. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift.

6 **SATISFACTORY CONDITION –** bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor streambed movement evident. Debris is restricting the channel slightly.

5 **FAIR CONDITION –** bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel.

4 **POOR CONDITION –** bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the channel.

3 **SERIOUS CONDITION –** bank protection has failed. River control devices have been destroyed. Streambed aggradation, degradation or lateral movement has changed the channel to now threaten the bridge and/or approach roadway.

2 **CRITICAL CONDITION –** the channel has changed to the extent the bridge is near a state of collapse.

1 **"IMMINENT" FAILURE CONDITION –** bridge is closed because of channel failure. Corrective action may put it back in light service.

0 **FAILED CONDITION –** bridge closed because of channel. Replacement is necessary.

One suggested method for establishing the channel and channel protection rating is to identify phrases within the general condition guideline language that describes a condition more severe than what actually exists. The correct rating number will be one number higher than the one describing the more severe condition.

For example, suppose the channel has extensive sloughing upstream, plus small amounts of debris within the channel. The sloughing has caused the stream to begin pooling behind one of the wingwalls, but there is no evidence of scour/undermining. Condition rating 5 indicates that bank protection is being eroded, and that the embankment has major damage. Trees and brush also restrict the channel. Condition rating 4 indicates that bank and bank protection is severely undermined. River control devices have severe damage, and large deposits of debris are in the channel. Using the method described above, Condition rating 4 describes a situation more severe that what actually exists within the channel. Therefore, a rating of 5 would be appropriate.

2.8.3 Waterway Adequacy National Bridge Inventory (NBI) Appraisal Rating

Waterway adequacy is an appraisal of the existing hydraulic opening and its ability to handle the water flowing through it. It compares the existing condition to the desirable condition of no roadway overtopping during a flood. The hydraulic opening is the opening available for water to pass under the bridge. It is essentially an area bounded by the streambed, abutment faces, and the underside of the bridge superstructure.

The waterway adequacy appraisal considers overtopping frequency as well as the significance of traffic delays caused by overtopping. When overtopping frequency information is available, the descriptions given in the table for chance of overtopping mean the following:

1. **Remote** – greater than 100 years.
2. **Slight** – 11 to 100 years.
3. **Occasional** – 3 to 10 years.
4. **Frequent** – less than 3 years.

Adjectives describing traffic delays mean the following:

1. **Insignificant** – minor inconvenience. Highway passable in a matter of hours.
2. **Significant** – traffic delays of up to several days.
3. **Severe** – long term delays to traffic with resulting hardship.

Points of concern regarding waterway adequacy include sediment/debris accumulation or vegetation growth that may block or partially block the hydraulic opening. The inspector should also note any newly paved areas or developments that would increase the amount of

runoff into the stream and high-water marks on the bridge or surrounding terrain. Local residents are often excellent sources of information regarding high water. With this said, the waterway adequacy appraisal rating is established during the Initial Inspection, and it should not be changed unless it has been reviewed and approved by the Inspection Program Manager.

The Federal Highway Administration (FHWA) waterway adequacy appraisal ratings are as follows:

Functional Classification			
Principal Arterials – Interstates, Freeways, or Expressways			
	Other Principal and Minor Arterials and Major Collectors		
		Minor Collectors, Locals	
Code			Description
N	N	N	Bridge not over a waterway
9	9	9	Bridge deck and roadway approaches above flood water elevations (high water). Chance of overtopping is remote.
8	8	8	Bridge deck above roadway approaches. Slight chance of overtopping roadway approaches.
6	6	7	Slight chance of overtopping bridge deck and roadway approaches.
4	5	6	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with insignificant traffic delays.
3	4	5	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with significant traffic delays.
2	3	4	Occasional overtopping of the bridge deck and roadway approaches with significant traffic delays.
2	2	3	Frequent overtopping of the bridge deck and roadway approaches with significant traffic delays.
2	2	2	Occasional or frequent overtopping of the bridge deck and roadway approaches with severe traffic delays.
0	0	0	Bridge is closed.

2.8.4 Procedure for Evaluation of Scour at In-Service Structures

The scour condition rating (B.C.11) represents the observed or measured scour at the bridge. The TL should review the scour vulnerability (B.AP.03) code to verify the bridge has been determined to be stable or unstable for the appraised scour conditions found during the inspection. If observed or measured scour condition is believed to affect bridge strength and/or stability, the TL should review the hydraulic design, scour appraisal/evaluation, and POA for the design scour depth, critical scour depths, and assumptions used to determine the scour vulnerably.

When evaluating the scour condition rating of a bridge for strength and stability, the TL may need to review the hydraulic design, scour appraisal/evaluation, and/or POA located in the bridge file for the design scour depth, critical scour depth, and any assumptions used. The TL should reevaluate the scour vulnerability (B.AP.03) when observed conditions are not consistent with the scour design or assumptions used in the scour appraisal. The TL must notify the responsible PM of the scour condition finding. Reevaluation may result in a scour plan of action being written and implemented.

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2.9 IMPACT PROTECTION SYSTEMS

2.9.1 Introduction

Impacts due to waterborne and roadway traffic can cause severe damage to structures due to the high energy imparted to the structure. The structure's integrity is important since it is typically in use by the general public at the time of impact. Impact Protection Systems are structural appurtenances that are often located adjacent to structures, such as bridges and traffic operations support systems. The purpose of impact protection systems is to shield structures from collision damage while reducing the likelihood of severe injury to the vehicle occupants caused by a collision with an immovable object. Most systems reduce the severity of crashes by redirecting errant vehicles or vessels, and absorbing the energy of crashes. Redirection and energy absorption minimize injury to individuals and damage to the structures being protected.

It is recommended that Impact Protection Systems be inspected at the time the associated structure is inspected. WisDOT has created an assessment for tracking the condition of the exposed areas of waterway impact protection systems. The assessment is Dolphin and Fender System (Assessment 9290). It is the inspector's responsibility to examine the exposed surfaces of each assessment and determine its assessment state. Additional information may be on the inspection form from the underwater inspection. Refer to part 2 chapter 7 for additional information on the Dolphin and Fender System Assessment, and the Assessment State definitions.

2.9.2 Waterway Impact Protection Systems

2.9.2.1 Cell Dolphins

Cell Dolphins are typically constructed of a large-diameter steel casing pipe or interlocking steel sheet piles in a circular configuration. A typical cell dolphin is constructed by driving the casing / steel sheet piles into the soil and then filling with cell with concrete or granular material. If loose fill materials are used, a reinforced concrete or asphalt cap is typically placed on top to retain the fill. Settlement of the fill material may result in deterioration of the cap. Frequently, timber piles are driven around the perimeter of the cell for increased protection. Refer to Figure 2.9.2.1-1 for a view of a cell dolphin.

Figure 2.9.2.1-1: Cell Dolphin During Flood Conditions.

2.9.2.2 Pile Cluster Dolphins

Pile cluster dolphins are composed of groups of battered or vertical piles that are fastened or bound together at the top. Typical fasteners include chains, steel cables, wood chocks, bolts, and steel shells. Piles are typically timber, but composite and steel piles are also used. When steel piles are used, a cast-in-place concrete cap is generally used. Refer to Figure 2.9.2.2-1 for a view of a timber pile cluster dolphin.

Figure 2.9.2.2-1: Timber Pile Cluster Dolphins.

2.9.2.3 Fender Systems

The purpose of fender systems is to protect piers, bents, and other structural bridge members from impact damage due to docking or errant vessels. Fenders are usually positioned so that the horizontal angle of impact (channel navigation orientation) with respect to the fender line orientation is small. Fenders are designed to accept the forces from vessel impact while redirecting the course of the vessel. This reduces or eliminates the impact force upon the structure. Fender systems can be either supported independently or attached to the substructure unit. Fender systems can be composed of timber, rubber, composite, concrete, or steel.

Independently Supported

Independently supported fender systems typically consist of vertical or battered piles with horizontal members connecting the piles so the system acts as a unit. Some are installed between dolphins on either side of the structure they protect. Refer to Figure 2.9.2.3-1 for a view of an independently supported timber fender system.

Figure 2.9.2.3-1: Independently Supported Timber Fender System.

Substructure Supported

Substructure-supported fenders vary in type from simple rails, used as rubbing strips, attached directly to the substructure to more complex systems, which are designed to dissipate energy when struck by a vessel. The inspector should review the fender manufacturer's literature in order to be familiar with the individual components of this more complex system. Chains or bolts that fasten these complex systems to the substructure unit must also be inspected. Refer to Figure 2.9.2.3-2 for a view of a substructure-supported timber fender system.

Figure 2.9.2.3-2: Substructure-Supported Timber Fender System.

2.9.2.4 Fender System Materials

A variety of materials are used either independently or in combination to create fender systems. The following items are materials typically used in the construction of fender systems.

Timber Fenders

Timber fenders are composed of vertical and/or horizontal timber members that are either attached to the substructure or are independently supported. Impact energy is absorbed by plastic deformation and crushing of the timber members. Timber members are an excellent choice for protection against minor collisions due to their low cost. However, timber fender system members for large-scale impacts would have to be extremely large and thus become both impractical and uneconomical.

Rubber Fenders

Rubber fenders are available in a wide variety of shapes. Rubber fender systems are attached directly to the concrete pier/abutment by bolts, or suspended in place using chains. Impact energy is absorbed by elastic deformation of the rubber elements through bending, compression, and shear.

Concrete Fenders

Concrete fenders are composed of hollow, thin-walled concrete boxes attached to the substructure. Typically, a timber fender is also attached to the outer face of the concrete fender. Impact energy is absorbed through the buckling and crushing of the concrete walls.

Steel Fenders

Steel fenders are composed of thin-walled members and bracing elements which form a box around, and are attached to, the substructure unit. Timber or composite facing is usually

attached to the steel fender to prevent sparks during impacts from steel ships. Compression, buckling, and bending of the steel elements will absorb the impact energy.

2.9.2.5 Jetties

Jetties, or manmade protection islands, typically consist of sand or rock core. An outer layer of heavy rock armor is placed over the sand or rock core to protect the island from erosion due to wave, current, and ice action.

2.9.2.6 Floating Protection Systems

Various types of floating systems are used to provide structure protection. Several of these systems include:

Cable Net Systems

Ships are stopped by a system of cables anchored to the bottom of the waterway and suspended by buoys located in front of the bridge piers.

Anchored Pontoons

Large floating pontoons are anchored to the bottom of the waterway in front of the bridge piers to absorb vessel impact.

Camels

Camels are circular members that float adjacent to the substructure. They are typically constructed of composites or timber.

2.9.3 Roadway Impact Protection Systems

Vehicle barrier systems are discussed in 16 5 and 11-45-1 of the Wisconsin Department of Transportation (WisDOT) Facilities Development Manual (FDM). The primary purpose of these systems is to protect the traveling public from severe injury caused by vehicular impact with rigid objects. They are also used to keep vehicles from driving down steep slopes, into waterways, or to separate oncoming traffic. This section will discuss the condition assessment of appurtenances that guard structures from vehicular impact.

2.9.3.1 Cable Guard

A cable guard is considered a semi-rigid impact protection system because it is designed to absorb the energy of vehicles through deformation of the system. Cable guard is comprised of a series of horizontal steel cables attached to vertical steel posts that are driven into the ground to provide anchorage. The cables are anchored to concrete blocks located underground at either end. When vehicles strike the system, the tensioned cables catch and redirect the vehicle away from the structure or hazard the system protects. Refer to Figure 2.9.3.1-1 for a view of a cable guard.

Figure 2.9.3.1-1: Section of Cable Guard Next to a Steel Plate Beam Guard.

2.9.3.2 Steel Plate Beam Guard

A beam guard is considered a semi-rigid impact protection system because it is designed to absorb the energy of vehicles through deformation of the system. Beam guard is comprised of corrugated horizontal steel rail sections that are bolted to vertical steel or wood posts driven into the ground to provide anchorage. When vehicles strike the system, the rails bend and develop tension to catch and redirect the vehicle away from the structure or hazard that the system protects. Refer to Figure 2.9.3.2-1 for a view of a newly installed beam guard.

Figure 2.9.3.2-1: Newly Installed Beam Guard.

2.9.3.3 Crash Cushions and Impact Attenuators

A crash cushion is considered a semi-rigid impact protection system because it is designed to absorb the energy of vehicles through deformation of the system. In Wisconsin, impact attenuator arrays (sand-filled barrels) and accordion style steel beam guards with compressible cartridges and cylinders are used in areas where space limitations prevent the use of beam guard systems. These systems are also designed to safely redirect vehicles that do not contact the system head on. Refer to Figure 2.9.3.3-1 for a view of an impact attenuator system and Figure 2.9.3.3-2 for a view of a crash cushion.

Figure 2.9.3.3-1: Impact Attenuator System.

Figure 2.9.3.3-2: Crash Cushion.

2.9.3.4 Concrete Barrier Walls

A concrete barrier wall is considered a rigid impact protection system because it is not designed to absorb the energy of vehicles that strike it. Instead, concrete barrier is designed to safely redirect errant vehicles. Refer to Figure 2.9.3.4-1 for a view of a concrete barrier wall.

Figure 2.9.3.4-1: Concrete Median Barrier Wall.

2.9.3.5 Concrete Crash Walls

Concrete crash walls are supplementary structural elements designed to resist the high-energy impacts associated with impacts from trucks and trains. These walls are located in advance of, and adjacent to, the structure to redirect vehicles that would strike structural elements susceptible to failure from impact. Refer to Figure 2.9.3.5-1 for a view of crash wall installed at an overpass bridge as part of a highway upgrade.

Figure 2.9.3.5-1: Concrete Crash Wall Along an Overpass Bridge Pier.

2.9.4 Inspection of Impact Protection Systems (Water)

While noticeable above water deterioration should be noted during routine inspections of the structure, the primary inspection would be conducted during the underwater inspection of the substructure since the inspector-diver will have closer access to all surfaces.

2.9.4.1 Inspection of Cell Dolphins

The steel sheet piling or steel casing should be inspected for corrosion, cracking, and sheet pile joint separation. Special attention should be given to areas in the splash zone where corrosion is usually most significant. The steel sheet piling should be examined for tightness of the interlocks, any seepage of loose fill materials, alignment, lateral movement, plumbness, and impact damage. The cell dolphin should also be inspected for a spalled cell cap, broken or missing cleats, and damaged fenders that may be present. Scour at the channel bottom should also be investigated by sounding the channel in the general area of the dolphin. The remaining thickness of the steel should be determined through non-destructive testing and the pile length recorded to compare inspection findings with previous inspections. Any timber piles surrounding the steel cell dolphin should be inspected according to the methods describes in Section 2.9.4.2. Refer to Figure 2.9.4.1-1 for a view of cell dolphin deterioration.

Figure 2.9.4.1-1: Cell Dolphin Deterioration.

2.9.4.2 Inspection of Pile Cluster Dolphins

The timber piling should be inspected for rot, fire damage, impact damage, and lateral movement. Special attention should be given to the tops of the piles and the areas in the splash zone where section loss is common. Scour at the channel bottom should also be investigated by sounding the channel in the general area of the dolphin. The soundness of the timber may be determined by incremental coring, but care should be taken to plug any holes created with treated hardwood dowels. Wire rope and connection bolts that secure the piles should be examined for corrosion. Any missing or loose bolts and wires should be noted. Refer to Figure 2.9.4.2-1 for a view of pile cluster dolphin deterioration.

Figure 2.9.4.2-1: Pile Cluster Dolphin Deterioration.

2.9.4.3 Inspection of Fender Systems

All piles of fender systems should be inspected in the same manner as presented in Section 2.9.4.2. Timber, concrete, and steel members should be inspected for their typical material deficiencies. All members should be examined for collision damage. Connections should be examined for loose and/or corroded bolts. Rubber fender systems should be inspected for loose or broken support chains. Pier supported fenders should be checked for adequate attachment to the pier. Refer to Figure 2.9.4.3-1 for a view of a failed rubber fender system. All failed fenders systems should be noted in the inspection report.

Figure 2.9.4.3-1: Failed Rubber Fender System.

2.9.4.4 Inspection of Jetties

Island protection systems should be examined for collision damage that may have displaced the heavy rock armor layer exposing the sand or rock core beneath it. The island should also be examined for scour.

2.9.4.5 Inspection of Floating Protection Systems

Cable net systems should be examined for deterioration or breaks in the netting that could weaken the system. Buoys should be examined to ensure that they are floating as intended and that they are not cracked.

Anchored pontoons should be examined for collision damage. Pontoons should be examined to insure that they are floating as intended and that they are not cracked.

2.9.5 Inspection of Impact Protection Systems (Land)

Neither elements with defects nor assessments have been incorporated into Wisconsin's structure inspection program for these roadway traffic safety systems, but due to their importance in protecting the structures from vehicular impact their condition should be

evaluation during structure inspection. Conditions are documents under the structure inspection notes of the structure inspection. Any concerns or maintenance actions are relayed onto the roadway maintenance engineer in charge of the roadway corridor.

2.9.5.1 Inspection of Cable Guard

The horizontal steel cables and vertical steel posts should be inspected for corrosion and collision damage. All loose or missing connections should be noted as well as material deterioration in the connections. Anchorages should be secure at both ends of the system, as well as at the intermediate steel posts throughout the system. Cable guards should also be inspected for excess sag in the cables. The distance to the structure or hazard, which the system is protecting, should be checked. This distance should be a minimum of 12 feet away from the cable guard system. Refer to Figure 2.9.5.1-1 for a view of a cable guard next to a beam guard.

Figure 2.9.5.1-1: Cable Guard Next to a Beam Guard.

2.9.5.2 Inspection of Steel Plate Beam Guard

The inspector should check the vertical posts for material deterioration, collision damage, anchorage and adequate spacing. The steel rail should be inspected for material deterioration and collision damage. Connections should be inspected for material deterioration. Loose or missing bolts should also be noted. Note that portions of the steel rail are not designed to be bolted to the posts in specific areas near the attenuator. Standard details can be obtained through the Wisconsin Department of Transportation (WisDOT) Facilities Development Manual (FDM) for reference on beam guard design. The distance to the structure or hazard, which the system is protecting, should be checked. Refer to Figure 2.9.5.2-1 for a view of a damaged beam guard.

Figure 2.9.5.2-1: Beam Guard that was Damaged During a Collision.

2.9.5.3 Inspection of Crash Cushions and Impact Attenuators

Crash cushions should be inspected for collision damage and functionality. The barrels or cells of the attenuator system should be inspected for cracking or leakage of the material from the inside the barrel. Refer to Figure 2.9.5.3-1 for a view of a damaged impact attenuator.

Figure 2.9.5.3-1: Damaged Impact Attenuator.

2.9.5.4 Inspection of Concrete Barrier Walls

The inspector should check concrete barrier walls for collision damage and deterioration. The barrier should be secure and capable of performing its designed intent. Tipping or settling portion of the barrier wall may create snag points that would catch rather than

redirect a vehicle if struck. These should be noted on the inspection form. Deterioration and spalling should also be noted. Refer to Figure 2.9.5.4-1 for a view of a concrete barrier wall.

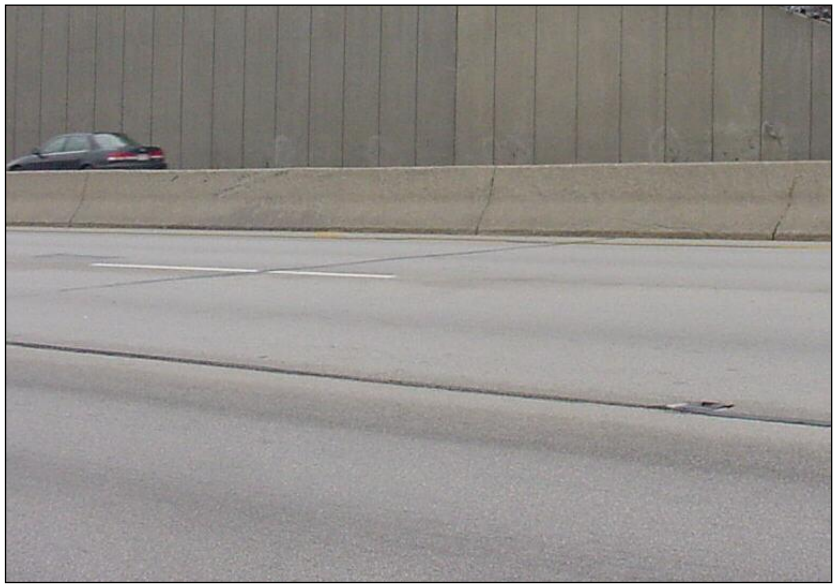

Figure 2.9.5.4-1: Concrete Median Barrier Wall.

2.9.5.5 Inspection Of Concrete Crash Walls

Concrete crash walls should be inspected for collision damage and material deterioration. Tipping or settling portions of the barrier wall may create snag points that would catch rather than redirect a vehicle if struck. These should be noted on the inspection form. Any spalling of the concrete should also be noted in the inspection form.

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Appendix A: Common Defects

Section A.1: Material Defects

A.1.1 Steel Defects

A.1.1.1 Corrosion (1000)

Section loss of steel is caused by corrosion heavy or severe enough to physically measure a reduction in member thickness.

The inspector should look for areas of laminate rust on the steel surface or areas of heavy blistering paint (caused by a build-up of rust under the paint surface). Common locations are horizontal surfaces, areas subject to salt spray from roads or areas near water surfaces. The most effective tools used to remove the laminate rust are chipping hammers and paint scrapers. Severe corrosion will usually consist of fairly loose, brown laminate rust, and a hard, tightly adhered black corrosion product on the base metal surface. A chipping hammer is usually required to remove this black corrosion. Safety glasses should always be worn when using a chipping hammer.

Once the corrosion has been removed, the amount of remaining section should be measured and recorded. Methods to measure the remaining section include the use of a caliper, a micrometer or ruler with a straight edge spanning the depression to indicate the original steel surface. The surface area affected by the corrosion should also be measured.

Some corrosion is so severe that holes are created through the steel. This situation should be recorded as “through thickness section loss” rather than “100 percent section loss” since the latter term suggests that the entire member or element has corroded away.

Condition States

Condition State 1 indicates no corrosion exists or it has been repaired and painted over corrosion that has been painted over will not prevent further corrosion from occurring and should not be considered a Condition State 1. Repairs suggest that the member or element has been retrofitted so that it has its original load-carrying capacity. The repairs may be in the form of bolted splices or welded splices.

Condition State 2 indicates that corrosion of the steel has initiated or freckled rust is present.

Condition State 3 indicates that section loss or pack rust exists. If the condition is not repaired/rehabilitated, a structural analysis may be warranted in the near future. The inspector should consider recommending cleaning and painting of the member to help prevent further section loss.

Condition State 4 indicates that excessive section loss exists, and that structural analysis results indicate member capacity or serviceability is compromised.

A.1.1.2 Cracking – Steel (1010)

Steel cracking addresses fatigue cracks, regardless if they occur within a Nonredundant steel tension member (NSTM) or non-NSTM bridge. Cracking in substructure elements will most often occur in steel pier caps or cross girders. Any cracks not previously detected should be evaluated to determine the potential for fracture.

Inspectors should be especially thorough at details which are prone to developing fatigue cracking. Refer to Appendix for a list of fatigue details. Detection of these cracks will most often occur during In-Depth or NSTM Inspections but can still be found during Routine Inspections as well. Any crack defects classified as a Condition State 3 or worse should be immediately reported to the Inspection Program Manager. Serious consideration should be given to closing the bridge until the extent and cause of cracking is determined and appropriate remedial action is taken.

Condition States

Condition State 1 indicates that no cracking exists in the member.

Condition State 2 indicates that a crack exists, but it has either been arrested by drilling holes at the tips or it has been repaired with bolted splices or welded splices. Shallow or short cracks may also be ground out. Unrepaired, self-arrested, or cracks not arrested with drilled holes may also fall into this Condition State, but they are located in a non-critical or redundant member.

Condition State 3 indicates that a crack which has not been arrested or repaired exists in a primary load-carrying member, including newly-discovered crack or existing cracks that are still growing between inspection cycles. Cracks of this nature suggest it is fatigue related, stable crack growth.

Condition State 4 indicates that a very large crack exists in any primary load-carrying member. Newly-discovered cracks are greater than 3 inches in length, or existing cracks have grown more than 6 inches within the last 24 months. Cracks of this nature suggest they developed in an uncontrolled brittle fashion.

A.1.1.3 Connection (1020)

Connections can be constructed from two or more members or elements that may be bolted, riveted, or welded together. Connection deficiencies include any deterioration or damage of fasteners, such as rivets, bolts, or welds and connection plates (gusset plates). Any suspected loose or deteriorated fasteners should be struck with a hammer to determine soundness. Welded connections should be visually inspected for fatigue cracks developing from weld imperfections or overstress. Connections are also often subjected to distortion due to pack rust (corrosion occurring between two pieces of lapped steel). The amount of separation caused by pack rust and any distortion to the steel should be measured and recorded.

Condition States

Condition State 1 indicates that the connection is in place and is functioning as intended.

Condition State 2 indicates that there are loose fasteners and/or pack rust without distortion. The connection is in place and functioning as intended despite these defects.

Condition State 3 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. Although these defects are present, the structural capacity of the structure is not reduced.

Condition State 4 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. These defects reduce the structural capacity of the member assembly.

A.1.1.4 Distortion (1900)

This defect is used to report distortion from the original line or grade of the element. It is used to capture all distortion regardless of cause.

Condition States

Condition State 1 indicates no distortion.

Condition State 2 indicates there is distortion that does not require mitigation or previously reported distortion which has been mitigated.

Condition State 3 indicates there is distortion present which requires mitigation but does not warrant a structural review.

Condition State 4 indicates there is distortion present which requires mitigation and warrants a structural review to determine the strength or serviceability of the element or a structural review has been completed and the distortion impacts the strength or serviceability of the bridge.

A.1.1.5 Microbial Induced Corrosion (8001) [ADD]

This defect is used to report degradation of steel caused by Microbial Induced Corrosion (MIC). Microbial induced corrosion is corrosion on metallic surfaces due to bacteria or fungi. Pitting is the typical visual evidence that MIC is occurring. However, in most instances, an inspector will not know if the corrosion is microbial induced unless specifically tested for MIC. Tests can be completed in the field to determine if MIC is present. The Microbial Induced Corrosion defect is only to be used on substructure elements and when determined through testing. The testing results shall be uploaded to the bridge inspection report to remain on file.

Condition States

Condition State 1 indicates no corrosion exists or it has been repaired or painted over and the water has been tested and no MIC exists.

Condition State 2 indicates water tested and MIC exists. Orange powder may exist but little or no corrosion exists. The surface under the orange powder may be shiny, indicating that MIC is actively attacking the steel member.

Condition State 3 indicates significant section loss exists or isolated areas of deep pitting and corrosion. The structural capacity of the member has not been reduced. The inspector should consider recommending cleaning and painting to help prevent further section loss.

Condition State 4 indicates that excessive section loss exists due to MIC attack and may have removed enough section loss to warrant a structural review. Results indicate member capacity or serviceability is compromised.

A.1.2 Reinforced Concrete Defects

A.1.2.1 Delamination/Spalls/Patched Areas (1080)

Delamination is the result of a loss of bond between concrete and rebar due to expanding corrosion product of corroded reinforcement and can often lead to spalling of the concrete. Spalling is a depression in concrete where sections of concrete separate from aggregate or the reinforcement. Spalls result from the separation and removal of a portion of the surface concrete, revealing a fracture roughly parallel to the surface. Inspectors should use sketches showing the location and dimensions of delaminations and spalls. The depth of spalls should also be recorded.

Condition States

Condition State 1 indicates that there are no apparent delaminations, spalls, or patched areas.

Condition State 2 indicates that there are delaminated and/or spalled areas that are less than 1” deep or less than 6” in diameter or in either the length or width dimension. . Any patched areas are sound.

Condition State 3 indicates that there are spalled areas that are greater than 1” deep or greater than 6” in diameter, or in either the length or width dimension. Patched areas are unsound or showing distress. The delamination, spall, or patched area does not warrant a structural review.

Condition State 4 indicates that the depths of delaminations, spalls, or patched areas warrant a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.2.2 Exposed Rebar (1090)

Exposed rebar can be the result of spalls, delaminations, or damage. Exposed rebar will be susceptible to accelerated corrosion and deterioration due to its exposure to the environment. Inspectors shall record the area and location of exposed rebar and record any measureable section loss.

Condition States

Condition State 1 indicates that there is no exposed rebar in the bridge member.

Condition State 2 indicates that rebar is exposed and surface corrosion may be present but without section loss.

Condition State 3 indicates that rebar is exposed and surface corrosion is present with measurable section loss but does not warrant a structural review. Caliper/thickness gauge measurements should be used to determine the amount of section loss.

Condition State 4 indicates that rebar is exposed and surface corrosion is present with measurable section loss and warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge. Caliper/thickness gauge measurements should be used to determine the amount of section loss

A.1.2.3 Cracking – Reinforced Concrete (1130)

Cracking is common in reinforced concrete due to concrete's lack of strength in tension. Cracking can be a result of structural forces, temperature, or other forces. Common locations for structural cracking include negative moments over a support, near supports from shear stress, and the bottom of elements near mid-span from positive moments. Temperature cracking is caused by thermal expansion and contraction. Shrinkage cracks are due to the shrinkage of concrete caused by the curing process. It can cause either pattern (map) cracking or random cracks. Mass concrete cracks occur due to thermal gradients, the differences between interior and exterior temperatures, in massive sections immediately after the placement and for a period of time afterwards. While all cracking allows moisture and chemicals to infiltrate the concrete, structural cracking is considered more critical as it could be a result of a larger issue. Inspectors should note all visible cracks, recording their type, width, length, and location.

Condition States

Condition State 1 indicates that there is cracking present less than 0.012" wide or any cracks larger are sealed.

Condition State 2 indicates that there is cracking present between 0.012" and 0.05" wide. Where efflorescence is present, it's minor and there is no evidence of rust staining.

Condition State 3 indicates that there is cracking greater than 0.05" wide. Where efflorescence is present there is heavy build-up and/or rust staining.

Condition State 4 indicates the cracking warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.2.4 Abrasion/Wear (1190) (PSC/RC)

Vehicular traffic is a source of abrasion and wear on concrete decks. Abrasion occurs on concrete piles that are subjected to flowing water or tidal flows. Mechanical wear of concrete members sometimes occurs due to movement of the fasteners against their holes when connections become loose.

Condition States

Condition State 1 indicates no abrasion.

Condition State 2 indicates abrasion has exposed coarse aggregate but the aggregate remains secure in the concrete.

Condition State 3 indicates coarse aggregate is loose or has popped out of the concrete matrix due to abrasion.

Condition State 4 indicates the condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.2.5 Chloride Concentration (8905) [ADD]

Chloride contamination is caused primarily by deicing agents. Chloride ions infiltrate concrete through surface cracks. Chloride ions break down the passivation layer around bare reinforcing steel allowing for corrosion to occur. Testing for chlorides involves drilling out samples and testing the content at certain depths. Once the chloride content reaches a certain limit at the reinforcing depth, bare steel reinforcing is at risk. This defect is used to capture defects in concrete that have chlorides in excessive amounts.

The Chloride Concentration defect is only to be used on deck and slab elements and when properly tested. Test results shall be uploaded to HSI with the bridge inspection report.

Condition States

Condition State 1 indicates chloride concentration at level of rebar tested below the threshold for potential active corrosion.

Condition State 2 indicates chloride concentration at level of rebar tested equal to or greater than the threshold for potential active steel corrosion. No visual signs of active corrosion exist.

Condition State 3 indicates chloride concentration at level of rebar tested greater than the threshold for potential active steel corrosion. Testing methods (such as half-cell potential) have been used and have verified active steel corrosion.

Condition State 4 is not used for this defect. Other reinforced or prestressed concrete defects control the Condition State over chloride concentrations (elevated levels of chloride concentrations may be cause of controlling defects).

Chloride concentration thresholds at the level of top mat reinforcing are as follows:

- **2.0 lbs/CY:** High probability of active corrosion. Recommend utilizing Half-Cell testing method to survey deck for active regions of corrosion.

A.1.2.6 Precast Concrete Connections (8906) [ADD]

This defect is used to quantify the condition of the connections used between two precast bridge members joined together, such as shear keys or fasteners used in accelerated bridge construction projects.

Condition States

Condition State 1 indicates no deficiencies are noted.

Condition State 2 indicates that minor cracking at the joints may be present. The connection is still functioning as intended.

Condition State 3 indicates cracking, spalling, and/or other deficiencies indicating that the members are acting independently. No displacement of members is evident.

Condition State 4 indicates the connection is failing or has failed. The structural capacity has been affected. Exterior members may be separating from the adjacent members. Members may be acting individually under traffic loads. The condition warrants a structural analysis to determine the effect on strength or serviceability of the bridge; or a structural review has been completed and the defect impacts the strength and/or serviceability of the bridge.

A.1.3 Prestressed Concrete Defects

A.1.3.1 Delamination/Spalls/Patched Areas (1080)

Delamination is the result of a loss of bond between concrete and rebar due to expanding corrosion product of corroded reinforcement and can often lead to spalling of the concrete. Spalling is a depression in concrete where sections of concrete separate from aggregate or the reinforcement. Spalls result from the separation and removal of a portion of the surface concrete, revealing a fracture roughly parallel to the surface. Inspectors should use sketches showing the location and dimensions of delaminations and spalls. The depth of spalls should also be recorded.

Condition States

Condition State 1 indicates that there are no apparent delaminations, spalls, or patched areas.

Condition State 2 indicates that there are delaminated and/or spalled areas that are less than 1” deep or less than 6” in diameter or in either the length or width dimension. . Any patched areas are sound.

Condition State 3 indicates that there are spalled areas that are greater than 1” deep or greater than 6” in diameter, or in either the length or width dimension. Patched areas are unsound or showing distress. The delamination, spall, or patched area does not warrant a structural review.

Condition State 4 indicates that the depths of delaminations, spalls, or patched areas warrant a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.3.2 Exposed Prestressing (1100)

Exposed prestressing is often the result of impact damage from traffic. Exposed strands will be susceptible to accelerated corrosion and deterioration due to its exposure to the environment. Inspectors should record the area and location of the exposed prestressing and pay special attention to any partially, fully severed or damaged strands. Loss of prestress due to failure of prestressing steel may result in a serious defect manifested in loss of camber, beam sag, and/or wide structural cracks.

Condition States

Condition State 1 indicates that there are no exposed or signs of exposed prestressing strands in the bridge member.

Condition State 2 indicates that there is exposed prestressing with no section loss.

Condition State 3 indicates that there is exposed prestressing with section loss. The amount of deterioration does not warrant structural review.

Condition State 4 indicates that there is exposed prestressing and it warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.3.3 Cracking – Prestressed Concrete (1110)

Concrete cracks might indicate a significant loss in strength in prestressed concrete members. Loss of prestress due to failure of prestressing steel may result in a serious defects manifested in loss of camber, beam sag, and/or wide structural cracks. Inspectors should note all visible cracks, recording their type, width, length, and location.

Condition States

Condition State 1 indicates that there is cracking present less than 0.004” wide or any wider cracks are sealed.

Condition State 2 indicates that there is cracking present between 0.004” and 0.009” wide. Where efflorescence is present, it’s minor and there is no evidence of rust staining.

Condition State 3 indicates that there is cracking present greater than 0.009”. Where efflorescence is present, there is heavy build up or rust staining.

Condition State 4 indicates that there is cracking which warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.3.4 Abrasion/Wear (1190) (PSC/RC)

Vehicular traffic is a source of abrasion and wear on concrete decks. Abrasion occurs on concrete piles that are subjected to tidal flows. Mechanical wear of concrete members sometimes occurs due to movement of the fasteners against their holes when connections become loose.

Condition States

Condition State 1 indicates no abrasion.

Condition State 2 indicates abrasion has exposed coarse aggregate but the aggregate remains secure in the concrete.

Condition State 3 indicates coarse aggregate is loose or has popped out of the concrete matrix due to abrasion.

Condition State 4 indicates the condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.3.5 Chloride Concentration (8905) [ADD]

Chloride contamination is caused primarily by deicing agents. Chloride ions infiltrate concrete through surface cracks. Chloride ions break down the passivation layer around bare reinforcing steel allowing for corrosion to occur. Testing for chlorides involves drilling out samples and testing the content at certain depths. Once the chloride content reaches a certain limit at the reinforcing depth, bare steel reinforcing is at risk. This defect is used to capture defects in concrete that have chlorides in excessive amounts.

The Chloride Concentration defect is only to be used on deck and slab elements. Test results shall be uploaded to HSI with the bridge inspection report.

Condition States

Condition State 1 indicates chloride concentration at level of rebar tested below the threshold for potential active corrosion.

Condition State 2 indicates chloride concentration at level of rebar tested equal to or greater than the threshold for potential active steel corrosion. No visual signs of active corrosion exist.

Condition State 3 indicates chloride concentration at level of rebar tested greater than the threshold for potential active steel corrosion. Testing methods (such as half-cell potential) have been used and have verified active steel corrosion.

Condition State 4 is not used for this defect. Other reinforced or prestressed concrete defects control the Condition State over chloride concentrations (elevated levels of chloride concentrations may be cause of controlling defects).

Chloride concentration thresholds at the level of top mat reinforcing are as follows:

- **2.0 lbs/CY:** High probability of active corrosion. Recommend utilizing Half-Cell testing method to survey deck for active regions of corrosion.

A.1.3.6 Precast Concrete Connections (8906) [ADD]

This defect is used to quantify the condition of the connections used between two precast substructure members joined together, such as shear keys or fasteners used in accelerated bridge construction projects.

Condition States

Condition State 1 indicates no deficiencies are noted.

Condition State 2 indicates that minor cracking at the joints may be present. The connection is still functioning as intended.

Condition State 3 indicates cracking, spalling, and/or other deficiencies indicating that the members are acting independently. No displacement of members is evident.

Condition State 4 indicates the connection is failing or has failed. The structural capacity has been affected. Exterior members may be separating from the adjacent members. Members may be acting individually under traffic loads. The condition warrants a structural analysis to determine the effect on strength or serviceability of the bridge; or a structural review has been completed and the defect impacts the strength and/or serviceability of the bridge.

A.1.4 Timber Defects

A.1.4.1 Connection (1020)

Timber members are often connected with steel fasteners including nails, screws, bolts, and tension rods and are thus subject to the same deficiencies as steel members. Loose or failed connections can be due to pull out or pull through of fasteners, shrinkage of the wood, crushing of the wood around the fastener, or from repetitive impact loading (working) of the connection. Loose or failed connections can reduce the bridge's load-carrying capacity.

Condition States

Condition State 1 indicates that the connection is in place and is functioning as intended

Condition State 2 indicates that there are loose fasteners and/or pack rust without distortion of the hardware but no loss of section. The connection is in place and functioning as intended despite these defects.

Condition State 3 indicates that there are missing bolts, nails, screws, or other fasteners, and/or pack rust with distortion of the fasteners. Although these defects are present, a structural review is not warranted.

Condition State 4 indicates that there are missing bolts, rivets, nails, broken welds, screws, or other fasteners, and/or pack rust with distortion warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.4.2 Decay /Section Loss/Abrasion/Wear (1140)

Natural decay of timber can occur when there are favorable combinations of oxygen, temperature, food, and moisture. This occurs at locations such as connections, ground lines, and splash zones. Either of these defects can result in section loss of the member. Inspectors should use hammer sounding and picks/awls to determine the extent of decay and inspect damage.

Condition States

Condition State 1 indicates that no decay or section loss is present in the bridge member.

Condition State 2 indicates that decay or section loss affects less than 10% of the member section

Condition State 3 indicates that decay or section loss affects greater than 10% of the member section but does not warrant structural review.

Condition State 4 indicates that amount of decay or section loss warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a

structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.4.3 Checks/Shakes/Cracks/ Splits/ Delamination (1150)

Natural defects are those inherent in the material and are a result of the natural behavior of timber. Checks are the separation of fibers occurring perpendicular to growth rings and parallel to the grain. Shakes are the separation of fibers parallel to the grain between growth rings. Splits are checks which extend completely through the member. Delaminations occur in glued-laminated members when the layers separate due to failure within the adhesive, or the bond between the adhesive and the laminate. They provide openings for decay to begin and may cause a reduction in strength. Each timber member has a certain ultimate load capacity. If this load capacity is exceeded, the member will fail, typically through cracking. Cracking occurs against the grain and should not be confused with natural occurring defects. Failure modes include horizontal shear failure, bending or flexural failure, and crushing.

Condition States

Condition State 1 Checks/Cracks penetrate <5% of member thickness. Member does not have Splits/Shakes/Delamination.

Condition State 2 Checks/Cracks penetrate 5%-50% of member thickness and not in tension zone. Member has Splits/Shakes with length less than member depth. Larger Cracks/Splits/Shakes have been arrested with effective repair.

Condition State 3 Checks/Cracks penetrate >50% of member thickness or >5% in tension zone. Member has Splits/Shakes with length greater than member depth and have not been arrested. Defects do not require a structural review.

Condition State 4 The condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.4.4 Abrasion/Wear - Timber (1180)

Vehicular traffic is the main source of abrasion on timber decks. Abrasion occurs on timber piles that are subjected to flowing water or tidal flows. Mechanical wear of timber members sometimes occurs due to movement of the fasteners against their holes when connections become loose.

Condition States

Condition State 1 indicates that no wear or abrasion is present.

Condition State 2 indicates that wear and abrasion has caused section loss less than 10% of the member thickness.

Condition State 3 indicates that wear and abrasion has caused section loss greater than 10% of the member thickness but strength or serviceability of the bridge is not affected and does not require a structural review.

Condition State 4 indicates that damage is greater than 10% of the member thickness and the condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.4.5 Distortion (1900)

This defect is used to report distortion from the original line or grade of the element. It is used to capture all distortion regardless of cause. Glulam and other built up timber members are connected with glue and are subject to failure by improper application, which can lead to distortion of the timber member.

Condition States

Condition State 1 indicates no distortion.

Condition State 2 indicates there is distortion that does not require mitigation or previously reported distortion which has been mitigated.

Condition State 3 indicates there is distortion present which requires mitigation but does not warrant a structural review.

Condition State 4 indicates there is distortion present which requires mitigation and warrant a structural review to determine the strength or serviceability of the element or a structural review has been completed and the distortion impacts the strength or serviceability of the bridge.

A.1.5 Masonry Defects

A.1.5.1 Mortar Breakdown (1610)

Inspection of mortar is similar to that of concrete. Inspectors should carefully inspect the joints for cracks, vegetation, water seepage, weathering, spalls and split blocks. Cracks or openings allow water, debris, and chemicals into the mortar which will accelerate the deterioration of the mortar and stones. Inspectors should detail the linear length of mortar deterioration and uses sketches where applicable.

Condition States

Condition State 1 indicates that there is no mortar breakdown on the bridge element.

Condition State 2 indicates that cracking or voids in the mortar cover less than 10% of the joints.

Condition State 3 indicates that cracking or voids in the mortar cover more than 10% of the joints but does not warrant a structural review.

Condition State 4 indicates that cracking or voids in the mortar cover more than 10% and warrants a structural review to determine the effect on strength or serviceability of the bridge element, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge

A.1.5.2 Spall/Split/Patched Area (1620)

Cracking of masonry stones occurs when part of the internal structure of the stone is fractured, allowing water/chemicals to seep into the stone interior. Splitting of masonry stones occurs when seams or cracks open up in stones, breaking them into smaller pieces. Patched areas are locations with previous defects that have been repaired. Inspectors should determine if the patch is functioning properly and sound, if the patch is showing any signs of distress, or if the serviceability of the bridge is affected and if another repair method needs to be implemented.

Condition States

Condition State 1 indicates that there are no splits or spalls on the bridge member.

Condition State 2 indicates that the block or stone has split or spalled with no shifting. Any patched areas are sound.

Condition State 3 indicates that the block or stone has split or spalled with shifting but does not warrant a structural review. Patched areas are not sound.

Condition State 4 indicates that the condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.5.3 Masonry or Panel Displacement (1640)

Displacement is a form of localized settlement or misalignment. This defect, which typically affects masonry structures due to the weight of the material, can result in differential settlement. Superstructure and substructure units should be checked for any new or unusual cracking. General alignment of stones along the element should remain constant. Missing stones allow moisture and vegetation to enter masonry units and develop or accelerate other forms of deterioration.

Condition States

Condition State 1 indicates that no masonry or panel displacement exists in the bridge member.

Condition State 2 indicates that blocks, panels or stones have shifted slightly out of alignment.

Condition State 3 indicates that blocks, panels or stones have shifted significantly out of alignment and/or blocks or stones are missing but does not warrant a structural review.

Condition State 4 indicates that masonry displacement warrants a structural review to determine the effect on strength or serviceability of the bridge element or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.6 Other Materials Defects

A.1.6.1 Corrosion (1000)

Section loss of a material is caused by corrosion heavy or severe enough to physically measure a reduction in member thickness.

The inspector should look for areas of laminate rust on the material surface or areas of heavy blistering paint (caused by a build-up of rust under the paint surface). Common locations are horizontal surfaces, areas subject to salt spray from roads or areas near water surfaces. The most effective tools used to remove the laminate rust are chipping hammers and paint scrapers. Severe corrosion will usually consist of fairly loose, brown laminate rust, and a hard, tightly adhered black corrosion product on the base metal surface. A chipping hammer is usually required to remove this corrosion. Safety glasses should always be worn when using a chipping hammer.

Once the corrosion has been removed, the amount of remaining section should be measured and recorded. Methods to measure the remaining section include the use of a caliper, a micrometer or ruler with a straight edge spanning the depression to indicate the original material surface. The surface area affected by the corrosion should also be measured.

Some corrosion is so severe that holes are created through the material. This situation should be recorded as “through thickness section loss” rather than “100 percent section loss” since the latter term suggests that the entire member or element has corroded away.

Condition States

Condition State 1 indicates no corrosion exists or it has been repaired and painted over corrosion that has been painted over will not prevent further corrosion from occurring and should not be considered a Condition State 1.

Condition State 2 indicates that corrosion has initiated. Freckled rust may be present.

Condition State 3 indicates that section loss or pack rust exists but does not warrant a structural review.

Condition State 4 indicates that excessive section loss exists, and that structural analysis results indicate member capacity or serviceability is compromised.

A.1.6.2 Cracking (1010)

Cracking addresses fatigue cracks of a material, regardless if they occur within a NSTM or non-NSTM bridge. Cracking in substructure elements will most often occur in pier caps or cross girders. Any cracks not previously detected should be evaluated to determine the potential for fracture.

Inspectors should be especially thorough at details which are prone to developing fatigue cracking. Refer to Appendix D for a list of fatigue details. Detection of these cracks will most often occur during In-Depth or NSTM Inspections but can still be found during Routine Inspections as well. Any crack defects classified as a Condition State 3 or worse should be immediately reported to the Inspection Program Manager. Serious consideration should be given to closing the bridge until the extent and cause of cracking is determined and appropriate remedial action is taken.

Condition States

Condition State 1 indicates that no cracking exists in the member.

Condition State 2 indicates that a crack exists, but it has either been arrested by drilling holes at the tips or it has been repaired with bolted splices or welded splices. Shallow or short cracks may also be ground out. Unrepaired, self-arrested, or cracks not arrested with drilled holes may also fall into this Condition State, but they are located in a non-critical or redundant member.

Condition State 3 indicates that a crack which has not been arrested or repaired exists in a primary load-carrying member, including newly-discovered crack or existing cracks that are still growing between inspection cycles. Cracks of this nature suggest it is fatigue related, stable crack growth.

Condition State 4 indicates that a very large crack exists in any primary load-carrying member. Newly-discovered cracks are greater than 3 inches in length, or existing cracks have grown more than 6 inches within the last 24 months. Cracks of this nature suggest they developed in an uncontrolled brittle fashion.

A.1.6.3 Connection (1020)

Connections can be constructed from two or more members or elements that may be bolted, riveted, or welded together. Connection deficiencies include any deterioration or damage of fasteners, such as rivets, bolts, or welds and connection plates (gusset plates). Any suspected loose or deteriorated fasteners should be struck with a hammer to determine soundness. Welded connections should be visually inspected for fatigue cracks developing from weld imperfections or overstress. Connections are also often subjected to distortion due to pack rust (corrosion occurring between two pieces of lapped steel). The amount of separation caused by pack rust and any distortion to the steel should be measured and recorded.

Condition States

Condition State 1 indicates that the connection is in place and is functioning as intended.

Condition State 2 indicates that there are loose fasteners and/or pack rust without distortion. The connection is in place and functioning as intended despite these defects.

Condition State 3 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. Although these defects are present, the structural capacity of the structure is not reduced.

Condition State 4 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. These defects reduce the structural capacity of the member assembly.

A.1.6.4 Delamination/Spalls/Patched Areas (1080)

Delamination is the result of a loss of bond between concrete and rebar due to expanding corrosion product of corroded reinforcement and can often lead to spalling of the concrete. Spalling is a depression in concrete where sections of concrete separate from aggregate or the reinforcement. Spalls result from the separation and removal of a portion of the surface concrete, revealing a fracture roughly parallel to the surface. Inspectors should use sketches showing the location and dimensions of delaminations and spalls. The depth of spalls should also be recorded. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates that there are no apparent delaminations, spalls, or patched areas.

Condition State 2 indicates that there are delaminated and/or spalled areas that are less than 1” deep or less than 6” in diameter or in either the length or width dimension. . Any patched areas are sound.

Condition State 3 indicates that there are spalled areas that are greater than 1” deep or greater than 6” in diameter, or in either the length or width dimension. Patched areas are unsound or showing distress. The delamination, spall, or patched area does not warrant a structural review.

Condition State 4 indicates that the depths of delaminations, spalls, or patched areas warrant a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.1.6.5 Deterioration (1220)

Deterioration can be considered the material breakdown of the element. This may include debonding, fraying or discoloration in FRP or similar materials.

Condition States

Condition State 1 indicates that there is no deterioration occurring.

Condition State 2 indicates that the material has initiated breakdown or deterioration.

Condition State 3 indicates that there is significant deterioration or breakdown, but it does not warrant a structural review.

Condition State 4 indicates that the deterioration or breakdown has become severe enough to warrant a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the deterioration or breakdown impact strength or serviceability of the element or bridge.

A.1.6.6 Distortion (1900)

This defect is used to report distortion from the original line or grade of the element. It is used to capture all distortion regardless of cause.

Condition States

Condition State 1 indicates no distortion.

Condition State 2 indicates there is distortion that does not require mitigation or previously reported distortion which has been mitigated.

Condition State 3 indicates there is distortion present which requires mitigation but does not warrant a structural review.

Condition State 4 indicates there is distortion present which requires mitigation and warrants a structural review to determine the strength or serviceability of the element or a structural review has been completed and the distortion impacts the strength or serviceability of the bridge.

Section A.2: Structural Defects

A.2.1 Substructure Defects

A.2.1.1 Settlement (4000)

Settlement refers to the vertical, rotational, and lateral movement of substructure elements caused by scour, soil failures or foundation failures. Any particular element may experience a combination of these movements as well.

Inspection

Settlement inspection for substructures should include the following items:

- Looking for expansion joints which have opened up excessively at abutments and have either closed completely or opened up above the piers. A closed expansion joint above a pier suggests that the pier has settled. These are all signs of differential pier settlement.
- Checking/measuring the clearance between girder ends and abutment backwall.
- Checking/measuring the clearance between the girder ends on bridges with multiple simple spans.
- Checking/measuring the alignment of expansion bearings relative to the masonry plate and backwall. Excessive superstructure expansion in hot weather or contraction in cold weather may actually be a sign of substructure rotation or sliding.
- Looking for transverse cracks in columns or shafts.
- Sighting down the superstructure parapet to look for unusual dips or sweeps.
- Checking for tipping/rotation by using a plumb bob and laterally sighting the element from a distance.
- Looking for bridge elements that do not line up with one another, such as wingwalls that have shifted laterally relative to the abutment at an expansion joint.
- Investigating existing cracks for continued widening since previous inspections. Measurements and dates taken should be taken and recorded, and written with keel directly on the element.
- Looking for heaving in front of abutments or wingwalls and settlement of approaches.
- Looking for embankment erosion in front of abutments and wingwalls.

Condition States

Condition State 1 indicates that no settlement has occurred.

Condition State 2 indicates that settlement has occurred and has been arrested with effective countermeasures.

Condition State 3 indicates that minor settlement has occurred. Countermeasures have been taken but movement is still evident. Currently, the settlement does not warrant a structural review.

Condition State 4 indicates that the settlement warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.2.1.2 Scour (6000)

Scour of a streambed is caused by the erosive effects of water flow. High floodwater velocities can lift soil and rock particles from the streambed and transport them downstream. The effect is even greater when obstructions within the stream flow, such as a bridge pier, abutment or wingwall, create localized turbulence (usually on the upstream face). Turbulent waters often scour deep holes adjacent to a bridge pier, possibly undermining the footing. Streambed voids created during high flow are often filled back in with upstream sediment after high flows; masking the fact that scour took place.

Many bridge failures have been caused solely by scour. Under extreme circumstances, scour can cause foundation undermining (soil erosion on the underside of a footing) and tipping of a substructure unit. Scouring underneath of a spread footing is also dangerous because it removes soil from underneath the footing's toe, the area of soil most highly stressed due to lateral loads. The existence of scour does not necessarily mean settlement or tipping of the substructure has occurred.

Inspection

Inspection for scour is one of the most important bridge inspection tasks since scour can fail an otherwise sound structure. Unfortunately, scour inspection can also be one of the most difficult tasks to perform. For purposes of this section, it is assumed that probing with a rod (usually a surveyor's range pole) can be performed from a boat or by wading. When this is not feasible, underwater inspection by divers or remote sensing using ground-probing radar is required.

Scour inspection using probing should include the following items:

- Using a probing rod around the substructure to check for scour. Signs of scour include an exposed top or side of the footing. It is rare that the original design called for the footing to be exposed. Also, excessively loose or unconsolidated sediment in the streambed (as compared to the sediment away from the substructure) suggests an area of scour during peak flows.

- Sounding with a weighted line around the substructure to check for scour. This will normally be done from a boat when water depth is too great for wading. This method will not easily detect undermining of the footing.

- Recording the length and depth of scour holes. Depth from the bottom of the hole to the water surface should be recorded. Tying the current water elevation to the bridge elevation will require a second measurement from the water surface to some fixed point on the bridge.

- Using a probing rod around the substructure to check for undermining. The bottom of a footing can be found by scraping the rod tip along the footing side until the bottom corner is found. The length and depth of the scour hole should be recorded.

- Careful probing around substructures with debris accumulation, as debris can increase water flow velocity and the potential for scour. Remove this debris during the inspection if possible. Debris accumulation can also be a safety hazard for the inspectors so care must be taken when working in these situations.

Condition States

Condition State 1 indicates no scour has occurred.

Condition State 2 indicates that scour has exposed the top of the footing. No undermining is evident. Counter measures are in place and functioning properly. Minor scour may be present around pile bents. No significant loss of channel material has occurred when compared to previous measurements.

Condition State 3 indicates that scour has exposed vertical face(s) of the footing. No undermining of spread footing or minor undermining of pile supported footing. There is moderate scour around pile bents. Measurements indicate active channel movement. The scour currently does not warrant a structural review.

Condition State 4 indicates that the condition of the scour condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

Section A.3: Element Specific Defects

A.3.1 Wingwall Defects

A.3.1.1 Wall Movement (8902) [ADD]

This defect is for wall and integral wingwall elements and is used to quantify the extent and severity of wall or wingwall movement. Wall or Wingwall movement can be induced through settlement of the wall or wingwall foundation, excessive stress from the soil back-fill, or improper construction of the wall or wingwall. The inspector should note the angle that the wall or wingwall is tipped to see if the tipping is advancing.

Condition States

Condition State 1 indicates that the wall or wingwall has not moved from its original position.

Condition State 2 indicates that the wall or wingwall has had some differential movement occur. Wall may be strapped to prevent further movement.

Condition State 3 indicates that the wall or wingwall rotation, sliding, or settlement is occurring. Sloughing of the retained material behind the wall is evident.

Condition State 4 indicates that the wall or wingwall has failed and no longer retains the material behind the wall.

A.3.1.2 Wall Deterioration (8903) [ADD]

This defect is for wall and integral wingwalls and is used to quantify the extent and severity of wall or wingwall material deterioration. Since wingwalls can be made of various materials, the inspector will refer to the specific material defects associated with the wingwall material.

Condition States

Condition State 1 indicates that the wall or wingwall has no deterioration.

Condition State 2 indicates that the wall or wingwall material has deterioration described in the applicable CS 2 material defects for section loss and wall integrity.

Condition State 3 indicates that the wall or wingwall material has deterioration described in the applicable CS 3 material defects for section loss and wall integrity.

Condition State 4 indicates that the wall or wingwall material has deterioration or section loss that has caused the wing to fail and no longer retain fill material.

A.3.2 Bearing Defects

A.3.2.1 Corrosion (1000)

Any bearing which is made from or has steel or other metal parts is susceptible to corrosion. Bearings are typically located near bridge joints, which increase their exposure to water, debris, and other roadway chemicals, which can accelerate the process of corrosion. Excessive corrosion can cause bearings to bind up or freeze. Inspectors should check all bearing elements for any pitting, section loss, or deterioration.

Condition States

Condition State 1 indicates no corrosion.

Condition State 2 indicates that corrosion of the steel has initiated and/or freckle rust is present.

Condition State 3 indicates that section loss is evident or pack rust is present but it does not warrant a structural review.

Condition State 4 indicates that the section loss and/or pack rust warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.3.2.2 Connection (1020)

Bearing connections to other bridge elements can be bolted, riveted, or welded together. Connection deficiencies include any deterioration or damage of fasteners, such as rivets, bolts, or welds. Any suspected loose or deteriorated fasteners should be struck with a hammer to determine soundness. Welded connections should be visually inspected for cracks developing from weld imperfections or overstress. Connections are also often subject to deterioration from pack rust (corrosion occurring between two pieces of lapped steel). The amount of separation caused by pack rust should be measured and recorded.

Condition States

Condition State 1 indicates that the connection is in place and is functioning as intended.

Condition State 2 indicates that there are loose fasteners and/or pack rust without distortion present. The connection is in place and functioning as intended despite these defects.

Condition State 3 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. Although these defects are present, a structural review is not warranted.

Condition State 4 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. These defects warrant a structural review to determine

the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.3.2.3 Movement (2210)

This defect is used to report the movement of bridge bearing elements. The ability of free movement is essential for allowing bridge bearings to function correctly. Inspectors should look for frozen bearings and any potential debris or defect that would prevent a bearing from using the full area of movement for which it was designed.

Condition States

Condition State 1 indicates the bearing is free to move.

Condition State 2 indicates there is minor restriction to the bearings movement.

Condition State 3 indicates the bearing is restricted from movement but the situation does not warrant a structural review.

Condition State 4 indicates the bearing is restricted from movement and the situation warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.3.2.4 Alignment (2220)

This defect is used to report the alignment of bridge bearing elements. Inspectors should compare bearing alignment to other bearings at the same location and other locations on the bridge. Bearing alignment should match environmental conditions at the time of inspection (i.e. expansion when hot, contraction when cold). Bearing assemblies must also be checked to ensure they are aligned in the direction of intended movement, that is they are not skewed to the direction of a girder.

Condition States

Condition State 1 indicates the lateral and vertical alignment is as expected for the temperature conditions.

Condition State 2 indicates the lateral or vertical alignment is inconsistent with the temperature conditions but the alignment is tolerable.

Condition State 3 indicates the lateral or vertical alignment is approaching the limits of bearings ability to function but does not warrant a structural review.

Condition State 4 indicates the bearings alignment warrants a structural review or that a structural review has been completed and the alignment impacts the strength or serviceability of the bridge.

A.3.2.5 Bulging, Splitting, or Tearing (2230)

This defect is used to report bulging, splitting, or tearing of elastomeric bearing elements. This defect is also applicable to the disk material of disk bearings. Excessive amounts of bulging, wearing, and tearing can dramatically decrease the life of a bearing and accelerate the deterioration caused by other defects.

Condition States

Condition State 1 indicates there is no bulging, splitting, or tearing in the bearing.

Condition State 2 indicates bulging is less than 15% of the thickness and there is no splitting or tearing.

Condition State 3 indicates bulging is 15% or more of the thickness or any splitting or tearing. The bearing's surfaces are no longer parallel but does not warrant a structural review.

Condition State 4 indicates bulging is 15% or more of the thickness or any splitting or tearing and requires a structural review to determine the effect of the defect on the strength or serviceability of the element or bridge or that a structural review has been completed and the bulging, splitting, or tearing impacts the strength or serviceability of the bridge.

A.3.2.6 Loss of Bearing Area (2240)

This defect is used to report the loss of bearing area for bridge bearing elements. The loss of bearing area can affect the alignment, movement, and structural safety of the bridge. Issues developing from bearing movement can cause structural defects in other elements (e.g. a girder subject to settlement at one end).

Condition States

Condition State 1 indicates there is no loss of bearing area.

Condition State 2 indicates the loss of area is less than 10% of the total bearing area.

Condition State 3 indicates the loss of area is 10% or more of the total bearing area but does not warrant a structural review.

Condition State 4 indicates the loss of area is 10% or more of the total bearing area and requires a structural review to determine the effect of the defect on the strength or serviceability of the element or bridge or that a structural review has been completed and the loss of area impacts the strength or serviceability of the bridge.

A.3.3 Joint Defects

A.3.3.1 Leakage/Seal Adhesion/ Damage/Cracking (2310)

This defect is used to report leakage through or around sealed bridge joints. Water leakage through a joint can lead to deterioration and section loss of the surrounding elements as well as the elements below the joints. Leakage can also be detrimental to the substructure if unexpected water flow is allowed to enter the soil around the foundation.

This defect is also used to report loss of adhesion in sealed bridge joints. Sealed bridge joints will typically use a type of adhesive chemical (contact cement, joint adhesive, glue, etc.) to adhere the seal to the adjacent bridge deck or steel extrusion. Over time this glue could lose its adhesive properties and start to degrade which will allow the seal to pull away and permit leakage through the joint.

Seal damage is used to report damage to the seal gland in bridge joint seals. Since the seal is continually subject to the traffic that passes over the bridge deck there can be damage caused by incidental contact (snow plows, large vehicles, etc.) or debris impaction. This damage can lead to leakage through the joint and should be closely monitored by the inspector.

Seal cracking is used to report cracking in the seal gland in bridge joint seals. Plastic or polymer material tends to lose its elasticity after exposure to UV rays from the sun. As the bridge continues to expand and contract the seal will begin to degrade which can lead to cracking. Once cracking has initiated it can quickly propagate if left in a poor environment. The inspector should note any cracking of the seal as it can lead to leakage if not repaired or retrofitted.

Condition States

Condition State 1 indicates that the joint is fully adhered.

Condition State 2 indicates there is minimal leakage through the joint. Minimal leakage is defined as minor or occasional leaking or water from localized areas of a joint. The seal is adhered for more than 50% of the joint height. There is seal abrasion but without through punctures or cracks.

Condition State 3 indicates there is moderate leakage through the joint. Moderate leakage is defined as multiple areas of leaking and passage of debris through the joint. The seal is adhered to 50% or less of the joint height but there is still some adhesion. The joint is punctured, cracked, ripped, or partially pulled out.

Condition State 4 indicates the joint has failed and there is a free flow of water and debris through the joint. There is a complete loss of seal adhesion or the seal is punctured, cracked, or torn completely through, pulled out, or missing.

A.3.3.2 Debris Impaction (2350)

Dirt, gravel and debris lodged in the joint may prevent normal expansion and contraction, causing cracking in the deck and backwall, and overstress in the bearings. In addition, as dirt and debris are continually driven into the joint, the joint material may eventually rupture or pull out.

Condition States

Condition State 1 indicates there is no debris in the joint. A shallow covering of loose debris may be present but not affecting the performance of the joint.

Condition State 2 indicates the joint is partially filled with hard-packed material but still allows for free movement and the joint can perform as expected.

Condition State 3 indicates the joint is completely filled with debris, which has an effect on the movement and performance of the joint.

Condition State 4 indicates the joint is completely filled with debris, which has completely prevented joint movement.

A.3.3.3 Adjacent Deck or Header/Metal Deterioration or Damage (2360)

Many joints are connected to the deck utilizing some type of anchorage. The inspector shall examine deck areas adjacent to deck joints for material deterioration such as section loss, spalls, delaminations, and vehicular/snow plow damage. Deterioration of the deck in these areas may be an indication of problems with the anchorage. Metal deterioration or damage is used to report steel damage or deterioration in the bridge joint (such as a finger joint, sliding plate joint, or steel extrusions of a seal joint). This will include corrosion, section loss, and cracking that have occurred through other means besides impact damage.

Condition States

Condition State 1 indicates the adjacent deck or header is sound. No spalling, delamination, metal deterioration or damage, or unsound patches are present.

Condition State 2 indicates there is edge delamination or spalls less than 1 in. deep or less than 6 in. in diameter. There are no exposed rebar in any present spalls and any patched areas are sound. Freckled rust may be present but the metal has no cracks, or impact damage. The connection may be loose but is still functioning as intended.

Condition State 3 indicates there is edge delamination with spalls greater than 1 in. deep or 6 in. in diameter. Spalls may have exposed rebar. Delamination or unsound patched areas or delaminations are present that make the joint loose. There is section loss, missing or broken fasteners, cracking of the metal, or impact damage but the joint is still functioning.

Condition State 4 indicates there are spalls, delaminations, unsound patched areas, or loose anchors which prevent the joint from functioning as intended. There is also metal

cracking, section loss, damage, or connection failure that prevents the joint from functioning as intended.

A.3.4 Steel Protective Coating Defect

A.3.4.1 Oxide Film Degradation Color/Texture Adherence – Weathering Steel (3430)

This defect is the only defect used to report oxide film degradation of texture in Weathering Steel. Weathering steel can be an effective steel protective coating if allowed to corrode as designed. If weathering steel is placed in an environment with substantial moisture then the steel will continue to corrode and lead to section loss of the steel. The inspector should note the color and pitting or section of the steel as these can be an indication of the health of the element.

Condition States

Condition State 1 indicates oxide film with a yellow-orange or light brown for early development, or a chocolate-brown to purple-brown for fully developed films. The oxide film is tightly adhered and capable of withstanding hammering or vigorous wire brushing.

Condition State 2 indicates an oxide film which has developed a granular texture.

Condition State 3 indicates an oxide film which has developed small flakes that are less than ½ in. diameter.

Condition State 4 indicates an oxide film with dark black color. The film has large flakes which are ½" in. or larger in diameter. The film may have developed laminar sheets or nodules.

A.3.4.2 Effectiveness (3440)

This defect is the only defect to be used on all steel protective coatings other than Weathering Steel. The ability for the protective coatings to protect the steel element it is placed over is its effectiveness. A protective coating can lose effectiveness through wear, deterioration, and leakage.

Condition States

Condition State 1 indicates a protective coating which is fully effective. Coating system may have minor fading, chalking, dulling, but there is no active corrosion and primer coat is intact.

Condition State 2 indicates a protective coating which is substantially effective. Coating system may have moderate deterioration such as chalking, peeling, blistering or cracking to finish coat. Surface/freckle rust or staining may be present, but there is no pack rust.

Condition State 3 indicates a protective coating which is limited effectiveness. Coating system has extensive deterioration to finish and primer coats. There is loss of pigment, peeling, bubbling, or cracking to finish and primer coats. Surface rust is prevalent. Pack rust may be present.

Condition State 4 indicates a protective coating which has failed or is non-existent and provides no protection of the underlying metal. There may be heavy corrosion and pack rust.

A.3.5 Wearing Surface Defects

A.3.5.1 Debonding/Spall/Patched Area/Pothole – Wearing Surface (3210)

Debonding is the result of lost bond between the overlay surface and the bridge deck below. This defect can occur in any type of overlay material. Once debonded, cyclical loading cycles, such as traffic, freeze/thaw, expansion and contraction will eventually deteriorate the delamination enough to failure. This can often lead to spalling of the wearing material. Spalls result from the separation and removal of a portion of the surface material, revealing a fracture roughly parallel to the surface. When spalls occur on the riding surface they are considered potholes. Inspectors should use sketches showing the location and dimensions of delaminations and spalls. The depth of spalls should also be recorded. When a spall exposes steel reinforcing, the wearing surface in that area is no longer considered effective and shall be coded Condition State 4.

Condition States

Condition State 1 indicates that no debonding, spalls, patched areas, or potholes exist.

Condition State 2 indicates that the wearing surface is debonding, has spalls less than 1 inch deep or less than 6 inches in diameter, has sound patched areas, or has partial depth potholes.

Condition State 3 indicates that the wearing surface is debonded, has spalls greater than 1 inch deep or more than 6 inches in diameter, has unsound patched areas that may be showing distress, or has full depth potholes.

Condition State 4 indicates that debonding, spalls, patched areas, or potholes have made the wearing surface no longer effective.

A.3.5.2 Crack – Wearing Surface (3320)

A crack is a linear fracture in the overlay material. Cracking can extend partially or completely through the wearing surface. There are two types of cracking; structural and nonstructural. Structural cracks are due to the loads imposed on the wearing surface and nonstructural cracks are due to temperature, shrinkage, and mass concrete.

Condition States

Condition State 1 indicates that cracks in the wearing surface are less than 0.012 inches or there are sealed cracks.

Condition State 2 indicates that cracks in the wearing surface are between 0.012 and 0.05 inches.

Condition State 3 indicates that cracks in the wearing surface are greater than 0.05 inches.

Condition State 4 indicates that cracking have made the wearing surface no longer effective.

A.3.5.3 Abrasion, Wear, Rutting, or Loss of Friction – Wearing Surface (8911) [ADD]

This defect captures how well the wearing surface is handling vehicular traffic. Over time, the wearing surface will undergo several changes in its ability to handle traffic. Abrasion and wear are similar in that the wearing surface is starting to degrade and is not of its original thickness, or the surface material is starting to be removed. Rutting occurs in the wheel paths of the travelled way. This can be caused by either the removal of the material in the wheel paths, or due to the wearing surface material not being able to withstand the traffic. Eventually the wearing surface material will begin to compress/shift to the adjacent area that is not carrying traffic. Typically, rutting will also have cracking associated with it due to the wearing surface material movement. Loss of friction can be created by the removal of the surface binder, which exposes the smooth aggregate. When vehicles ride on the smooth aggregate in adverse weather conditions, the coefficient of friction is reduced between the vehicle tire and the wearing surface.

Condition States

Condition State 1 indicates that there is no abrasion, wear, rutting, or loss of friction.

Condition State 2 indicates that there is minimal loss of surface material in wheel paths. There is no significant loss of friction. The asphalt overlay is exhibiting minor isolated rutting.

Condition State 3 indicates that there is a loss of surface material which has become prevalent in the wheel paths. The loss of friction is noticeable and/or the asphalt overlay has moderate to severe isolated rutting.

Condition State 4 indicates that there is widespread loss of surface material throughout the overlay. Exposed reinforcing may be present with or without corrosion or section loss. The loss of friction is prevalent and potentially dangerous in adverse weather conditions. The asphalt overlay has significant rutting throughout the length of the structure.

A.3.6 Concrete Reinforcing Steel Protective System Defects

A.3.6.1 Effectiveness – Protective System (e.g. cathodic) (3600)

This defect is used to report the effectiveness of internal concrete protective systems (cathodic protection). The ability for the protective coating to protect the concrete element it is placed within is its effectiveness.

Condition States

Condition State 1 indicates that the coating is fully effective.

Condition State 2 indicates that the coating is still substantially effective.

Condition State 3 indicates that the coating has limited effectiveness.

Condition State 4 indicates that the protective system has failed or is no longer effective.

A.3.7 Concrete Protective Coating Defects

A.3.7.1 Effectiveness - Concrete (3540)

This defect is used to report the effectiveness of external concrete protective systems. The ability for the protective coating to protect the concrete element it is placed over is its effectiveness. A protective coating can lose effectiveness through wear, deterioration, and leakage.

Condition States

Condition State 1 indicates that the coating is fully effective.

Condition State 2 indicates that the coating is still substantially effective.

Condition State 3 indicates that the coating has limited effectiveness.

Condition State 4 indicates that the protective system has failed or is no longer effective.

A.3.8 Strengthening/Repair System Defects

A.3.8.1 Deterioration (1220)

Deterioration can be considered the material breakdown of the element. This may include cracking, corrosion, section loss, discoloration, debonding, etc. It encompasses the deterioration defects of all materials. This defect is only to be used for FRP and Culvert Liner.

Condition States

Condition State 1 indicates that there is no deterioration occurring.

Condition State 2 indicates that the FRP or culvert liner has initiated breakdown or deterioration.

Condition State 3 indicates that there is significant deterioration or breakdown, but it does not warrant a structural review.

Condition State 4 indicates that the deterioration or breakdown has become severe enough to warrant a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the deterioration or breakdown impact strength or serviceability of the element or bridge.

A.3.8.2 Distortion (1900)

This defect is used to report distortion from the original line or grade of the element. It is used to capture all distortion regardless of cause. This defect is used for FRP and Culvert liner only.

Condition States

Condition State 1 indicates no distortion.

Condition State 2 indicates there is distortion that does not require mitigation or previously reported distortion which has been mitigated.

Condition State 3 indicates there is distortion present which requires mitigation but does not warrant a structural review.

Condition State 4 indicates there is distortion present which requires mitigation and warrants a structural review to determine the strength or serviceability of the element or a structural review has been completed and the distortion impacts the strength or serviceability of the bridge.

A.3.8.3 Delamination/Spalls/Patched Areas (1080)

Delamination is the result of a loss of bond between concrete and rebar due to expanding corrosion product of corroded reinforcement and can often lead to spalling of the concrete. Spalling is a depression in concrete where sections of concrete separate from aggregate or the reinforcement. Spalls result from the separation and removal of a portion of the surface concrete, revealing a fracture roughly parallel to the surface. Inspectors should use sketches showing the location and dimensions of delaminations and spalls. The depth of spalls should also be recorded. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates that there are no apparent delaminations, spalls, or patched areas.

Condition State 2 indicates that there are delaminated and/or spalled areas that are less than 1” deep or less than 6” in diameter or in either the length or width dimension. . Any patched areas are sound.

Condition State 3 indicates that there are spalled areas that are greater than 1” deep or greater than 6” in diameter, or in either the length or width dimension. Patched areas are unsound or showing distress. The delamination, spall, or patched area does not warrant a structural review.

Condition State 4 indicates that the depths of delaminations, spalls, or patched areas warrant a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.3.8.4 Exposed Rebar (1090)

Exposed rebar can be the result of spalls, delaminations, or damage. Exposed rebar will be susceptible to accelerated corrosion and deterioration due to its exposure to the environment. Inspectors should record the area and location of corrosion to reinforcement and record any measureable section loss. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates that there is no exposed rebar in the bridge member.

Condition State 2 indicates that rebar is exposed and surface corrosion may be present but without section loss.

Condition State 3 indicates that rebar is exposed and surface corrosion is present with measurable section loss but does not warrant a structural review. Caliper/thickness gauge measurements should be used to determine the amount of section loss.

Condition State 4 indicates that rebar is exposed and surface corrosion is present with measurable section loss and warrants a structural review to determine the effect on

strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge. Caliper/thickness gauge measurements should be used to determine the amount of section loss

A.3.8.5 Cracking – Reinforced Concrete (1130)

Cracking is common in reinforced concrete due to concrete's lack of strength in tension. Cracking can be a result of structural forces, temperature, or other forces. Common locations for structural cracking include negative moments over a support, near supports from shear stress, and the bottom of elements near mid-span from positive moments. Temperature cracking is caused by thermal expansion and contraction. Shrinkage cracks are due to the shrinkage of concrete caused by the curing process. It can cause either pattern (map) cracking or random cracks. Mass concrete cracks occur due to thermal gradients, the differences between interior and exterior temperatures, in massive sections immediately after the placement and for a period of time afterwards. While all cracking allows moisture and chemicals to infiltrate the concrete, structural cracking is considered more critical as it could be a result of a larger issue. Inspectors should note all visible cracks, recording their type, width, length, and location. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates that there is cracking present less than 0.012" wide or any cracks larger are sealed.

Condition State 2 indicates that there is cracking present between 0.012" and 0.05" wide. Where efflorescence is present, it's minor and there is no evidence of rust staining.

Condition State 3 indicates that there is cracking greater than 0.05" wide. Where efflorescence is present there is heavy build-up and/or rust staining.

Condition State 4 indicates the cracking warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.3.8.6 Corrosion (1000)

Section loss of steel is caused by corrosion heavy or severe enough to physically measure a reduction in member thickness.

The inspector should look for areas of laminate rust on the steel surface or areas of heavy blistering paint (caused by a build-up of rust under the paint surface). Common locations are horizontal surfaces, areas subject to salt spray from roads or areas near water surfaces. The most effective tools used to remove the laminate rust are chipping hammers and paint scrapers. Severe corrosion will usually consist of fairly loose, brown laminate rust, and a hard, tightly adhered black corrosion product on the base metal surface. A chipping hammer is usually required to remove this black corrosion. Safety glasses should always be worn when using a chipping hammer.

Once the corrosion has been removed, the amount of remaining section should be measured and recorded. Methods to measure the remaining section include the use of a caliper, a micrometer or ruler with a straight edge spanning the depression to indicate the original steel surface. The surface area affected by the corrosion should also be measured.

Some corrosion is so severe that holes are created through the steel. This situation should be recorded as “through thickness section loss” rather than “100 percent section loss” since the latter term suggests that the entire member or element has corroded away. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates no corrosion exists or it has been repaired and painted over corrosion that has been painted over will not prevent further corrosion from occurring and should not be considered a Condition State 1. Repairs suggest that the member or element has been retrofitted so that it has its original load-carrying capacity. The repairs may be in the form of bolted splices or welded splices.

Condition State 2 indicates that corrosion of the steel has initiated or freckled rust is present.

Condition State 3 indicates that section loss or pack rust exists. If the condition is not repaired/ rehabilitated, a structural analysis may be warranted in the near future. The inspector should consider recommending cleaning and painting of the member to help prevent further section loss.

Condition State 4 indicates that excessive section loss exists, and that structural analysis results indicate member capacity or serviceability is compromised.

A.3.8.7 Cracking – Steel (1010)

Steel cracking addresses fatigue cracks, regardless if they occur within a NSTM or non-NSTM bridge. Cracking in substructure elements will most often occur in steel pier caps or cross girders. Any cracks not previously detected should be evaluated to determine the potential for fracture.

Inspectors should be especially thorough at details which are prone to developing fatigue cracking. Refer to Appendix D for a list of fatigue details. Detection of these cracks will most often occur during In-Depth or NSTM Inspections but can still be found during Routine Inspections as well. Any crack defects classified as a Condition State 3 or worse should be immediately reported to the Inspection Program Manager. Serious consideration should be given to closing the bridge until the extent and cause of cracking is determined and appropriate remedial action is taken. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates that no cracking exists in the member.

Condition State 2 indicates that a crack exists, but it has either been arrested by drilling holes at the tips or it has been repaired with bolted splices or welded splices. Shallow or short cracks may also be ground out. Unrepaired, self-arrested, or cracks not arrested with drilled holes may also fall into this Condition State, but they are located in a non-critical or redundant member.

Condition State 3 indicates that a crack which has not been arrested or repaired exists in a primary load-carrying member, including newly-discovered crack or existing cracks that are still growing between inspection cycles. Cracks of this nature suggest it is fatigue related, stable crack growth.

Condition State 4 indicates that a very large crack exists in any primary load-carrying member. Newly-discovered cracks are greater than 3 inches in length, or existing cracks have grown more than 6 inches within the last 24 months. Cracks of this nature suggest they developed in an uncontrolled brittle fashion.

A.3.8.8 Connection (1020)

Connections can be constructed from two or more members or elements that may be bolted, riveted, or welded together. Connection deficiencies include any deterioration or damage of fasteners, such as rivets, bolts, or welds and connection plates (gusset plates). Any suspected loose or deteriorated fasteners should be struck with a hammer to determine soundness. Welded connections should be visually inspected for fatigue cracks developing from weld imperfections or overstress. Connections are also often subjected to distortion due to pack rust (corrosion occurring between two pieces of lapped steel). The amount of separation caused by pack rust and any distortion to the steel should be measured and recorded. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates that the connection is in place and is functioning as intended.

Condition State 2 indicates that there are loose fasteners and/or pack rust without distortion. The connection is in place and functioning as intended despite these defects.

Condition State 3 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. Although these defects are present, the structural capacity of the structure is not reduced.

Condition State 4 indicates that there are missing bolts, rivets, broken welds, fasteners, and/or pack rust with distortion. These defects reduce the structural capacity of the member assembly.

A.3.8.9 Cracking – Prestressed Concrete (1110)

Concrete cracks might indicate a significant loss in strength in prestressed concrete members. Loss of prestress due failure of prestressing steel may result in a serious defects manifested in loss of camber, beam sag, and/or wide structural cracks. Inspectors should note all visible cracks, recording their type, width, length, and location. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates that there is cracking present less than 0.004” wide or any wider cracks are sealed.

Condition State 2 indicates that there is cracking present between 0.004” and 0.009” wide. Where efflorescence is present, it’s minor and there is no evidence of rust staining.

Condition State 3 indicates that there is cracking present greater than 0.009”. Where efflorescence is present, there is heavy build up or rust staining.

Condition State 4 indicates that there is cracking which warrants a structural review to determine the effect on strength or serviceability of the element or bridge, or a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.3.8.10 Abrasion/Wear (1190) (PSC/RC)

Vehicular traffic is a source of abrasion and wear on concrete decks. Abrasion occurs on concrete piles that are subjected to flowing water or tidal flows. Mechanical wear of concrete members sometimes occurs due to movement of the fasteners against their holes when connections become loose. This defect is used only for jacketing and external post tensioning only.

Condition States

Condition State 1 indicates no abrasion.

Condition State 2 indicates abrasion has exposed coarse aggregate but the aggregate remains secure in the concrete.

Condition State 3 indicates coarse aggregate is loose or has popped out of the concrete matrix due to abrasion.

Condition State 4 indicates the condition warrants a structural review to determine the effect on strength or serviceability of the element or bridge; OR a structural review has been completed and the defects impact strength or serviceability of the element or bridge.

A.3.9 Wall Defects

Wall defects can be found in Part 4 of this manual.

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APPENDIX B: FATIGUE PRONE DETAILS

General Conditions	Situation	Kind of Stress	Stress Category	Illustrative Example (See Pgs 1-D-5 & 6)
Plain Member	Base metal with rolled or cleaned surface. Flame-cut edges with ANSI smoothness of 1,000 or less.	T or Rev[a]	A	1,2
Built-Up Members	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds (with backing bars removed) or by continuous fillet welds parallel to the direction of applied stress.	T or Rev	B	3,4,5,7
	Base metal and weld metal in members of built-up plates or shapes (without attachments) connected by continuous full penetration groove welds with backing bars not removed, or by continuous partial penetration groove welds parallel to the direction of applied stress.	T or Rev	B'	3,4,5,7
	Calculated flexural stress at the toe of transverse stiffener welds on girder webs or flanges.	T or Rev	C	6
	Base metal at ends of partial length welded coverplates with high-strength bolted slip-critical end connections (see Note[b])	T or Rev	B	22
	Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends, or wider than the flange without welds across the ends.			
	(a) Flange thickness ≤ 0.8 in. (b) Flange thickness > 0.8 in.	T or Rev T or Rev	E E'	7 7
	Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	T or Rev	E'	7
Groove Welded Connections	Base metal and weld metal in or adjacent to full penetration groove weld splices of rolled or welded sections having similar profiles when welds are ground flush with grinding in the direction of applied stress and weld soundness established by nondestructive inspection.	T or Rev	B	8,10
	Base metal and weld metal in or adjacent to full penetration groove weld splices with 2-foot radius transitions in width, when welds are ground flush with grinding in the direction of applied stress and weld soundness established nondestructive inspection.	T or Rev	B	13
	Base metal and weld metal in or adjacent to full penetration groove weld splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2½, with grinding in the direction of the applied stress, and weld soundness			

	established by nondestructive inspection:			
	(a) AASTHO M 270 Grades 100/100W (ASTM A 709) base metal	T or Rev	B’	11,12
	(b) Other base metals	T or Rev	B	11,12
	Base metal and weld metal in or adjacent to full penetration groove weld splices, with or without transitions having slopes no greater than 1 to 2½, when the reinforcement is not removed and weld soundness is established by nondestructive inspection.	T or Rev	C	8,10,11, 12
Groove Welded Attachments—Longitudinally Loaded[c]	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is less than 2 inches.	T or Rev	C	6.15
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is between 2 inches and 12 times the plate thickness, but less than 4 inches.	T or Rev	D	15
	Base metal adjacent to details attached by full or partial penetration groove welds when the detail length, L, in the direction of stress, is greater than 12 times the plate thickness or greater than 4 inches:			
	(a) Detail thickness < 1.0 in.	T or Rev	E	15
	(b) Detail thickness ≥ 1.0 in.	T or Rev	E’	15
	Base metal adjacent to details attached by full or partial penetration groove welds with a transition radius, R, regardless of the detail length:			
	--With the end welds ground smooth	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 in. > Transition radius ≥ 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	--For all transition radii without end welds ground smooth.	T or Rev	E	16
Groove Welded Attachments—Transversely Loaded[c,d]	Detail base metal attached by full penetration groove welds with a transition radius, R, regardless of the detail length and with weld soundness transverse to the direction of stress established by nondestructive inspection:			
	--With equal plate thickness and reinforcement removed	T or Rev		16
	(a) Transition radius ≥ 24 in.		B	
	(b) 24 inc. > Transition radius > 6 in.		C	
	(c) 6 in. > Transition radius ≥ 2 in.		D	
	(d) 2 in. > Transition radius ≥ 0 in.		E	
	--With equal plate thickness and reinforcement not removed	T or Rev		16
	(a) Transition radius ≥ 6 in.		C	

	(b) 6 in. > Transition radius ≥ 2 in. (c) 2 in. > Transition radius ≥ 0 in.		D E	
	--With unequal plate thickness and reinforcement removed (a) Transition radius ≥ 2 in. (b) 2 in. > Transition radius ≥ 0 in.	T or Rev	 D E	16
	--For all transition radii with unequal plate thickness and reinforcement not removed.	T or Rev	E	16
Fillet Welded Connections	Base metal at details connected with transversely loaded welds, with the welds perpendicular to the direction of stress:			
	(a) Detail thickness ≤ 0.5 in. (b) Detail thickness > 0.5 in.	T or Rev T or Rev	C See Note[e]	14
	Base metal at intermittent fillet welds.	T or Rev	E	--
	Shear stress on throat of fillet welds.	Shear	F	9
Fillet Welded Attachments—Longitudinally Loaded	Base metal adjacent to details by fillet welds with length, L, in the direction of stress, is less than 2 inches and stud-type shear connectors.	T or Rev	C	15,17,18,20
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, between 2 inches and 12 times the plate thickness but less than 4 inches.	T or Rev	D	15,17
	Base metal adjacent to details attached by fillet welds with length, L, in the direction of stress, greater than 12 times the plate thickness or greater than 4 inches. (a) Detail thickness < 1.0 in. (b) Detail thickness ≥ 1.0 in.	 T or Rev T or Rev	 E E'	 7,9,15,17 7,9,15
	Base metal adjacent to details attached by fillet welds with a transition radius, R, regardless of the detail length:			
	--With the end welds ground smooth (a) Transition radius ≥ 2 in. (b) 2 in. > Transition radius ≥ 0 in.	T or Rev	 D E	16
	--For all transition radii without the end welds ground smooth.	T or Rev	E	16
Fillet Welded Attachments—Transversely Loaded with the Weld in the Direction of Principal Stress	Detail base metal attached by fillet welds with a transition radius, R, regardless of the detail length (shear stress on the throat of fillet welds governed by Category F):			
	--With the end welds ground smooth (a) Transition radius ≥ 2 in. (b) 2 in. > Transition radius ≥ 0 in.	T or Rev	 D E	16
	--For all transition radii without the end welds ground smooth.	T or Rev	E	16

Mechanically Fastened Connections	Base metal at gross section of high-strength bolted slip-resistant connections, except at axially loaded joints which induce out-of-plane bending in connecting materials.	T or Rev	B	21
	Base metal at net section of high-strength bolted bearing-type connections.	T or Rev	B	21
	Base metal at net section of riveted connections.	T or Rev	D	21
Eyebar or Pin Plates	Base metal at the net section of eyebar head or pin plate	T	E	23,24
	Base metal in the shank of eyebars, or through the gross section of pin plates with:			
	(a) rolled or smoothly ground surfaces	T	A	23,24
	(b) flame-cut edges	T	B	23,24

[a] "T" signifies range in tensile stress only. "Rev" signifies a range of stress involving both tension and compression during a stress cycle.

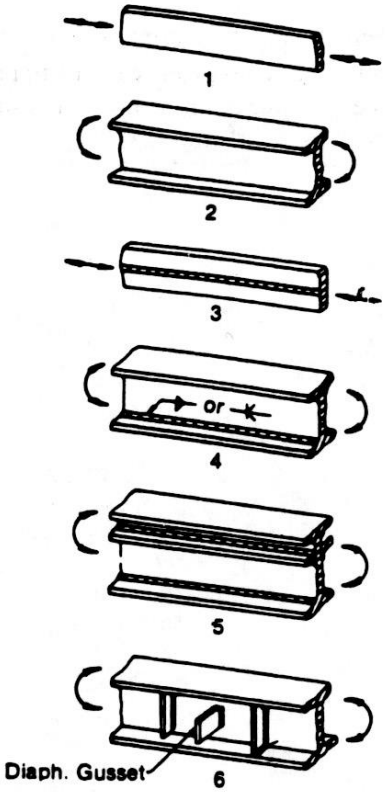
[b] See Watter, Albrecht and Sahli, *Journal of Structural Engineering*, ASCE, Vol. III, No. 6, June 1985, pp. 1235-1249.

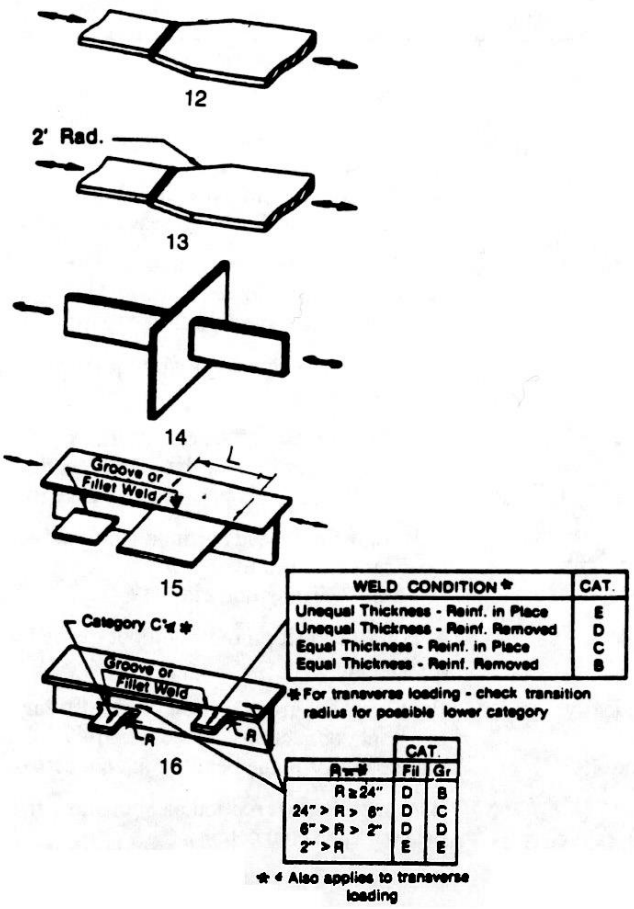
[c] "Longitudinally Loaded" signifies direction of approved stress is parallel to the longitudinal axis of the weld. "Tranversely Loaded" signifies direction of applied stress is perpendicular to the longitudinal axis of the weld.

[d] Transversely loaded partial penetration groove welds are prohibited.

[e] Allowable fatigue stress range on throat of fillet welds transversely loaded is a function of the effective throat and plate thickness. (See Frank and Fisher, *Journal of the Structural Division*, ASCE, Vo. 105, No. ST9, Sept. 1979).

[f] Gusset plates attached to girder flange surfaces with only transverse fillet welds are prohibited.



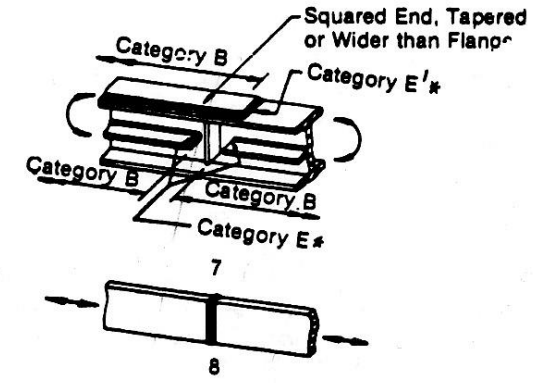


WELD CONDITION *	CAT.
Unequal Thickness - Reinf. in Place	E
Unequal Thickness - Reinf. Removed	D
Equal Thickness - Reinf. in Place	C
Equal Thickness - Reinf. Removed	B

* For transverse loading - check transition radius for possible lower category

R =	CAT. Fil	CAT. Gr
R ≥ 24"	D	B
24" > R > 6"	D	C
6" > R > 2"	D	D
2" > R	E	E

* 4 Also applies to transverse loading



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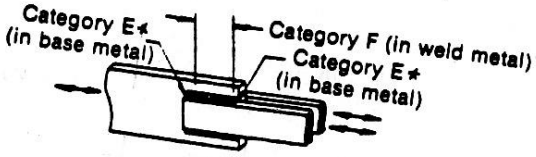

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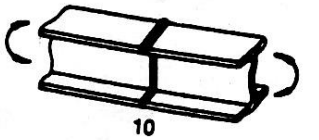
* At End of Weld, Has No Length



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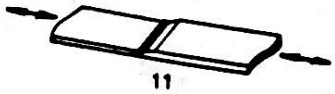

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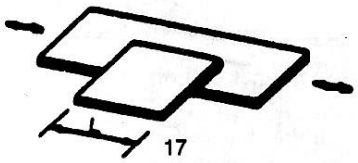

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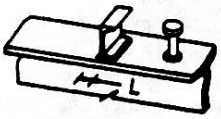

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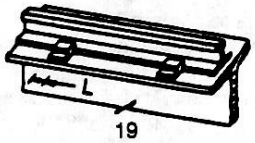

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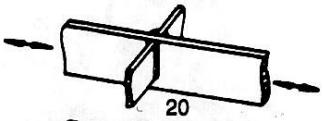

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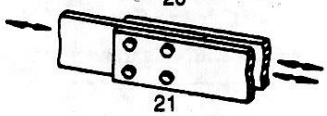

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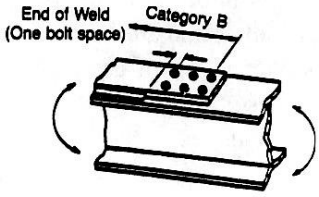

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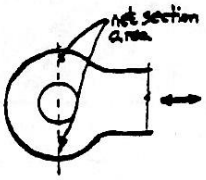

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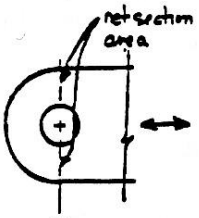

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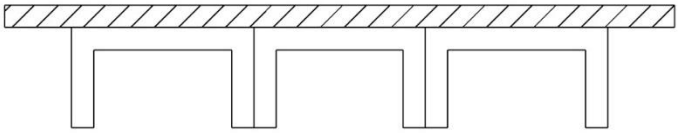
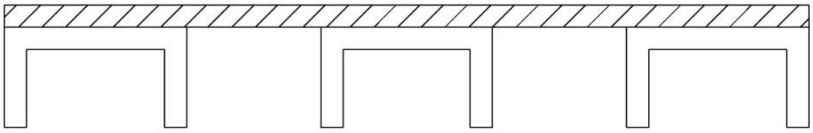
APPENDIX C: STATE MEASUREMENT CONVENTIONS

SECTION C.1 GIRDER MEASUREMENT CONVENTIONS

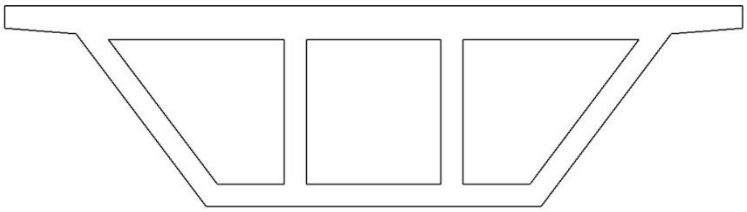
The “number of girders” for a bridge as carried in the management database can be determined using one of the following examples. The total quantity recorded is equal to the girder length multiplied by the number of girders.



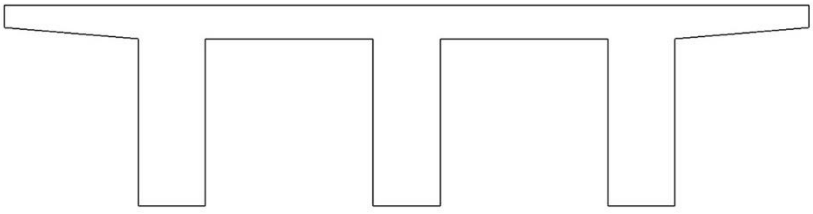
3 GIRDERS



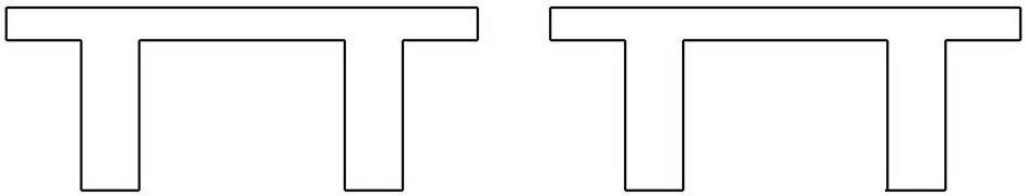
3 GIRDERS



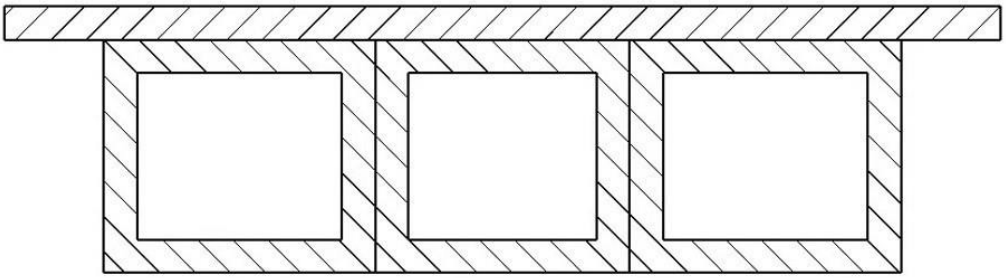
1 GIRDER



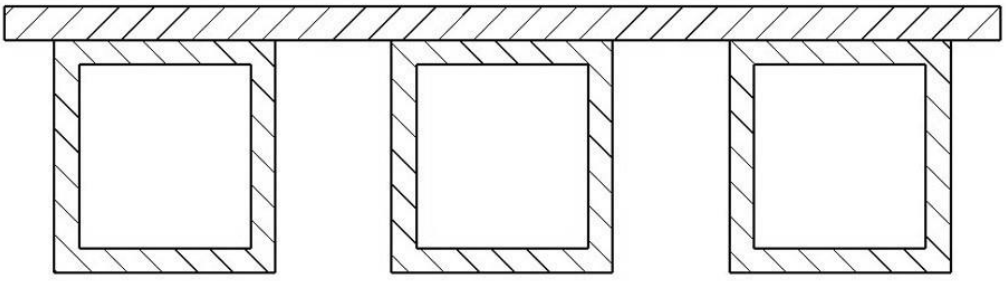
3 GIRDERS



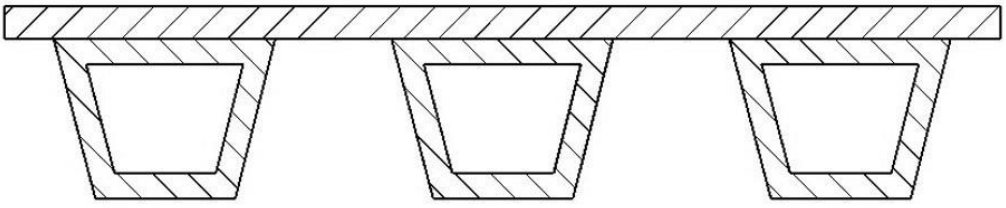
2 GIRDERS



3 GIRDERS



3 GIRDERS



3 GIRDERS

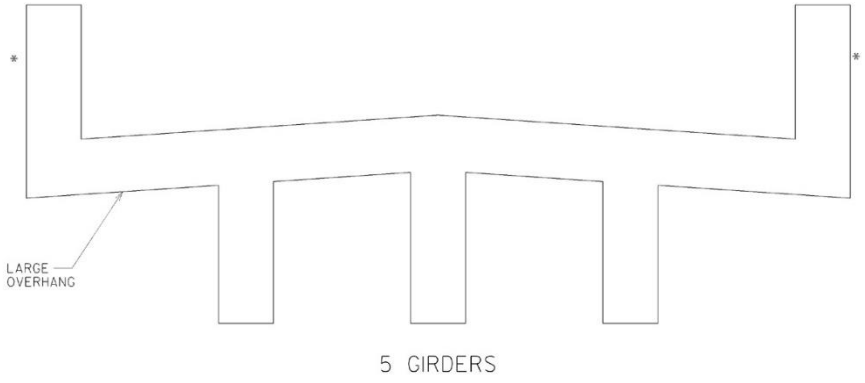
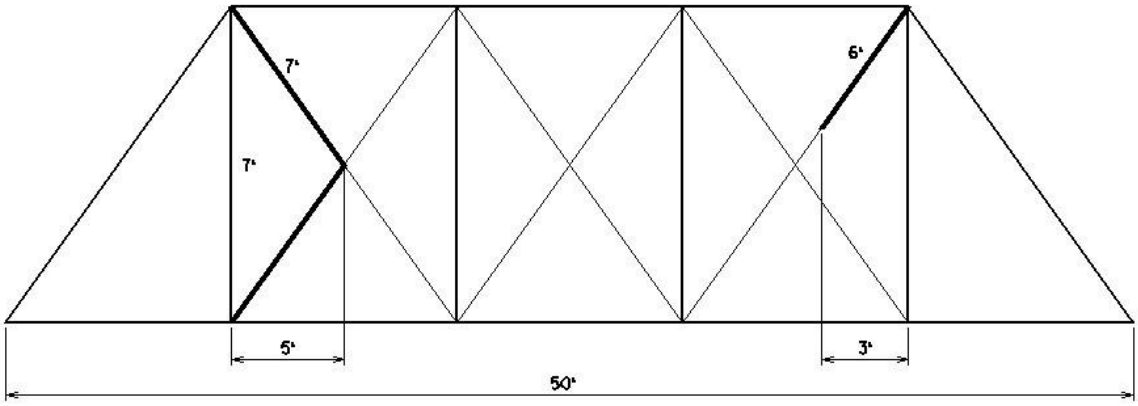


Figure C-1: On old reinforced concrete deck girders with large overhangs, the rail will act as a girder if it is a solid rail.

SECTION C.2 TRUSS MEASUREMENT CONVENTIONS



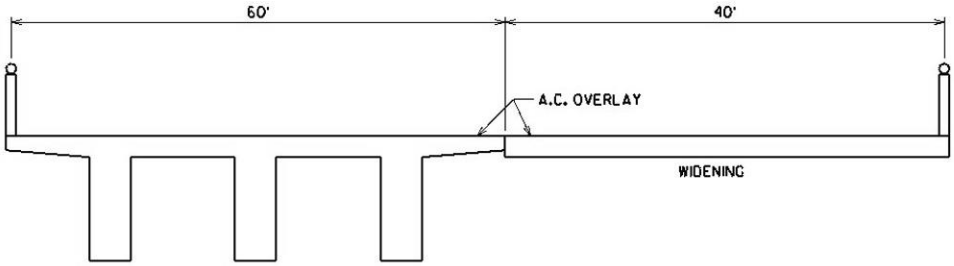
— DETERIORATED PARTS OF TRUSS

All measurements of a truss are along the horizontal projection, including the deterioration measurements. The total length of the above truss is 50 feet, while the recorded length of the deteriorated part of the truss is equal to 8 feet (5 feet + 3 feet).

The total quantity for the truss element for the above example bridge is 100 feet (50 * 2, since there is a truss on each side of the bridge).

The convention used for spandrel arches is similar to that used for trusses.

SECTION C.3 DECK MEASUREMENT CONVENTIONS



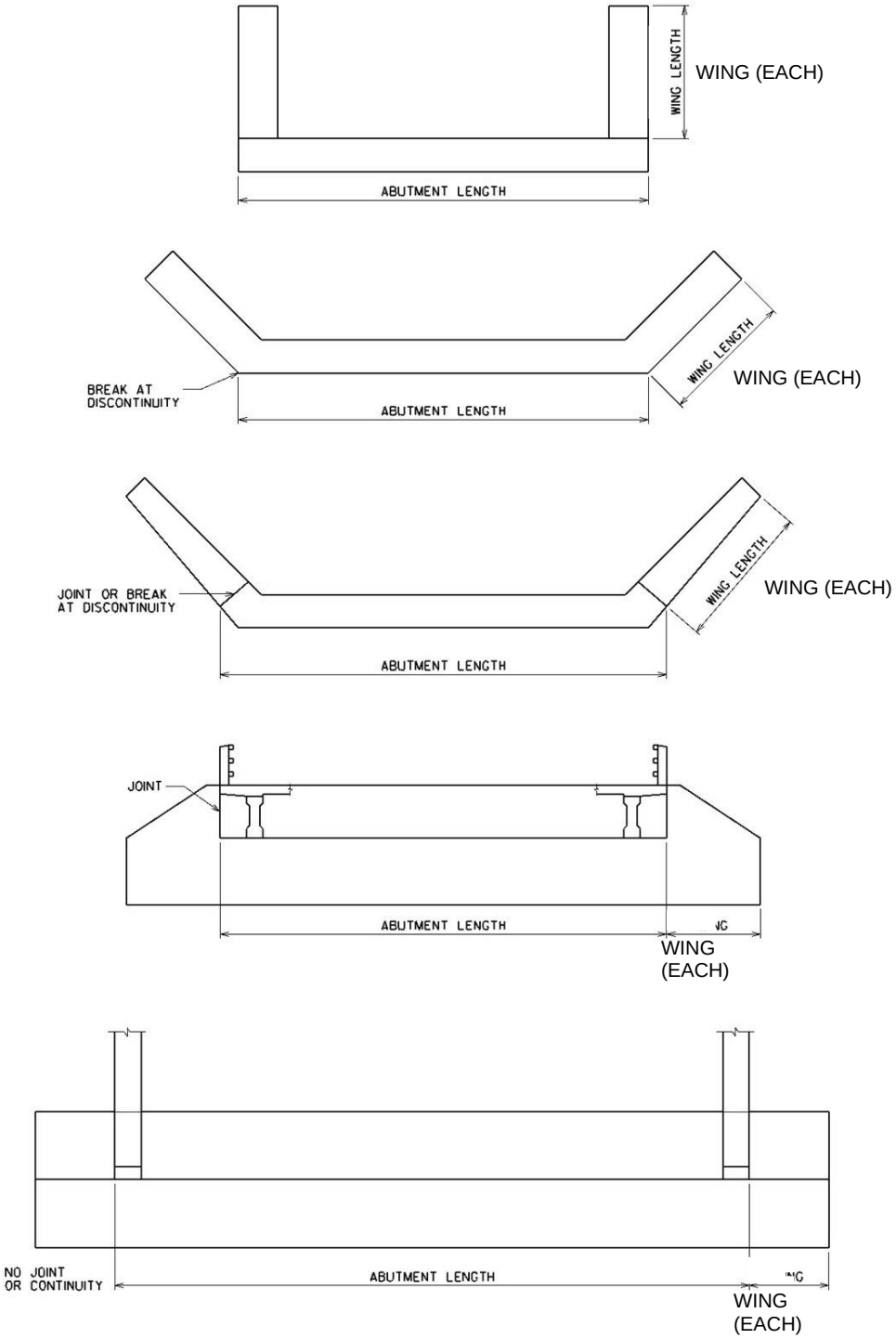
TYPICAL SECTION

(Assumes a 100 foot long structure)

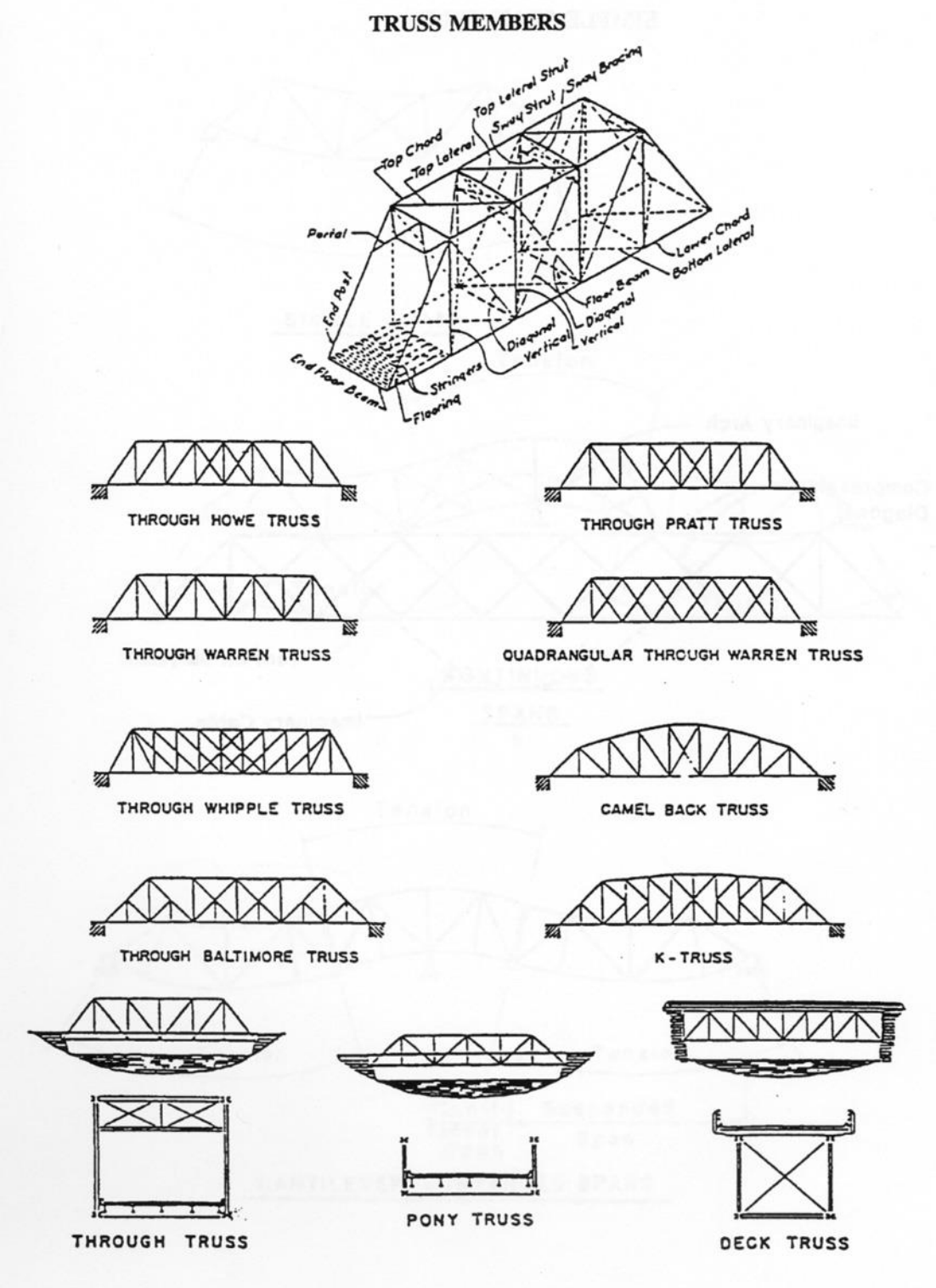
The unit for deck or slab elements is “square feet”. Thus, when a bridge has one deck/slab type, the quantity is the deck area. However, when a bridge has two different types of deck/slab elements (See the example below with Elements 12 and 38), the inspector should record a quantity equal to the deck area for each deck type element. The entire quantity for each deck or slab element is assigned to a single Condition State.

Element No.	Description	Quantity	Unit
12	Reinforced Concrete Deck	6,000	SF
38	Reinforced Concrete Slab	4,000	SF

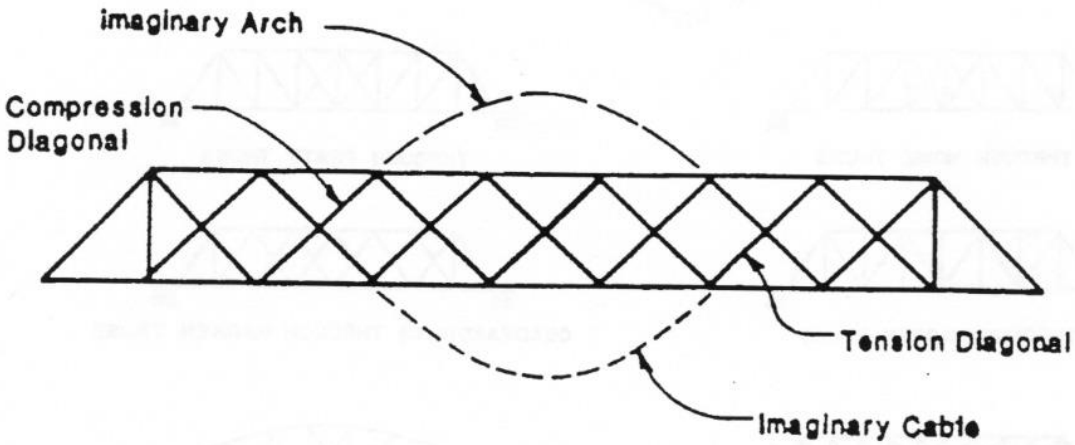
SECTION C.4 ABUTMENT AND WINGWALL LENGTH MEASUREMENT CONVENTIONS



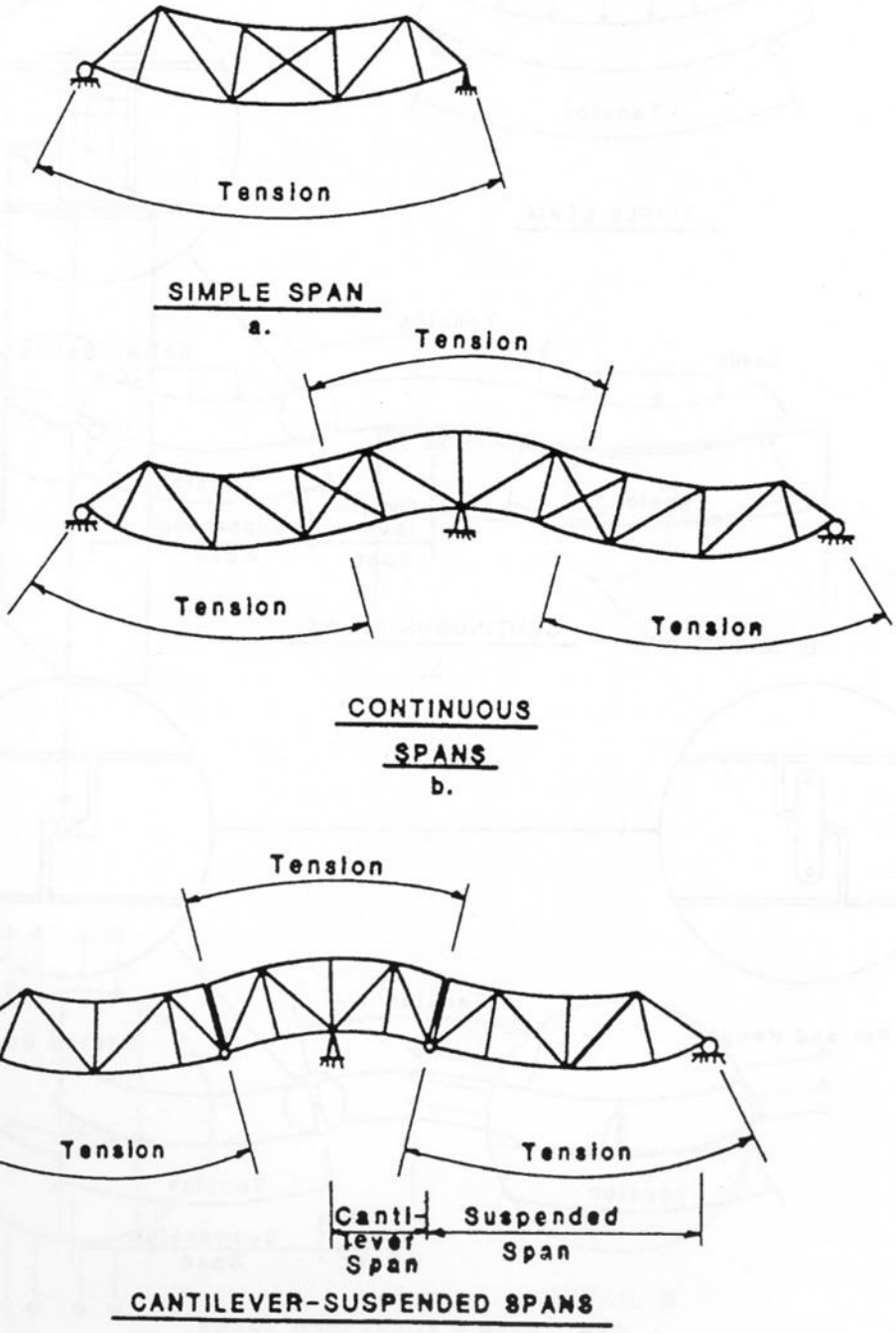
APPENDIX D: Nonredundant Steel Tension Member (NSTM) DETAILS



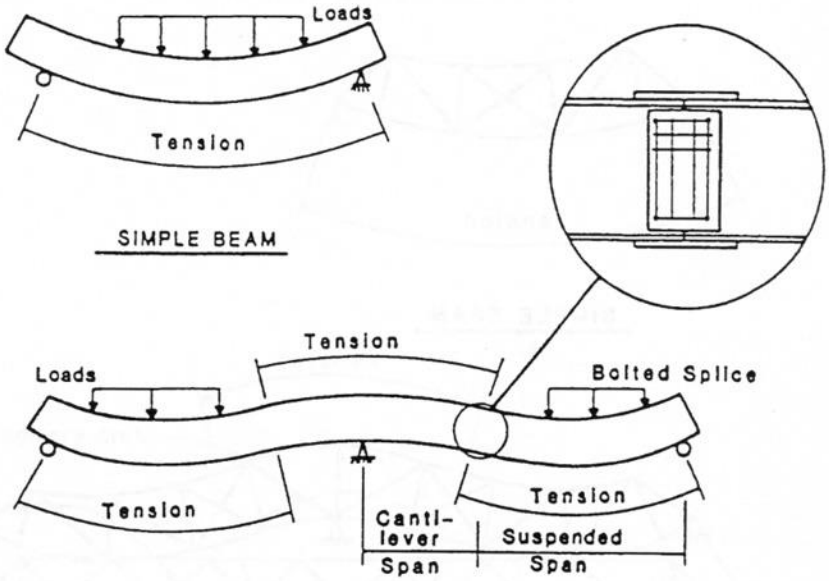
SIMPLE SPAN TRUSS



BENDING IN TRUSS



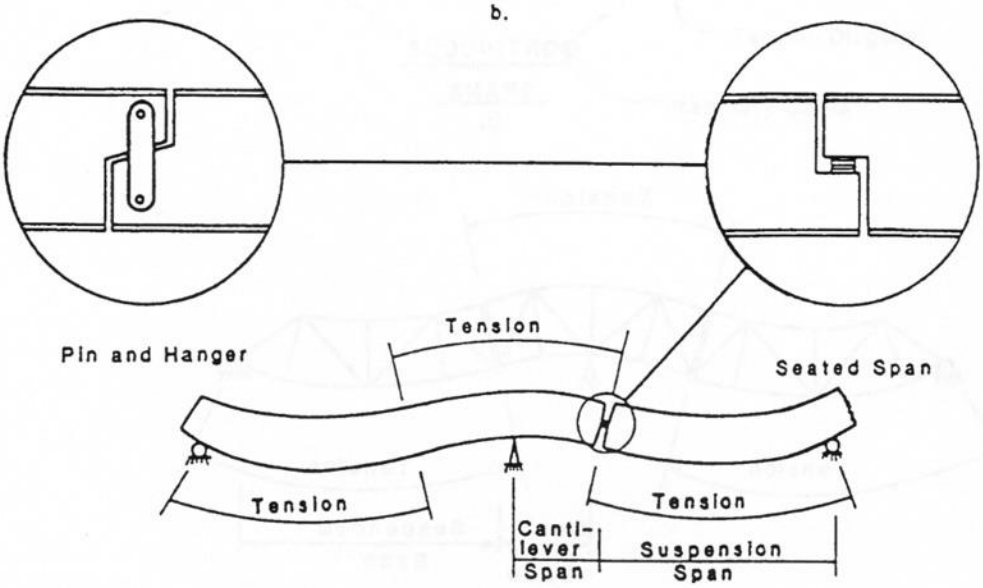
BENDING IN GIRDERS



SIMPLE BEAM

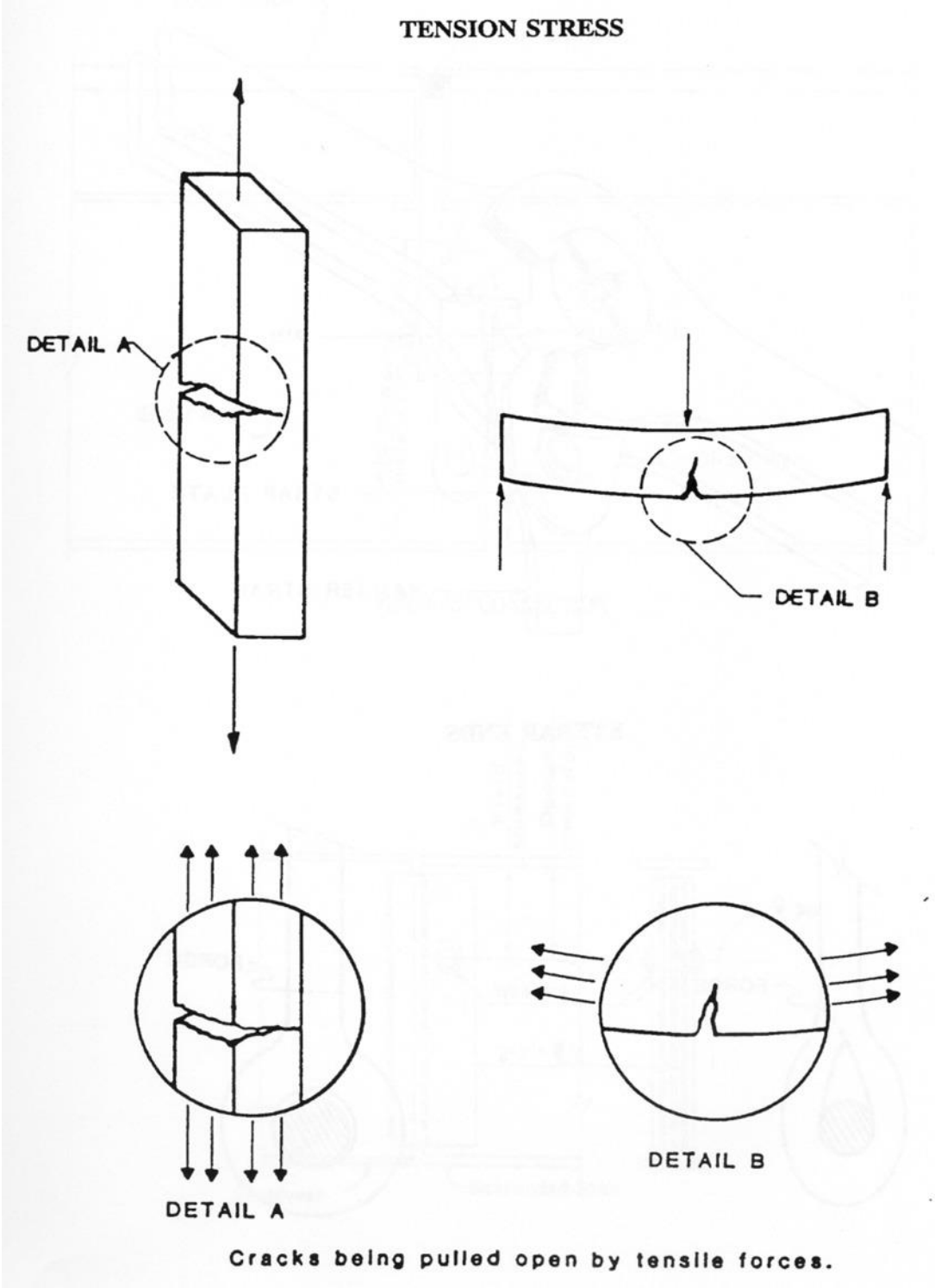
CONTINUOUS SPANS

b.



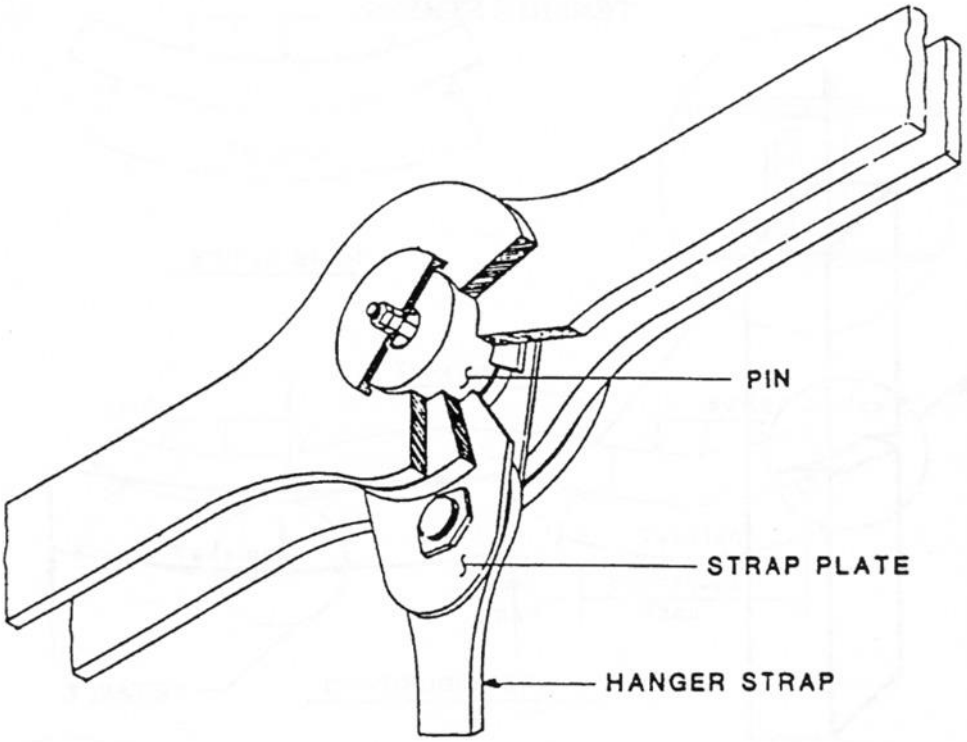
c.

CANTILEVER - SUSPENDED SPANS

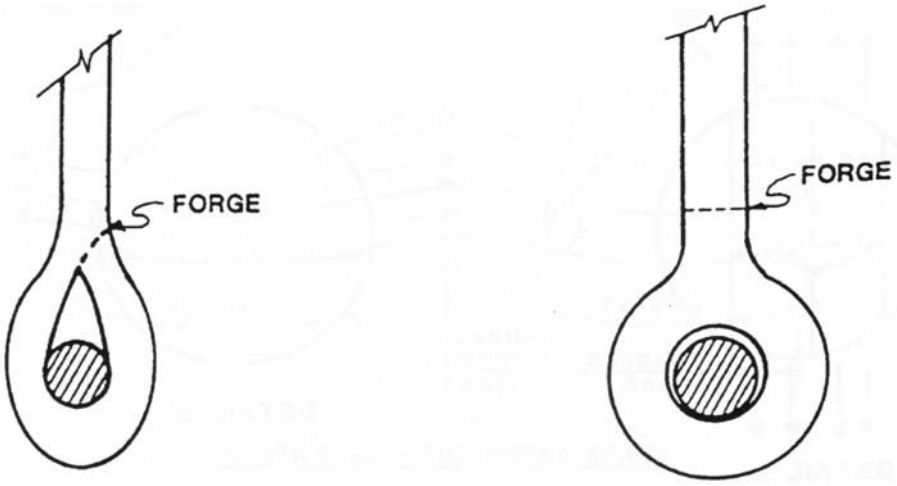


Cracks being pulled open by tensile forces.

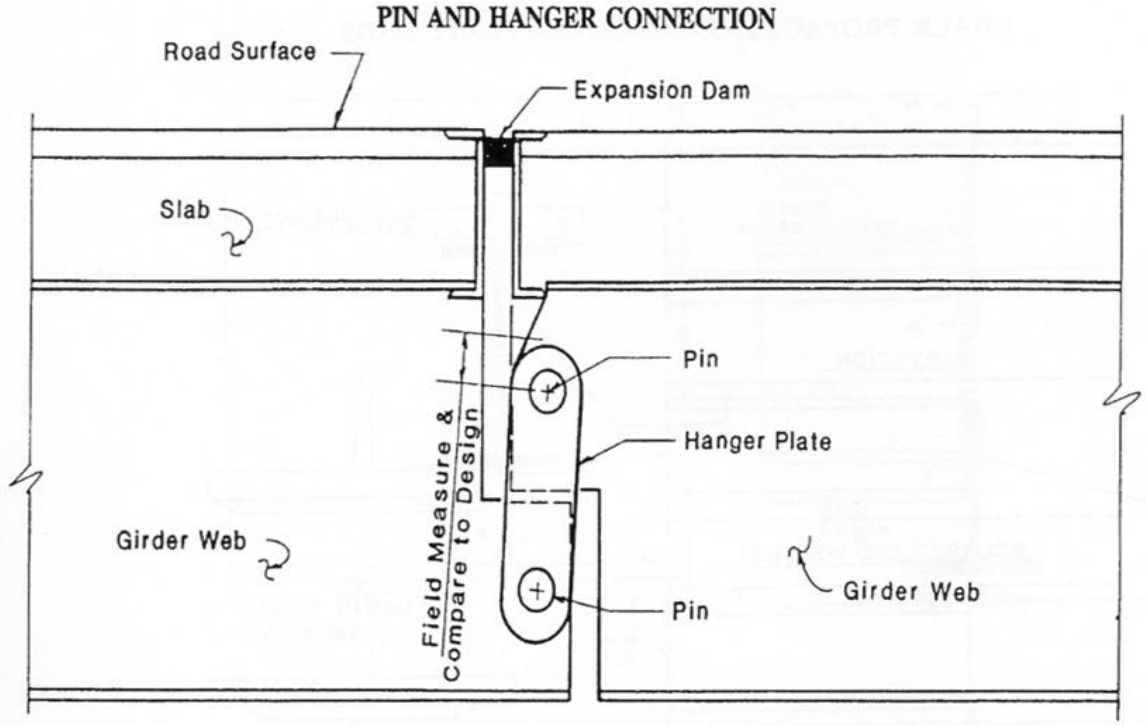
EYEBAR HANGER CONNECTION

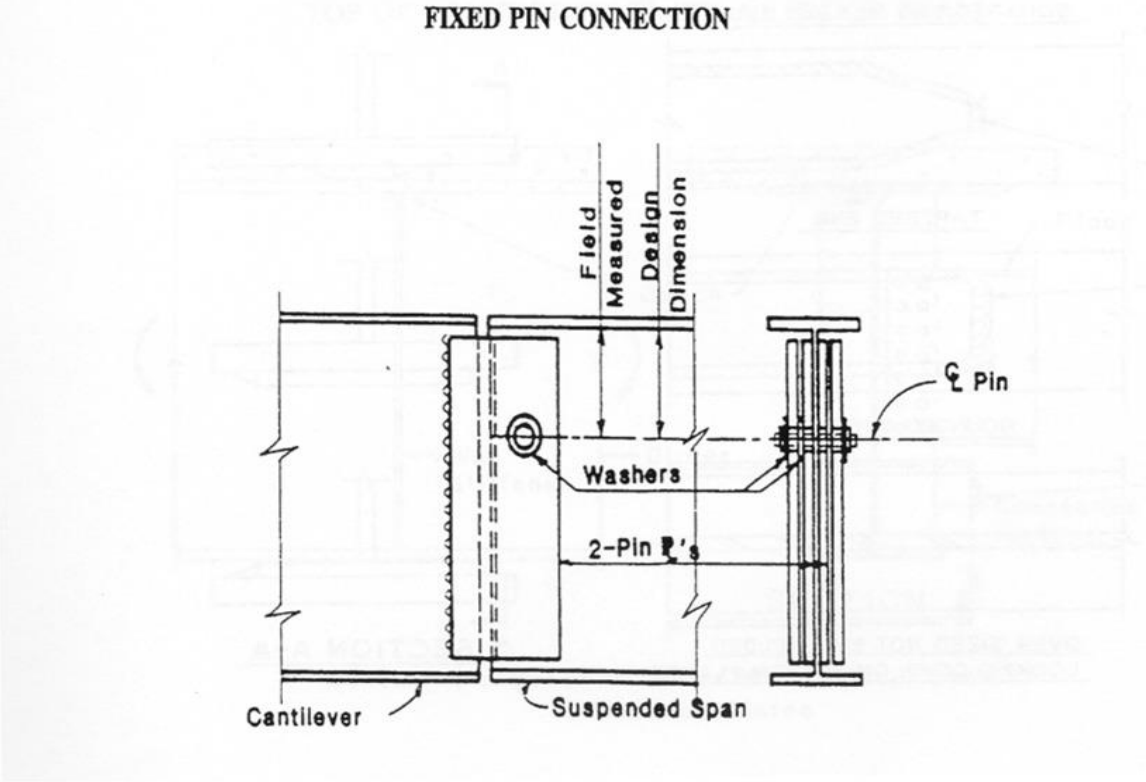


EYEBAR ENDS

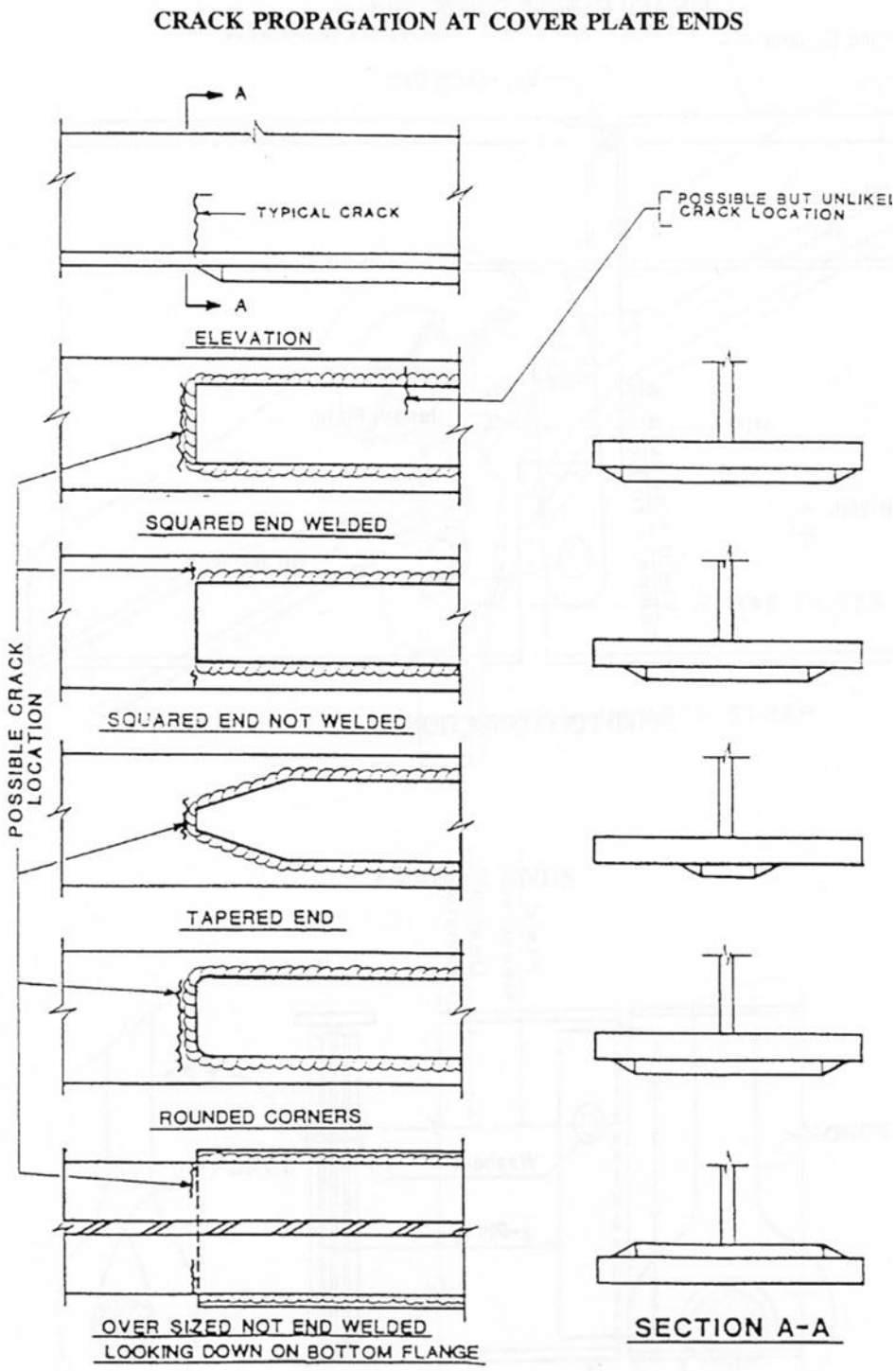


Fatigue Categories A, B (on eyebar body), or E (on net section of eyebar head)

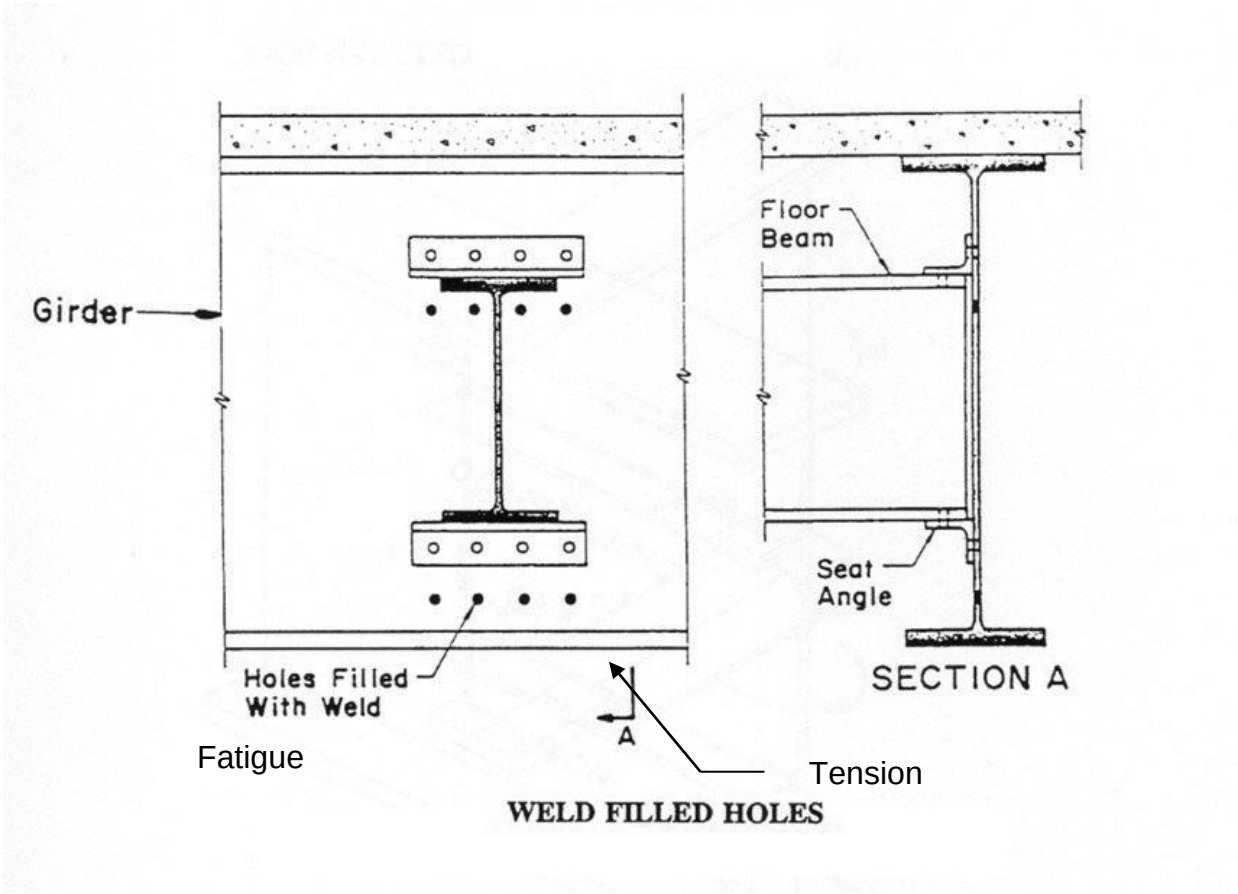


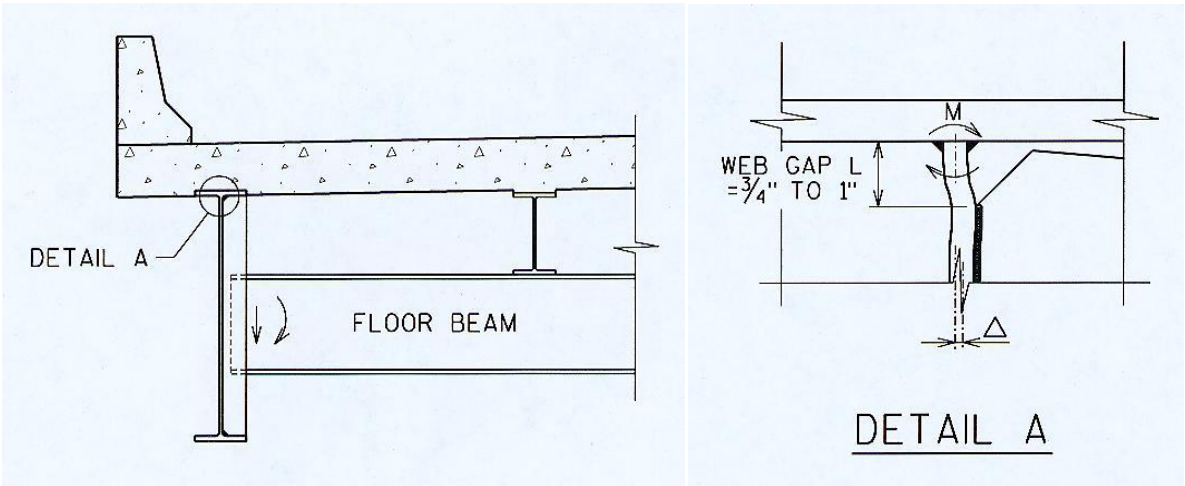


Fatigue Category A, B (on hanger plate body), or E (on net section of hanger or pin plate)



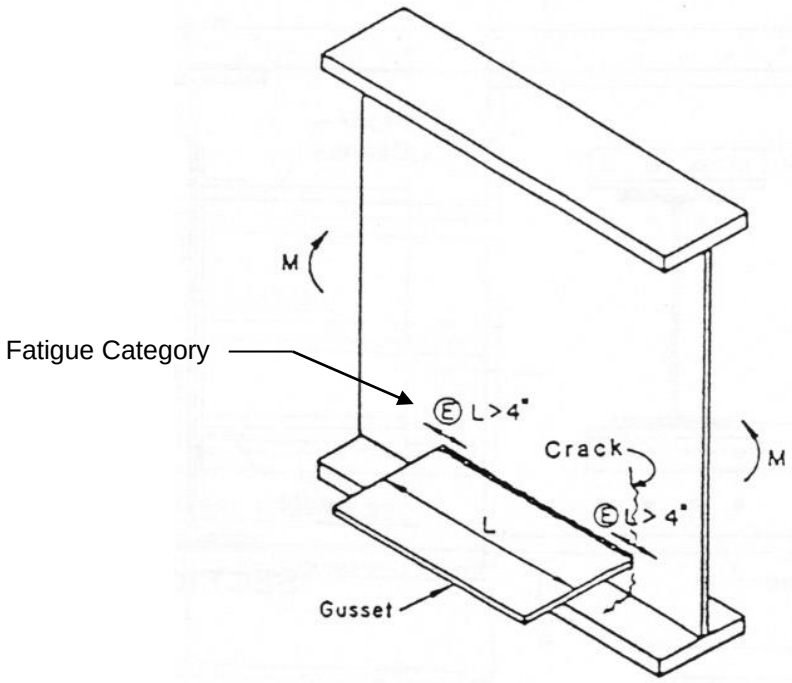
Fatigue Categories E and E’

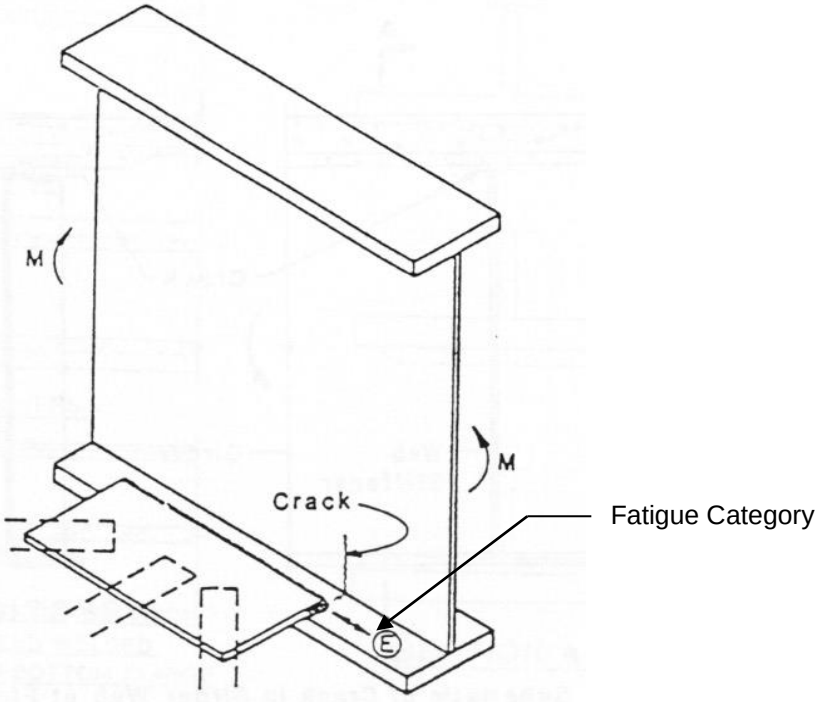




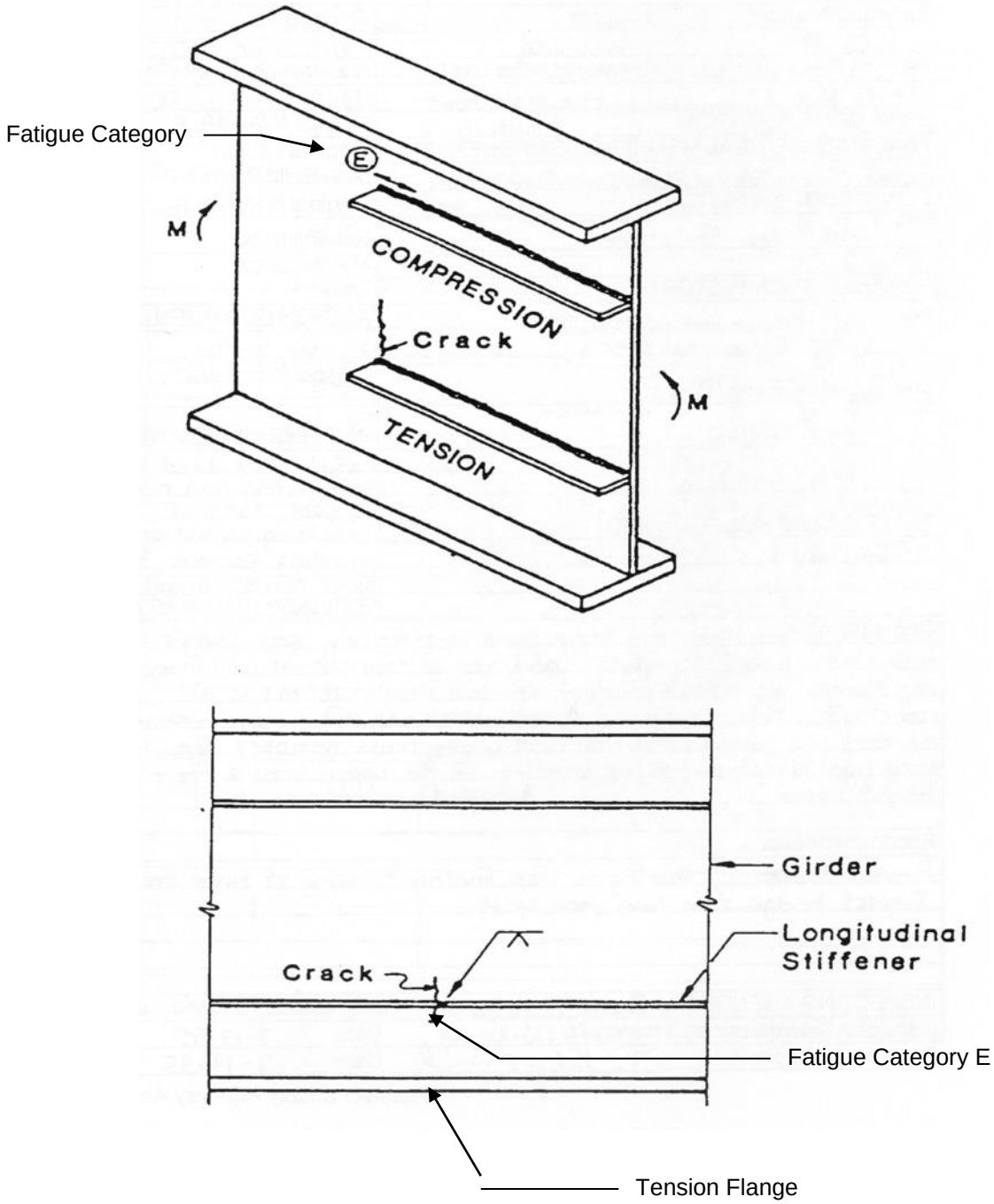
Web Out-of-Plane Bending at Floor Beam Connection Plate

FLANGE AND WEB ATTACHMENTS





LONGITUDINAL STIFFENERS



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3.1 GENERAL

3.1.1 Introduction

The most common movable structure in the transportation network is the movable bridge. Although there are other movable structures in the network, such as movable components of ferry terminals, weigh scales, and other similar type structures, Part 3 of this manual will mainly refer to movable bridges. Many of the principles and systems of movable bridges are similar to those of other movable structures. No matter what type of movable structure, the primary reference document will be the structure's own Operation and Maintenance (O & M) Manual.

A movable bridge is a bridge across a navigable waterway that has at least one span which can be temporarily moved in order to increase the vertical clearance for vessels sailing through the channel. Such a bridge is built where site conditions preclude constructing a bridge with only fixed spans that has an acceptable vertical profile. Although military movable bridges have been erected since ancient times, it was not until the advent of transportation canals and railways that many permanent movable bridges were built as part of a country's infrastructure. Because the early movable bridges were manually powered, ingenious mechanisms were devised to gain mechanical advantage. By 1900, the now common types of movable bridges had been developed and many were patented.

Part 3 of the Wisconsin Department of Transportation (WisDOT) Structure Inspection Manual is concerned primarily with the electrical, mechanical, and hydraulic machinery for operating and stabilizing the movable span. Inspection of the superstructure and substructure of the movable bridge will not be addressed herein because these tasks are similar to those for fixed bridges, which are treated in Part 2. However, the interaction between the movable structure and the machinery will be addressed. An understanding of the mechanical/structural interaction of the movable bridge is important for adequate inspection and maintenance of the machinery.

The addition of electrical and mechanical systems within a movable bridge defines the structure as complex. NBIS Metric #19: Inspection Procedures – Complex Bridges states:

- Complex bridges have the following identified:
    - Specialized inspection procedures which clearly identify the complex features, specify the inspection interval of those features, describe any specific risk factors unique to the bridge, and clearly detail inspection methods and equipment to be employed.
    - Additional inspector training and experience required to inspect complex bridges.
- Complex bridges are inspected according to those procedures.

3.1.1.1 Inspection Forms

The following Inspections formed are to be utilized and uploaded as part of any Moveable Bridge inspection:

For Bascule Bridges:

DT 2014 *Movable Bridge Electrical Inspection Report*

DT 2015 *Movable Bridge Structural Inspection Report*

DT 2016 *Movable Bridge Hydraulic Inspection Report*

DT 2017 *Movable Bridge Mechanical Inspection Report*

For Vertical Lift Bridges:

DT 2027 *Movable Bridge Electrical Inspection Report*

DT 2028 *Movable Bridge Hydraulic Inspection Report*

DT 2029 *Movable Bridge Mechanical Inspection Report*

DT 2030 *Movable Bridge Structural Inspection Report*

3.1.1.2 Local Lift Bridge Aids

The Lift Bridge Aids Program applies to moveable bridges on connecting highways. A Program Manager from the Department's Bureau of Transit, Local Roads, Railroads and Harbors administers the program. This is covered under section 86.32(2)(a) of the Wisconsin statutes.

> *86.32(2)(a)* ***(a)*** *Cities, villages and towns shall be reimbursed for actual costs, as approved by the department, incurred in maintaining and operating lift bridges. Documentation of costs shall be submitted by each city, village and town by January 31 and reimbursement shall be made, starting in 1982-83, on the first Monday in July for costs incurred during the prior calendar year. If the amount appropriated under s. 20.395 (1) (ft) is insufficient to pay the actual costs approved by the department for the maintenance and operation of lift bridges, the department shall prorate the amount appropriated in the manner it deems desirable.*

The cities currently included in the program:

Green Bay and Manitowoc in the North East Region

Racine and Milwaukee in the South East Region

Must get the previous calendar year expenses & projected expenses for the next calendar year submitted by Jan. 31st of each year. The previous year expenses, along with projected expenses for the upcoming year are reported on form DT2088 with all supporting documentation. The Regional Inspection Program Manager, or their delegate reviews the documentation and the Statewide Inspection Program manager in the Bureau of Structures signs off on the documentation.

3.1.2 Inspection Procedures

It is necessary for the inspector to identify the inspection interval of the unique movable bridge components and describe the inspection procedure used to carry out the inspection. This is accomplished through the use of the WisDOT field inspection forms. There is space available for the inspector to document the procedures utilized prior to and during the inspection of the bridge. As an alternative, a more complex or lengthy inspection procedure may be a separate document that is uploaded during the entry of the structure inspection. The inspector should note the operator was called to notify him of inspection of the bridge, whether any electrical control boxes were shut down during inspection and describing any specific procedures used while observing of the structure during operation, as an example.

Sample inspection procedures for a movable bridge are included below for reference:

Olde Oneida Bascule Bridge over the Fox River Navigational System Canal at Appleton, Wisconsin

Movable Bridge Inspection Procedures

Bridge Description *– This movable bridge carries one lane of traffic in each direction and accommodates pedestrians by sidewalks on both sides. Its single leaf trunnion-style bascule span is driven by an electro-mechanical system that is operated on-site from the control house at its northwest quadrant.*

Bridge Owner and Contact:

City of Appleton; Chad M. Weyenberg, P.E., Project Engineer, telephone: (920) 832-5915, email: chad.weyenberg@appleton.org

Inspection Interval:

An inspection will be made of the bascule span's mechanical and electrical system and its structural elements associated with operation as a movable structure at an interval not to exceed 12 months.

Inspection Forms:

The following WisDOT forms provide a systematic procedural checklist of items to inspect. All applicable items on them shall be fully completed while on-site during inspection.

- *Movable Bridge Electrical Inspection Report*
- *Movable Bridge Structural Inspection Report*
- *Movable Bridge Mechanical Inspection Report*

Complex Features:

The above referenced inspection forms provide a comprehensive list of the specialized electrical, mechanical and structural features of this movable bridge to be inspected. Noteworthy ones include the following:

- *Barrier gate at south end of bridge providing positive restraint to approaching vehicles during bridge openings*
- *Toe locks that hold the end of the movable span closed for safe vehicle passage*
- *Safety interlocking system as part of the control system for bridge operation*
- *Motor brakes that stop the moving bascule leaf and machinery brakes that keep the leaf stationary*
- *Counterweight system that minimizes load on machinery system and motors in all positions of opening*
- *Breaks between movable and non-movable structural components in roadway and sidewalks*

Risk Factors:

The following are risk factors for inspection staff, the public and the bridge:

- *Certain features of this bridge require the bascule span to be operated for inspecting them. This requires inspectors to fully understand where those safe areas are that they must be during test operations.*
- *Unanticipated operation of the bridge during inspection can be hazardous. Before performing any test operation, a clear protocol for communication between the bridge operator and all inspection staff must be established and understood by all parties. An clear line of communication between the operator and primary point of contact for the inspection team must be established and maintained. Communication can be via cell phone, walkie-talkie or similar device.*
- *Inspection of the electrical control system requires working near energized components that can pose potential electrocution hazards.*
- *Safety for motorists and pedestrians during test operations is of paramount importance. Attention must be paid to ensuring the bridge and both sidewalks on the movable span and its adjacent approaches are clear of all motorists and pedestrians before and during test operations.*
- *The bridge's safety interlocking system is designed to protect the public and various components of the bridge during operation by only allowing steps of operation to be performed in a specific sequence. For example, the brakes holding the movable span closed cannot be released until all traffic gates and the barrier gate have been lowered;*

power to the drive motors cannot be applied until the brakes have been released and the toe locks have been withdrawn, etc. Manual bi-passes on the control console allow portions of the interlocking system to be overridden for emergency short term situations such as the failure of a position sensing device that would otherwise render the bridge inoperable. When testing interlocking system bypasses during inspection, special attention must be paid to potential risks it may present to highway traffic, pedestrians and bridge equipment. After testing bypasses, the interlocking system must be immediately restored to its fully functional condition.

- *Nonredundant Steel Tension Member (NSTM) bascule girders.*
- *Operate bridge as many times as necessary to observe operation of all items listed on the movable bridge inspection forms.*

Inspection Methods and Equipment:

This movable inspection was separate from the routine and NSTM inspection. The following inspection equipment should be on-hand for inspecting this movable bridge:

- *Flotation vest for inspection of the timber fender system and navigation lights*
- *Reflective vest for inspection of areas on and adjacent to the roadway*
- *Fall protection for working in areas that present a potential fall hazard.*
- *Flash light or helmet mounted head lamp for inspecting components in areas with limited lighting such as the counterweight pit, electrical cabinets and interior of machinery gear box.*
- *Camera with zoom and flash capability*
- *A clip board for neatly recording information on the inspection forms and securely them during the inspection. Alternately, an electronic tablet can be used to electronically record and secure inspection data.*
- *Disruptions to bridge users during test operations should be minimized by planning in advance what elements of the bridge's mechanical, electrical and structural can be concurrently observed and inspected during each test operation.*

Inspection Team Member Roles:

The Inspection Team Leader is in charge of the inspection team and is responsible for planning, preparing for, performing, and documenting the overall movable bridge inspection. All inspections must be conducted under the supervision of the Inspection Team Leader. The Inspection Team Leader is also responsible for performing quality assurance, verifying all key elements of each of the movable bridge's functional systems have been inspected and proper condition states have been assigned and entered for them on the inspection forms. The Inspection Team Leader must supervise and coordinate work performed by the

Lead Inspectors for each functional discipline and ensure they fully complete in a timely manner all required inspection documentation.

Lead Inspectors are responsible for inspecting the specific mechanical, electrical, hydraulic and structural systems of the movable bridge for which they have experience and technical expertise. A single individual may serve as the Lead Inspector for more than one bridge system discipline if they have the necessary training and relevant recent experience in inspection, design, maintenance, and/or construction of movable bridges. The experience must be in the specific system discipline for which the individual has the responsibility to lead. Each Lead Inspector must directly oversee on-site all work performed by any supporting staff assisting them during the inspection.

Planning and Preparing for the Inspection:

Adequate preparation must be performed in advance of the movable bridge inspection. This will help ensure a complete, comprehensive and efficient inspection can be completed during one site visit. A list of all components of each functional system to be inspected should be reviewed.

Steps must be taken to ensure all components will be accessible for inspection and for observation during test openings of the bridge. Access methods and associated equipment for achieving arms-length reach of all components to be inspected must be identified and mobilized.

Any specific traffic control needs for the inspection must be arranged. Any traffic lane reductions for accommodating an under-bridge access unit must be planned. Specific time-of-day restrictions during peak traffic periods that would restrict when operational testing can be performed must be identified and conveyed to the inspection team so they can schedule their inspection tasks around bridge planned bridge openings accordingly.

Arrangements must be made to have the bridge thoroughly cleaned before the inspection so that any dirt and debris is removed that would otherwise inhibit making of complete visual observations and taking of precise measurements.

Inspection of functional system components during bridge test openings is essential for a thorough and complete inspection. Arrangements must therefore be made to ensure an operator will be present during the inspection. Prior to the day of the inspection, if the bridge has not been recently opened, it should be operated through a full opening and closing cycle to ensure all of its systems will be functional for test operations during the inspection. The functionality of any backup power system or alternate incoming power source should also be verified in advance of the inspection so its operation can be observed during the inspection.

Movable bridge inspection planning must address provisions for inspector safety. All applicable personal protective equipment needed during the inspection must be identified and secured to ensure it will be on site. Such equipment may include reflective vest, safety boots, hard hat, eye protection, fall protection and flotation vests. The protocol for communicating between team members and with the bridge operator must be identified and any equipment needed to support it must be identified to ensure it will be on site.

In planning the inspection, the inspection team leader must facilitate coordination between lead discipline inspectors to ensure they understand how they must interface with one

another during the inspection. This coordination should include reviewing and scheduling what each inspector will concurrently do during test openings. This will serve to maximize what can be inspected during each operation and thereby limit how many traffic-disrupting openings are needed to complete and a comprehensive inspection. The inspection team leader should compile inspection reports and any measurement data from previous inspections of the bridge and facilitate each discipline lead inspector reviewing previous findings for the specific functional systems they will be inspecting.

All potentially useful reference materials to have available on-site during the inspection must be identified. Examples of these include plans of the bridge including those for its specialty functional systems and completed forms from previous inspections.

A list of all tools that may potentially be needed for the inspection should be prepared and those tools secured before the day of the inspection.

Inspection forms for recording all inspection data and observations should be printed for use during the inspection.

Performing the Inspection:

The names of the inspection team leader and lead inspectors and their qualifications for each functional system shall be recorded on the inspection forms.

An on-site safety meeting must be conducted with all members of the inspection team at the start of the inspection. This will promote a clear understanding of communication protocol between each inspection team member. The bridge operator must be included in this meeting to confirm communication procedures between the operator and inspectors and review procedures for ensuring all inspection staff are in safe locations prior to initiating any bridge openings.

The movable bridge inspection must be performed in a thorough, comprehensive and complete manner. The elements listed in the bridge inspection forms should be used as a guide to assure complete inspections. Information shall be entered for all items listed on the forms. All components of each functional system of the bridge shall be inspected during each inspection. If, for any reason, a specific component cannot be inspected, it must be noted on the bridge inspection form. Any items on the form not applicable to the bridge shall be noted with “N/A” to ensure nothing was accidentally overlooked.

Full documentation of the inspection must include high quality photographs. They should be taken to document all issues observed during the inspection. All photographs shall be referenced on the inspection forms. When there is more than one of the same elements on the bridge, the photograph’s reference should clearly say which one it is.

Functional testing of all components during bridge operation must be performed. Where there is more than one of the same components on the bridge, each one should be separately observed under operation.

The bridge operator should be interviewed as part of the inspection to obtain their input regarding any issues they may be aware of or have witnessed during bridge openings. Information provided by the operator about any issues should be recorded on the inspection forms.

An important part of the movable bridge inspection is identifying and documenting any short term repair needs that may be needed for the bridge's functional systems.

3.1.3 Inspector Qualifications

Movable bridges are large, complex, pieces of machinery. Each part of a movable bridge has a relationship to, and must interact with, many other parts. In order for a movable bridge to operate efficiently and to provide a long and serviceable life, all parts of the bridge must be maintained properly. Proper maintenance requires that all functional systems (including electrical, mechanical, hydraulic, and structural) must be inspected and evaluated by personnel who are sufficiently experienced in that line of work.

The personnel who perform inspection of movable structures should have experience beyond that which may be sufficient for the inspection of fixed bridges. This may seem obvious for the inspection of electrical, mechanical, and hydraulic equipment. It is also true for the inspection of the structural components of a movable structure. As with a fixed bridge, the structural members of a movable bridge must safely withstand the stresses imposed by the dead loads, live loads, and other loads typically encountered. In addition, many of the primary and secondary structural members should also withstand the stresses imposed by the operating equipment and the movement of the bridge. It takes significant first-hand experience on movable structures and with the applicable codes to adequately understand these relationships, evaluate the conditions of the various components, and to recommend appropriate courses of action.

Inspection Team Leader

The lead person (Inspection Team Leader) designated to manage the inspection of a movable bridge should be familiar with all functional systems that are a part of that bridge. The Inspection Team Leader should have the knowledge and experience to understand and evaluate all of the inspection findings. The total experience should be broad in nature, so that the individual understands the inter-relationships inherent in movable structures and inherent for the specific type/style of movable structure being inspected. The Inspection Team Leader must meet the National Bridge Inspections Standard's (NBIS) definition of an inspection team leader.

The Inspection Team Leader is in charge of the inspection team and is responsible for planning, preparing for, performing, and documenting the overall movable bridge inspection. All inspections must be conducted under the supervision of an Inspection Team Leader. The Inspection Team Leader is also responsible for performing quality control, verifying all key elements of each of the movable bridge's functional systems have been inspected and proper condition states have been assigned and entered for them on the inspection forms.

The inspection team leader for movable bridges shall meet the following minimum requirements:

- Be certified by the State of Wisconsin as a Movable Bridge Inspection Team Leader

- Completed the NHI 3-day Nonredundant Steel Tension Member Inspection Techniques for Steel Bridges course (if responsible for leading the bi-annual NSTM inspection).

- Completed the 10-day FHWA-NHI Safety Inspection of In-Service Bridges course

• Acquire a minimum of 3 years Inspection experience during a 10 year period performing a combination of the following services for movable structures, having similar type or style to that being inspected.

- Design of new movable structures or rehabilitations of existing ones
- Construction oversight of new movable structures or rehabilitation of those existing
- Maintenance and/or troubleshooting of movable bridges
- Movable bridge safety inspections
- Previously acting as inspection team leader during safety inspections of movable structures.

The following qualification is also strongly recommended:

• Attended the Heavy Movable Structures (HMS) bi-annual symposium or similar, for continued information exchange on movable structures

The Inspection Team Leader can also serve as a Lead Inspector if that person has the required qualifications in the functional discipline that they will be responsible for inspecting.

Lead Inspectors:

The inspection teams should include structural, electrical, mechanical, and when appropriate, hydraulic Lead Inspectors. A single individual may serve as Lead Inspector in more than one of the areas if the individual possesses the necessary experience. Each Lead Inspector should have experience in the inspection, design, maintenance, or construction of sufficient movable structures of the type/style being inspected to understand their function and operation; this experience should include past inspection experience of movable structures. This experience should be in the specific systems for which the individual is leading the inspection, e.g., the Lead Electrical Inspector should have this experience in electrical systems. Each Lead Inspector must supervise and monitor any work performed by anyone assisting in the efforts.

Lead Structural Inspector Qualifications:

The Lead Structural Inspector is the individual who will lead the inspection of the structural elements which are unique to movable bridges. The Lead Structural Inspector shall meet all the following minimum requirements:

- Have a minimum of 5 years of bridge inspection experience. A P.E. license in civil/structural engineering can substitute for 2 years of experience.
- Have a minimum of 3 years of experience in the design, inspection, or maintenance of movable bridge structures.
- Have experience with at least 3 different movable bridges in the last 5 years.
- Have sufficient training/experience to be able to perform the following:
    - Safely access and navigate between various locations on and near the movable span.
    - Evaluate the movable span during operation.

- Evaluate bridge balance and recommend method of counterweight adjustments.
- Inspect the condition of the counterweights, rack frames, trunnion supports, breaks, and supporting members.
- Evaluate unintended movement of rack frames and trunnion supports.
- Measure clearances and evaluate proper alignment and seating of bridge when in closed and various open positions.
- Evaluate and recommend improvements to access systems.
- Inspect and recommend repairs to operator house structural systems including roof, windows and doors.
- Inspect and recommend repairs to drainage and weatherproofing systems.
- Identify and evaluate any other portions of the structure not covered by routine or NSTM inspections.

Submit qualifications to BOS for approval. A person may serve as both the Inspection Team Leader and Lead Structural Inspector for a movable bridge only if they meet the qualification requirements for both roles.

Lead Electrical Inspector Qualifications:

The Lead Electrical Inspector is the individual who will lead the inspection of the electrical system elements of the movable bridge. The Lead Electrical Inspector shall meet all the following minimum requirements:

- Have a minimum of 5 years of electrical and control equipment inspection experience. A P.E. license in electrical engineering can substitute for 2 years of experience.
- Have a minimum of 3 years of experience in the design, inspection, or maintenance of movable bridge electrical systems.
- Have experience with at least 3 different movable bridges in the last 5 years.
- Have sufficient training/experience to be able to perform the following:
  - Test and evaluate power distribution systems, including transformers, switchgear, and circuit protection devices.
  - Understand and evaluate motor control systems, including variable frequency drives (VFDs), thyristor drives, secondary resistance, soft starters, and relay-based control systems.
  - Identify and diagnose issues related to electrical power quality, including voltage imbalances and overloaded equipment.
  - Understand the sequence of operation for each style of movable bridge.
  - Troubleshoot bridge control systems, including programmable logic controllers (PLCs), control relays, and human-machine interfaces (HMIs).
  - Inspect and evaluate bridge lighting systems, including navigation lights, roadway lighting, and safety/emergency lighting.
  - Perform insulation resistance testing.
  - Safely perform interlock testing.
  - When load testing is required, perform load testing and evaluating electrical system performance under operating conditions.

Submit qualifications to BOS for approval. Note: it is very unlikely that a structural or mechanical inspector will also qualify as a Lead Electrical Inspector.

Lead Mechanical Inspector Qualifications:

The Lead Mechanical Inspector is the individual who will lead the inspection of the mechanical system elements of the movable bridge. The Lead Mechanical Inspector shall meet all the following minimum requirements:

- Have a minimum of 5 years of machinery inspection experience. A P.E. license in mechanical engineering can substitute for 2 years of experience.
- Have a minimum of 3 years of experience in the design, inspection, or maintenance of movable bridge machinery.
- Have experience with at least 3 different movable bridges of the type being inspected (i.e. trunnion bascule, Scherzer, swing, vertical lift) in the last 5 years.
- Have sufficient training/experience to be able to perform the following:
    - Inspect and record measurements for alignment, backlash and wear of open gearing
    - Inspect and report on conditions of enclosed reducers
    - Measure sleeve bearings and report on the clearances within and outside of tolerance
    - Identify and record problematic issues with roller bearings
    - Inspect and make adjustments to thruster brakes as needed.
    - Inspect locking mechanism tolerances and record data.
    - Inspect and evaluate bridge seating operations

Submit qualifications to BOS for approval. Note: it is very unlikely that a structural or electrical inspector will also qualify as a Lead Mechanical Inspector.

Lead Hydraulic Inspector Qualifications

The Lead Hydraulic Inspector is the individual who will lead the inspection of the hydraulic system elements of the movable bridge. The Lead Hydraulic Inspector shall meet all the following minimum requirements:

- Have a minimum of 5 years of hydraulic equipment inspection experience. A P.E. license in mechanical engineering can substitute for 2 years of experience.
- Have a minimum of 3 years of experience in the design, inspection, or maintenance of movable bridge hydraulic systems.
- Have experience with at least 3 different hydraulically operated movable bridges in the last 5 years.
- Have sufficient training/experience to be able to perform the following:
    - Test hydraulic oil and understand when filtering or replacement is necessary
    - Understand flow control valves including proportional control valves and servo valves and have the ability to oversee adjustments of these valves
    - Identify when there are thermal issues within the hydraulic system
    - Understand the sequence of operation and the various pressure/flow requirements for each of the systems
    - Troubleshooting hydraulic systems
    - Inspect and evaluate bridge seating operations as related to the hydraulic system
    - When balance testing is required, perform balance testing with pressure transducers

Submit qualifications to BOS for approval. A person may serve as both the lead mechanical and lead hydraulic inspector only if they meet the qualification requirements for both

roles. Note: it is very unlikely that a structural or electrical inspector will also qualify as a Lead Hydraulic Inspector.

Other individuals:

Other individuals may assist in the inspection. It is recommended that each of these inspectors have experience in the inspection, design, maintenance, or construction of sufficient movable structures of the type/style being inspected to understand their function and operation; this experience should include past inspection experience of movable structures. However, if their experience level is less than the recommended experience, their efforts should be closely supervised and monitored by the Lead Inspector or another inspector possessing at least the minimum recommended experience for a Lead Inspector.

All members of the inspection team, their roles and their qualification shall be recorded in the area of the inspection form labelled “team members”.

3.1.4 Inspection Safety

Nothing is more important during a bridge inspection than safety. This includes safety of the public, the bridge operating and maintenance staff, and the bridge inspection team. The inspectors need to be alert and aware of the work environment and safety issues. They should not rely solely upon bridge operating and maintenance staff to create a safe environment. The team leader should take an active role in arranging pedestrian and vehicular traffic control and de-energizing electrical equipment. For basic safety requirements, refer to Part 1, of this manual. Also, potential personnel and public safety hazards are discussed in Chapter 1.3 of the American Association of State Transportation and Highway Officials (AASHTO) Movable Bridge Inspection, Evaluation, and Maintenance Manual.

Before beginning a movable bridge inspection, a test opening should be performed to determine if the movable span is operable and if there are any serious defects which need special consideration or that would preclude a safe inspection. Such issues should be resolved with the bridge maintenance and design groups if possible. While the inspection is underway the inspector should be cognizant of the mechanisms which are intended to stabilize the movable span. No components should be disconnected that would thereby create an unsafe condition. After completion of the inspection, the inspector should check the operating equipment to make certain that it is energized and ready for operation. A test opening should then be made in order to verify that the bridge is in at least as good a condition as before the inspection began.

Electrical and hydraulic equipment presents special hazards to the inspection team. Before handling or inspecting any equipment the inspector should verify that the equipment is de-energized. This can be done by using local disconnect switches or turning off protection devices (such as circuit breakers) electrically upstream of the equipment. If the disconnect switch or protection device is not within eyesight of the equipment, an official site lockout procedure should be implemented. This is required to prevent someone who is unaware of the work underway from energizing a device while it is being inspected. The minimum requirements for such a procedure include notifying all personnel of the inspection work by posting signs, physically preventing the device from being returned to operation (such as pad-locking a switch), and encouraging the inspector be alert.

The inspector should be aware that a panel or device may be powered from a second utility source or a stand-by generator. Therefore removing power from a single source may be insufficient to de-energize a panel. An inspector should always test the equipment with a voltmeter or similar power detector to determine whether the equipment is safe to inspect.

Special attention should be given to all rotating or moving machinery. Portions of the inspection will require the inspector to observe the machinery and equipment while in operation. Because any rotating equipment can be a hazard, the inspector should be very careful not to come into contact with the machinery or let hair or loose clothing be caught in the machinery.

Extreme caution should be used when inspecting hydraulic equipment. Never loosen or disconnect any hydraulic system component before verifying that the component is not under pressure. Fluid samples should be taken only from the correct sampling ports. Any spilled hydraulic fluid should be cleaned up immediately.

With older bridges the inspector may encounter electrical panels and enclosures that may be difficult to open and even harder to properly seal after inspection. The inspection of a bridge should not be destructive and the inspector should always take care to seal all panels and equipment properly to prevent later damage. Before attempting to open any panel, the inspector should determine if the panel is warped or damaged. If the equipment is damaged, do not inspect it unless there is a maintenance crew readily available to repair or replace the damaged equipment. Otherwise, the inspector should note in his report that the equipment is inaccessible due to damage. Leaving a panel open, or cracked, can expose the bridge wiring to corrosion or damage and expose people to electrical contact which could lead to permanent injury, disability, or death.

Unless otherwise instructed, the inspector should not attempt to repair or modify the bridge equipment. Accurately recording any deficiencies and notifying the appropriate maintenance authority immediately will enable the repairs to be made in a documented and orderly manner.

The inspector should always use appropriate safety equipment while on the bridge or roadway. This includes, at a minimum, the use of a hard hat, safety glasses, safety reflecting vest, appropriate footwear, and clothing that is not loose. Life vests and safety harnesses shall be used when inspecting over the water or areas with a fall hazard, as required by Occupational Safety and Health Administration (OSHA) and US Coast Guard regulations. Each inspector is responsible for his own safety and should take all necessary actions required to minimize risk.

Lock Out / Tag Out Procedure

Prior to the inspection of a movable bridge inspection, it is imperative that the Bridge Operator is contacted and acknowledges that inspectors will be assessing the condition of all the structural and mechanical components. If the bridge controls for the structure on not located on site, it is recommended that the control box for that structure undergo the lock out/ tag out procedure to ensure that a third party unaware of inspectors within the structure do not accidently operate the machinery creating a potentially life threatening situation. This procedure is an industry recommended procedure to reinforce safety in potentially hazardous areas.

More information on this program can be viewed at:

https://www.osha.gov/dts/osta/lototraining/

The operation of the movable components of the bridge during the inspection should only occur after confirming with the inspectors to do so. Similarly, the inspectors may require the bridge to be operated in order to see and hear any potential defects during the opening/closing procedure. Clear communication between the Bridge Operator and Inspectors is of the utmost importance. Failure to do so may be life threatening.

3.1.5 Emergency Notification

Just as serious structural deficiencies may lead to localized failures or collapse of a bridge, mechanical and electrical defects can cause a movable span to become unstable in either, or both, the closed or open position. In Part 1, procedures are detailed for reporting serious structural defects and the actions to be taken to limit live loading or temporarily close the bridge to traffic (both on the bridge and in the waterway). The Critical Findings Procedure is used when conditions require emergency notification. While the process is nearly identical for movable bridges as it is for typical bridges, the Movable Bridge inspection team leader must also contact the U.S Coast Guard. The Coast Guard is the governing body of navigable waterways and must be informed when a Critical Finding is discovered. Notification must be made regardless of the amount of daily boat traffic (or lack thereof) under the bridge.

When a critical finding is observed the movable bridge Inspection team leader shall notify the Coast Guard Sector Lake Michigan at (414) 747-7100 or Coast Guard Sector Upper Mississippi River at (314) 269-2500.

Serious defects of the span drive or stabilizing machinery (see Chapter 5) should be reported in a manner similar to structural deficiencies. A movable span may be unstable in any position (open, closed, or in-between) due to defects in the span drive or stabilizing machinery. Much depends on the type of movable bridge and redundancy of the overall system. Because of the variations in movable bridge design, no firm rules can be set down. It is a matter of analyzing each situation. Following are examples of some situations that occurred on movable bridges:

1. Bridge Type: Single-leaf, simple trunnion bascule.

   Type of Drive: Type 1-AD, all brakes on motor shaft.

   Defect: Cracks in both pinion shafts emanating from corners of keyways. Very toe-heavy span imbalance for all opening angles. Improper brake setting creating much vibration when stopping.

   Analysis: Because span drive contains differential and all brakes are on input shaft the drive is non-redundant. If one pinion shaft fails the toe-heavy leaf would descend uncontrolled.

   Critical Findings classification – Severe.

2. Bridge Type: Double-leaf rolling lift (Scherzer) with rear floor break far behind center of roll.

Type of Drive: Type 1-ND with hydraulic motor and disc brake on motor shaft. Drive shafting is not dependably “wound-up” when bridge is closed.

Defect: Tail locks cannot be driven.

Analysis: Because rear floor break is at a far distance behind the center of roll, live load can be applied to the leaf behind the center of roll. That load creates a moment tending to open the leaf, which is resisted by the tail locks and the wind-up of the span drive. Because the tail locks cannot be driven to support the leaf, and the drive shafting is not prestressed, an appreciable live load on the rear of the leaf would cause the leaf to rotate (be unstable) while under traffic until the brake and shafting rotate sufficiently to provide resisting torque (assuming the braking capacity is adequate). Shore rear of leaf to prevent downward motion of tail behind the center of roll when subjected to live load.

Critical findings classification: Urgent.

3. Bridge Type: Tower drive vertical lift.

   Type of Drive: Tower drive

   Defect: Cracks in trunnion shafts at the fillet in shaft near face of counterweight sheave. Bridge is not to be operated until remedial measures are determined based on fatigue stress analysis.

Critical findings classification: Urgent.

The above three examples for machinery defects illustrate that a knowledge of the structural/mechanical interaction involved in the load path is necessary in assessing the need for machinery defect notification and repairs.

3.1.6 Classification of Movable Bridges

The motions of all movable spans are a combination of rotation and translation. The difference between the types results from the selection of the axes of displacement. Movable spans may be categorized on the basis of the displacements and axes of displacement as follows:

Displacement	**Movable Bridge Type**
Rotation about a fixed horizontal axis	Trunnion Bascule
Rotation about a fixed vertical axis	Swing
Translation along a fixed horizontal axis	Retractile
Translation along a fixed vertical axis	Vertical Lift
Rotation about a horizontal axis that simultaneously translates	Rolling Lift Bascule

All the movable bridge types listed above have sub-types, some of which are described subsequently.

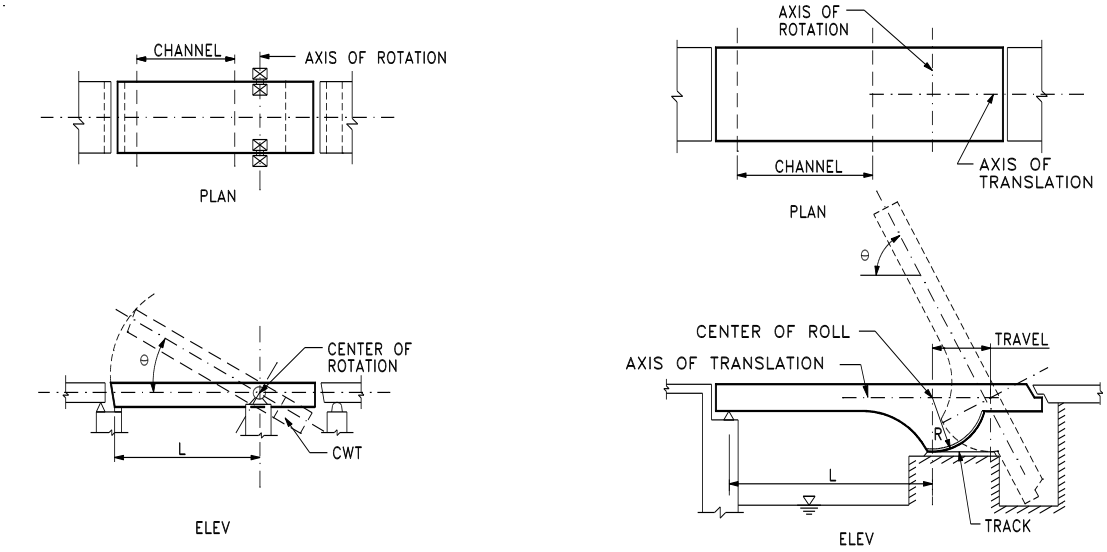
Figure 3.1.5-1 (a) depicts a simple trunnion bascule bridge with a counterweight fastened to the bascule girders. Usually the trunnions are fixed to the bascule leaf and rotate in bearings mounted on the bascule pier. The leaf rotates about the fixed horizontal axis, similar to a "see-saw." A counterweight may fully or partially balance the leaf structure. There are many sub-types of trunnion bascules, distinguished by the location and amount of the counterweight, as will be described later.

Rotation about a horizontal axis that simultaneously translates distinguishes the rolling lift bascule (sometimes called a rolling lift bridge). The most common rolling lift bascules are those based on concepts in a series of patents granted to William and Albert Scherzer starting in 1893. Almost all rolling lift bascules are nominally balanced by counterweights fixed to the bascule girder. The counterweights may be located above the deck, or below it, or outboard of the moving span. Figure 3.1.5-1 (b) depicts a single-leaf rolling lift bascule bridge with the counterweight below the deck.

Most early 20th century movable bridges were of the swing type, with rotation about a fixed vertical axis as shown in Figure 3.1.5-1 (c). The swing bridges are often termed draws (the swing span or draw length = L). If L1 equals L2 the draw is said to be symmetrical, common for most swing bridges, otherwise it is known as a "bobtail" swing span. Swing bridges have sub-types based on the type of rotational bearing at the pivot pier.

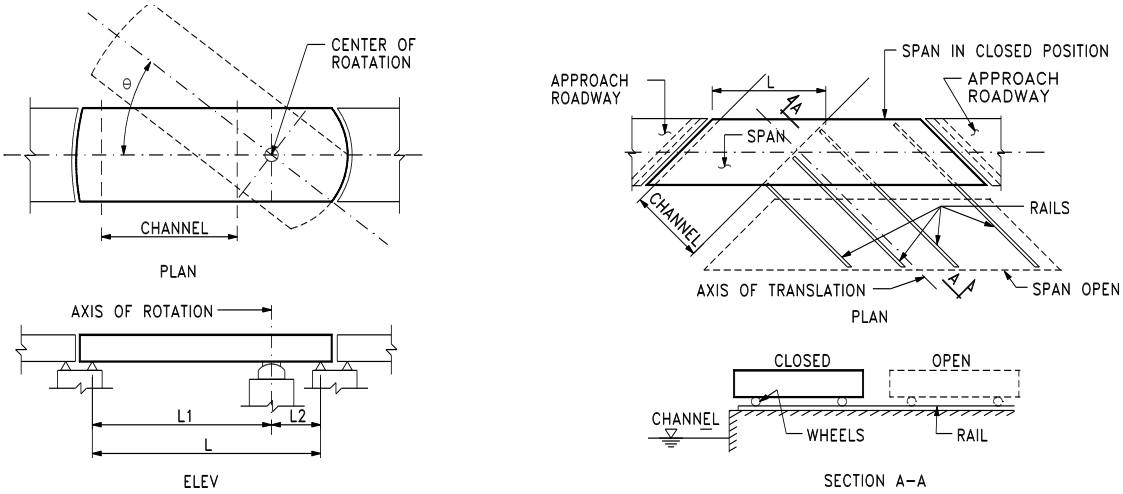
Retractile bridges translate horizontally, usually along a straight path which may be normal to or at an angle to the channel, as shown in Figure 3.1.5-1 (d). If the translation is at an angle to the channel (approximately 45 degrees), as in the figure, the movable span does not have to be lifted or depressed in order to clear a path for motion.

Figure 3.1.5-1 (e) depicts a span that moves vertically along a fixed vertical axis in order to obtain additional vertical clearances for navigation. It is termed a vertical lift bridge. There are many sub-types, distinguished mainly by the location of the span drive machinery. The modern vertical lift bridge design is credited to Waddell who designed the South Halsted Bridge across the Chicago River, which was completed in 1895 along the lines of his U.S. Patent.



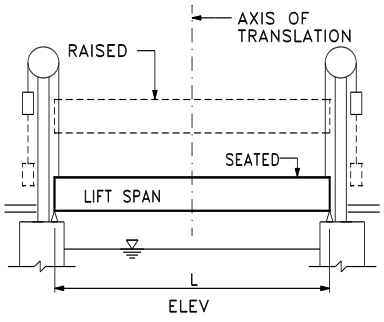
(a) SIMPLE TRUNNION BASCULE

(b) ROLLING LIFT BASCULE



(c) SWING BRIDGE

(d) RETRACTILE BRIDGE



(e) VERTICAL LIFT BRIDGE

Figure 3.1.5-1: Basic Motion of Movable Bridges.

3.1.7 Short History of Movable Bridges in Wisconsin

In the early years of this country's history, transportation was by foot, by horseback or horse drawn wagons, and by watercraft. Commercial transportation by water grew quickly and became the primary mode for shipping goods to market. In the second half of the 19th century a conflict arose between the expanding commercial navigation interests and the growing need for more and better land transportation. Fast growing cities, such as Milwaukee, needed more bridges so that people and goods could be moved from one side of a river to the other side. At the same time, fast growing railroads generally required relatively low level bridges to cross rivers. The U.S. Congress established navigation as the primary mode of transportation and prohibited the construction of bridges or other obstructions over navigable waterways. Only bridges authorized by an act of Congress could be constructed over a navigable waterway, and these had to be either movable or have sufficient height to allow the passage of vessels. To this day, at the intersections of land traffic and water traffic, water traffic has the primary right of way. Although an act of Congress is no longer required for the construction of a bridge, a rigorous permitting process is in place.

According to the 1994 National Directory - Heavy Movable Structures, Inc., Bridge Edition, there were 93 movable bridges in Wisconsin or on rivers bordering Wisconsin. Of these, 65 support highway or pedestrian traffic and 28 support railway traffic. The oldest of these bridges date from the beginning of the 20th century. Generally, the railway bridges are owned by private companies and the highway bridges are owned by the State of Wisconsin, the county, or local municipalities. The publicly-owned movable bridges in Wisconsin are listed in Figures 3.1.6-1 and 3.1.6-2.

Prior to 1900, most of the movable bridges were swing spans or vertical lift spans. More recently, most of the highway movable bridges constructed in Wisconsin have been bascule bridges. The most recent bridges include: North Emmber Lane in Milwaukee, State Street in Racine, Ray Nitschke Bridge (Main Street) in Green Bay, Kinnickinnic Avenue in Milwaukee, Eighth Street in Manitowoc, Eighth Street in Sheboygan, Maple-Oregon Street in Sturgeon Bay, Wisconsin Street in Oshkosh and 17th Street in Two Rivers.

Waterway	Miles From Mouth	Roadway Name	Location	Struct. No.	Type	Owner	Year Built
Fox Rvr.	1.58	USH-141/Main St.	Green Bay	B-05-0311	RL-B	City	1998
Fox Rvr.	1.81	STH-29/Walnut St.	Green Bay	B-05-0269 (2)	RL-B	State	1986
Fox Rvr.	2.27	STH-54/Mason St.	Green Bay	B-05-0134	ST-B	State	1973
Fox Rvr.	17.36	STH-96/Ferry St.	Wrightstown	B-05-0736	ST-B (Fixed)	State	1934
Fox Rvr. Cnl.	23.78	Wisconsin Ave./ Island St.	Kaukauna	B-44-0081	VL	City	1984
Fox Rvr. Cnl.	26.53	Island Park Trail	Little Chute	B-44-0287	ST-B	Village	2016
Fox Rvr. Cnl.	31.37	Lawe St.	Appleton	P-44-0719	ST-B	City	1954
Fox Rvr. Cnl.	31.74	Olde Oneida St.	Appleton	P-44-0724	ST-B	City	1959
Fox Rvr. Cnl.	37.52	STH-114/Tayco St.	Menasha	B-70-0097 (2)	RL-B	State	1993
Fox Rvr. Cnl.	37.91	Racine St.	Menasha	B-70-0001 (2)	RL-B	State	1952
Fox Rvr.	55.97	USH-45/Main St.	Oshkosh	B-70-0056 (2)	RL-B	State	1973
Fox Rvr.	56.22	Oregon/Jackson St.	Oshkosh	B-70-0011 (2)	RL-B	State	1956
Fox Rvr.	56.72	STH-44/Wisconsin St.	Oshkosh	B-70-0247 (2)	RL-B	State	2008
Fox Rvr.	58.01	STH-21/Congress Ave.	Oshkosh	B-70-0091 (2)	RL-B	State	1982
Manitowoc Rvr.	0.29	USH-10 WB/8th St.	Manitowoc	B-36-0142	RL-B	City	1995
Manitowoc Rvr.	0.43	USH-10 EB/10th St.	Manitowoc	B-36-0128 (2)	RL-B	City	1987
Menominee Rvr.	0.7	Ogden St.	Marinette	B-38-0016 (3)	RL-B	State	1973
Root Rvr.	0.31	STH-32/Main St.	Racine	B-51-0069	ST-B	City	1988
Root Rvr.	0.53	STH-38/State St.	Racine	B-51-0080	ST-B	City	1996
Sheboygan Rvr.	0.69	8th St.	Sheboygan	B-59-0154	NC-B	City	1995
St. Croix Rvr.	0.3	US-10	Prescott	B-47-0040	RL-B	State	1990
St. Croix Rvr.	23.4	SR-64	St. Joseph (Houlton) WI/ Stillwater MN	B-55-0919	VL	State	1931
St. Louis Rvr.	13.91	DM&IR Rwy./SR-39	Oliver WI	B-16-0755	Sw (fixed)	Railroad	?
Sturgeon Bay	2.80	STH-42/STH-57	Sturgeon Bay	B-15-0004 (3)	RL-B	State	1978
Sturgeon Bay	4.18	Maple-Oregon St.	Sturgeon Bay	B-15-023 (3)	RL-B	State	2008
Sturgeon Bay	4.30	Michigan St.	Sturgeon Bay	B-15-0100 (3)	RL-B	State	1930
East Twin Rvr.	0.48	17th St.	Two Rivers	B-36-0189	RL-B	City	2013
Wolf Rvr.	2.40	STH-116/Main St.	Winneconne	B-70-0913 (2)	RL-B	State	1934

RL-B = Rolling Lift Bascule (Scherzer Type) = -0002 Unit
ST-B = Simple Trunnion Bascule = -0003 Unit
NC-B = Simple Trunnion Bascule without counterweight
Sw = Swing span
VL = Vertical Lift bridge

Figure 3.1.6-1: Movable Bridges in Wisconsin Outside of Milwaukee – On Public Roadways.

Waterway	Miles From Mouth	Roadway Name	Location	Struct. No.	Type	Owner	Year Built
Kinnickinnic Rvr.	1.67	STH-32/ Kinnickinnic Ave.	Milwaukee	B-40-0591	ST-B	City	1999
Kinnickinnic Rvr.	1.78	1st St.	Milwaukee	P-40-0830	ST-B	City	1958
Menomonee Rvr.	1.08	Plankinton Ave.	Milwaukee	P-40-0539	ST-B	City	1984
Menomonee Rvr.	1.37	6th St. (N)	Milwaukee	B-40-0414(2)	ST-B	City	2002
So. Menomonee Cnl.	1.51	6th St. (S)	Milwaukee	B-40-0413(2)	ST-B	City	2002
Menomonee Rvr.	1.95	Emmber Lane	Milwaukee	B-40-0605 (6 & 7)	ST-B	City	2000
Menomonee Rvr.	2.14	16th St.	Milwaukee	B-40-0550	ST-B	City	1929
Milwaukee Rvr.	0.79	STH-32/Broadway	Milwaukee	B-40-0952	ST-B	City	1982
Milwaukee Rvr.	0.94	Water St.	Milwaukee	B-40-0548	ST-B	City	1987
Milwaukee Rvr.	1.21	St. Paul Ave.	Milwaukee	P-40-0523	VL	City	1966
Milwaukee Rvr.	1.28	Clybourn Ave.	Milwaukee	P-40-0868	VL	City	1968
Milwaukee Rvr.	1.37	Michigan St.	Milwaukee	P-40-0886	VL	City	1978
Milwaukee Rvr.	1.46	Wisconsin Ave.	Milwaukee	B-40-0488	VL	City	1975
Milwaukee Rvr.	1.61	USH-18 EB/ Wells St.	Milwaukee	B-40-0544	VL	City	1985
Milwaukee Rvr.	1.70	Kilbourn Ave.	Milwaukee	P-40-0881	ST-B	City	1929
Milwaukee Rvr.	1.79	USH-18 WB/ State St.	Milwaukee	B-40-0980	ST-B	City	1924
Milwaukee Rvr.	1.9	Highland Ave.(Ped.)	Milwaukee	B-40-907	VL	City	1998
Milwaukee Rvr.	2.06	Juneau Ave.	Milwaukee	B-40-757	VL	City	2012
Milwaukee Rvr.	2.12	McKinney Ave./ Knapp St.	Milwaukee	B-40-0062	VL	City	2004
Milwaukee Rvr.	2.29	Cherry St.	Milwaukee	P-40-0864	ST-B	City	1940
Milwaukee Rvr.	2.58	Pleasant St.	Milwaukee	B-40-0406	VL	City	1972

RL-B = Rolling Lift Bascule (Scherzer Type) = -0002 Unit
ST-B = Simple Trunnion Bascule = -0003 Unit
NC-B = Simple Trunnion Bascule without counterweight
Sw = Swing span
VL = Vertical Lift bridge

Figure 3.1.6-2: Movable Bridges In Milwaukee – On Public Roadways.

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3.2 BASCULE BRIDGES

3.2.1 Introduction

The origin of bascule bridges was the medieval draw bridge. These early bridges had no counterweights, and therefore, their size and utility was limited. Modern bascules almost always have a counterweight to balance the weight of the leaf about its support. Although a few bascule bridges were constructed in Europe prior to 1850, modern bascule bridge design can be traced to about 1893 and the construction of the Van Buren Street bridge in Chicago, a rolling lift bascule, and the Tower Bridge in London, a roller-bearing trunnion bascule.

Figure 3.2.1-1: 8th Street Bridge and 10th Street Bridge, Manitowoc.

Bascules are often selected for narrow to moderately wide channels where unlimited vertical clearance is required for navigation. They have been built in many configurations. Usually, a bascule leaf is comprised of two longitudinal bascule girders, or trusses, which support the roadway deck framing. A wide leaf may be supported by a multiplicity of girders. Each leaf of the Mason Street Bridge in Green Bay has six bascule girders and is rotated by one set of span drive machinery. Some bridges have leaves with four girders and two span drives per leaf. Such leaves are usually constructed as two, two-girder leaves that may be connected along adjacent longitudinal edges by structural diaphragms.

3.2.1.1 Counterweights

A bascule leaf is usually nominally balanced by a counterweight, which is fixed to the girders and located below the roadway. Such bascules are called balanced bascules. Counterweights are normally provided so that the bascule leaf is balanced about the main trunnion of a trunnion bascule or about the center of roll for a rolling lift bascule. There are two main purposes for utilizing counterweights. The first is to reduce the size of the mechanical systems that are required to operate the bridge. The second is to provide an increased margin of safety in the event that a control system or mechanical failure leads to a runaway condition.

Some bascules may also be constructed with little or no counterweight; they are termed partially balanced or unbalanced bascules. Unbalanced or partially balanced trunnion bascules are built at sites where the trunnions have to be located at an elevation not far above high water level and it is not feasible to construct a bascule pier with a counterweight chamber which can be kept dry or overhead counterweights are not architecturally acceptable. Because of the large forces required to equilibrate a leaf that is in a partially or fully open position, unbalanced or partially balanced bascules are moved by hydraulic cylinders. The larger machinery required by unbalanced leaves may be justified for short spans if the bridge can be configured so that the operating machinery is located at or above roadway level, thereby simplifying bascule pier construction and maintenance. However, if a large hollow pier is required to house the hydraulic machinery, then the rational for an unbalanced bascule with its higher first cost for machinery, higher operating and maintenance costs is questionable. An example of an unbalanced simple trunnion bascule is the South 8th Street Bridge in Sheboygan.

3.2.1.2 Single-Leaf Bascule

A single-leaf bascule has one leaf that spans the entire navigation channel. Only single sets of span drive and stabilizing machinery are required. However, the machinery may be significantly larger than that found on a double-leaf bascule spanning a channel of like width. Wind load moment on the bascule leaf, when it is in the open position, is approximately four (4) times the moment that would occur on each leaf of a double-leaf bascule.

3.2.1.3 Double-Leaf Bascule

A double-leaf bascule has two leaves that each spans half of the channel. The two leaves usually meet at the center of the navigation channel. The leaves of a double-leaf bascule will be approximately one-half the length of a single-leaf bascule leaf required to span the same channel. A double-leaf bascule requires span drive and stabilizing machinery for both leaves. A shear transfer device between mated girder ends of the two leaves when in the closed position is common to stiffen the deck and allow both leaves to share live load.

3.2.1.4 Deck Bascules

In deck bascules the bascule girders or trusses are located below the roadway surface. Most simple trunnion bascules are of this type. Rolling lift bascules and bascules with articulated counterweights are also common in this form.

3.2.1.5 Through Bascules

This category includes through-truss structures, such as Michigan Street in Sturgeon Bay. In this case the roadway deck is located at, or near, the bottom chord of the main trusses.

Also in this category are half-through girder structures, such as Eighth Street in Manitowoc and Tayco Street in Menasha. In this type of structure the bottom flanges of the floorbeams and the bottom flanges of the bascule girders are in nearly the same plane. The top flanges of the bascule girders are above the roadway deck.

3.2.2 Simple Trunnion Bascules

Figure 3.2.2-1 shows a single-leaf simple trunnion bascule (schematically) with the span drive machinery located on the bascule pier. The trunnions are usually inserted through the webs of the bascule girders or vertical/diagonal members of the trusses. The trunnions may rotate with the girders or the bascule girders may rotate about fixed trunnions. The leaves of trunnion bascules only rotate, they do not translate. Counterweights are usually fixed to the bascule girders so that the force required to move the leaf is minimized in all positions of opening. The balance principal requires that the trunnion axes should intersect (or nearly so) a line connecting the centers of gravity of the leaf and the counterweight.

The pinion shown in Figure 3.2.2-1 is the output of rotating span drive machinery and engages the rack to move the span. However, simple trunnion bascules may also be operated by hydraulic cylinders, which act linearly. The cylinders are mounted on the bascule piers and the piston rods are connected to the leaves in a manner such that extending or retracting the rods rotates the leaf. An example of a simple trunnion bascule with a hydraulic cylinder span drive is the South 8th Street Bridge at Sheboygan.

For the bridge of Figure 3.2.2-1, live load on the channel side of the axis of rotation is supported by the bascule girder spanning between the trunnion and the bearing on the rest pier. If the rear break in the deck is located on the shore side (rearward) of the trunnion, then when live load is between the trunnion and the rear break an uplift is created at the rest pier (if the live load moment exceeds the toe-heavy imbalance moment). This uplift is resisted by the span locks at the toe of the leaf, or tail locks, or by the locked span drive machinery.

Simple trunnion bridges are often built in double-leaf form, especially for highways. Midspan shear locks are necessary to keep the toes of the mating leaves in vertical alignment and transfer live loads between the leaves.

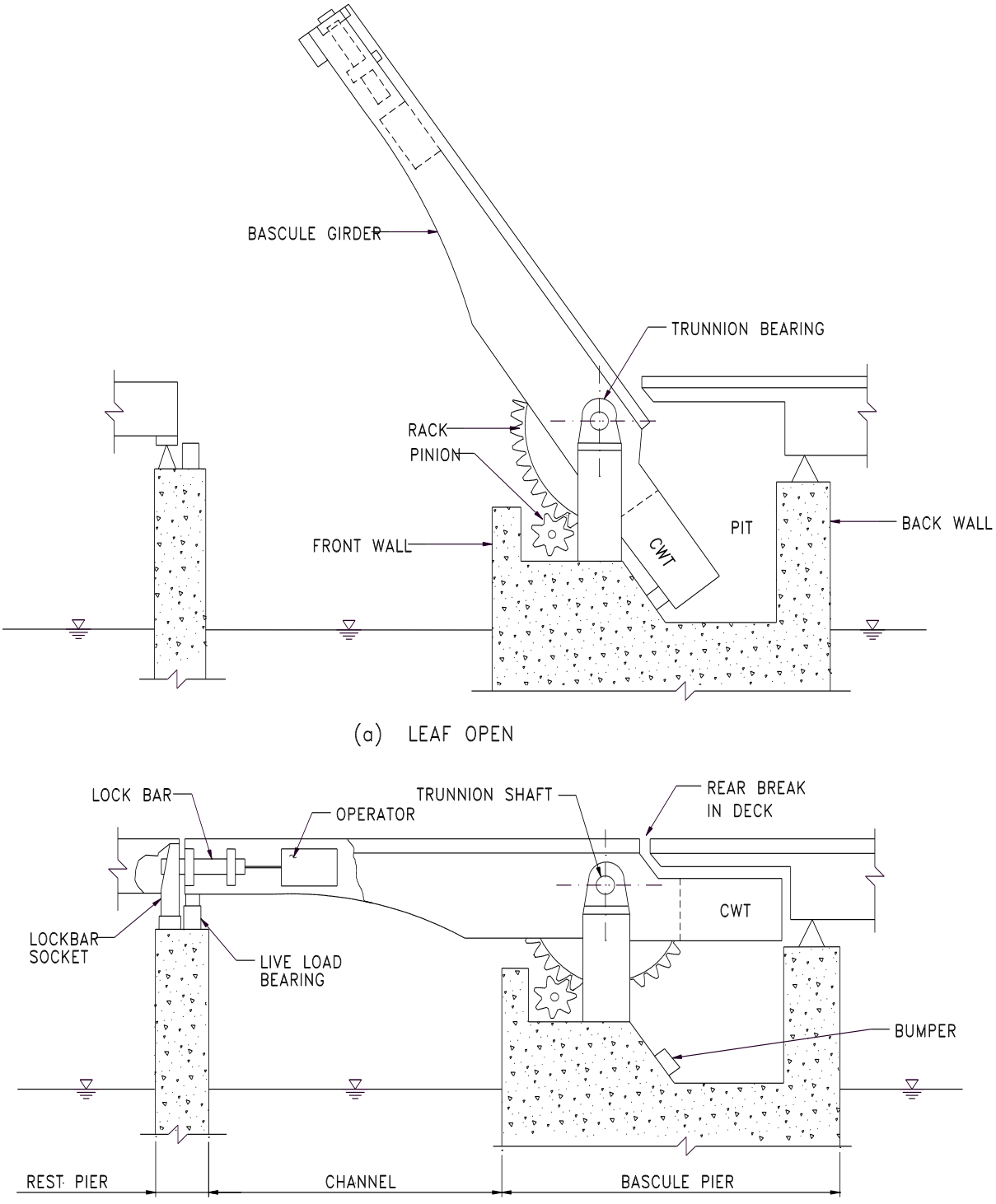


Figure 3.2.2-1: Single Leaf Simple Trunnion Bascule.

3.2.3 Rolling Lift Bascules – Scherzer Type

William Scherzer received a U.S. Patent for a rolling lift bascule in 1893, which became known as a rolling lift bridge, and that term is still used. The leaf motion is due to rotation about a horizontal axis, which simultaneously translates in the horizontal direction. Three common types of rolling lift bascules are depicted in Figure 3.2.3-1: View (a) the deck; View (b), the half-through plate girder (pony); and View (c), the through truss. Each of these types has variations.

Rolling lift bascule bridges of the Scherzer type are characterized by cylindrically curved parts of the bascule girders or trusses at the ends over the bascule piers. Because of their large size, these parts of the girders or trusses of the early Scherzer bridges were assembled from segments and the girders were called “segmental girders”, a term still in use. During bridge operation, the motion is rotation about an axis that translates. Each segmental girder may be viewed as a segment of a wheel. However, instead of rotating on an axle, these wheels are rigidly attached to the bascule leaf. As the wheels roll along the tracks, the bascule leaf rotates open or closed.

Figure 3.2.3-2 schematically depicts a single-leaf deck type Scherzer bascule. As the curved ends of the girders roll away from the channel the leaf tilts open to clear the channel. Slippage between the segmental girder treads and the tracks on which they ride is prevented by lugs or teeth that mechanically engage sockets. Typically, the protruding lugs are located on the track and the receiving holes or notches are in the segmental girder treads. Treads and tracks are described in more detail in Section 3.5.3.3 of this Part. The rack is shown located above the pinion, as is common on many newer Scherzers. However, on some older bridges the rack is located below the pinion.

Most double-leaf rolling lift bascule bridges in Wisconsin have center locks that are not mechanically operated. As with all double-leaf bascules, center locks are required to transfer vertical shear between the two leaves and to assure proper lateral and vertical alignment. Center locks are discussed in Section 3.5.3.3 of this Part.

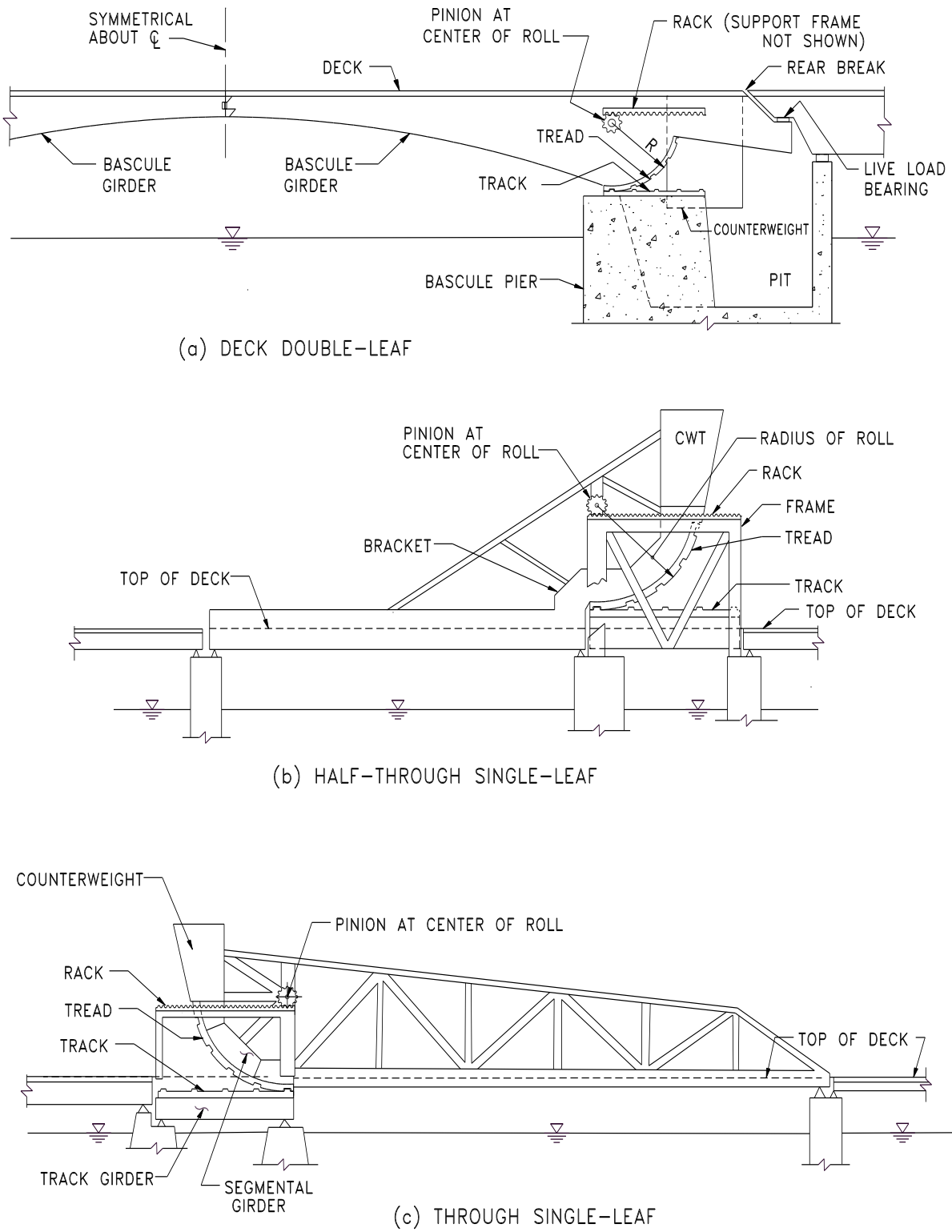


Figure 3.2.3-1: Types of Scherzer Rolling Lift Bascules.

The tracks on which the bascule leaf rolls may be mounted directly on the substructures or they may be mounted on flanking spans. As shown in Figure 3.2.3-3, the Racine Street Bridge over the Fox River Canal in Menasha is unique in Wisconsin in that the leaf at left rolls back over the bascule abutment while the leaf at right rolls onto the flanking span.

Figure 3.2.3-3: Double-Leaf Rolling Lift Bascule, Racine Street Bridge in Menasha. Note: Left leaf rolls on a bascule abutment, right leaf rolls on a flanking span.

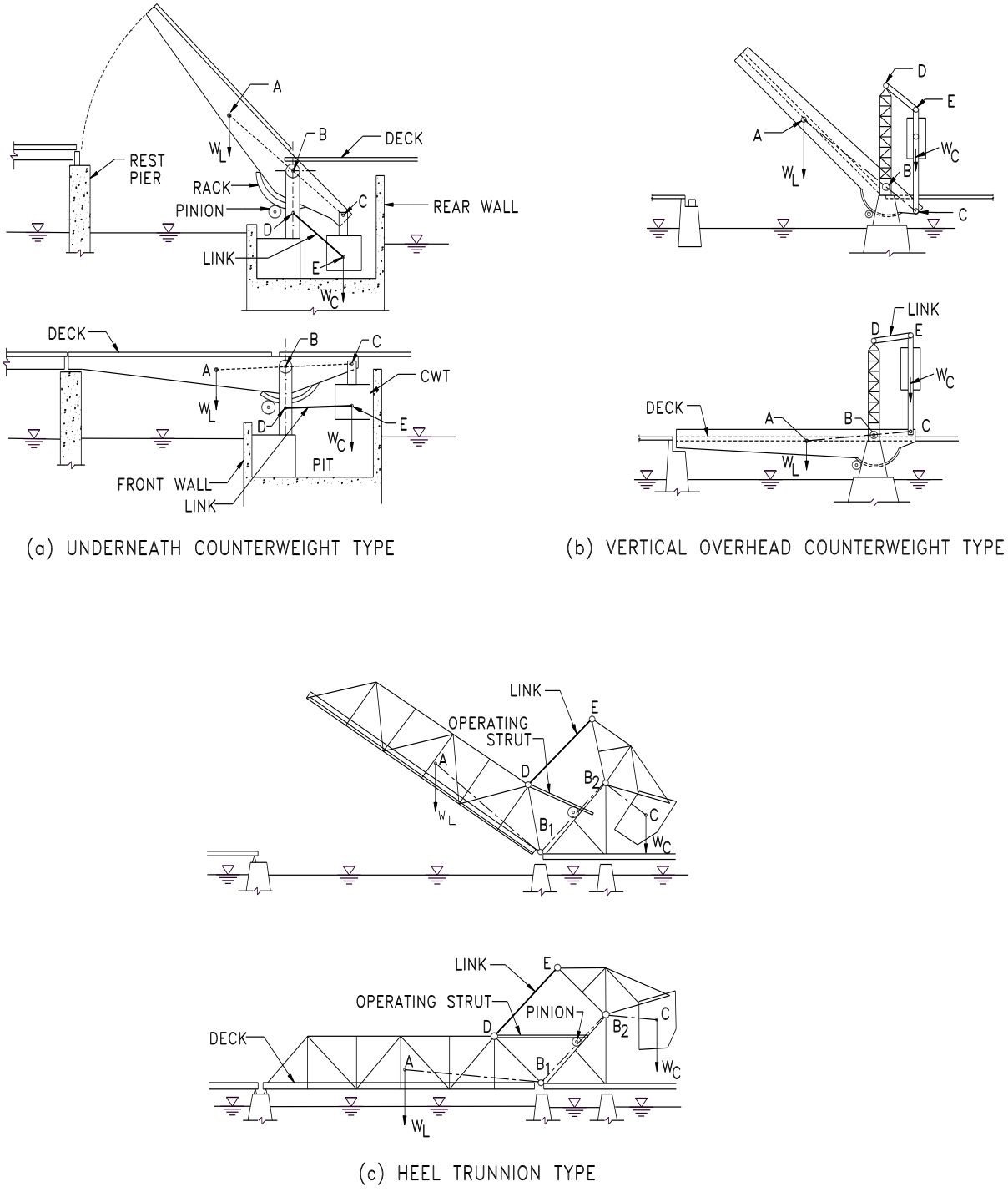
3.2.4 Articulated Counterweight Bascules – Strauss Type

Medium and long span simple trunnion deck bascules with the roadway profile close to the high-water level generally require bascule piers with counterweight pits. The walls of the bascule piers keep the water from entering the pits and so provide dry spaces for the counterweights as the bascule leaves rotate open. This is also the case for rolling lift deck bascules, but to a lesser extent. Counterweight pits can be avoided if the counterweights are located remote from the bascule girder, usually overhead.

Trunnion bascules may be balanced by counterweights that are not fixed to the bascule truss or girders. This concept is old, but U.S. Patents for bascules with remote counterweights were granted to Joseph B. Strauss around the year 1900 and bridges with his specific counterweight arrangements are often called Strauss bascules. Three common versions are shown in Figure 3.2.4-1. The distinguishing feature of all Strauss bascules is the parallelogram connecting the counterweight to the movable leaf. In all Strauss bascules the motion of the leaf is by rotation about a fixed horizontal axis.

In some situations, it is not the depth of the required pit but the front-to-back dimension of the pit that needs to be minimized, as in Figure 3.2.4-1 (a). In such cases articulating the

counterweight by suspending it from the leaf may be advantageous. In this section, three of the articulated counterweight trunnion bascules promoted by Strauss will be described.



(a) UNDERNEATH COUNTERWEIGHT TYPE

(b) VERTICAL OVERHEAD COUNTERWEIGHT TYPE

(c) HEEL TRUNNION TYPE

Figure 3.2.4-1: Types of Strauss Bascule Bridges.

3.2.4.1 Underneath Counterweight

Figure 3.2.4-1 (a) depicts a single leaf Strauss trunnion bascule with an under-deck counterweight. The counterweight hangs from two trunnions and the direction of the vertical axis C-E of the counterweight is maintained by the link between the counterweight and the trunnion tower. The need for this link has often been questioned. One argument is that at a small angle of opening the friction in the counterweight trunnion bearings may not permit the hanger to rotate such that the axis C-E remains vertical. As the angle of opening increases the moment applied to the bearing would increase and when it exceeded the bearing friction moment the counterweight would swing free. This motion would cause a dynamic load on the leaf which may interfere with control of the moving leaf.

Excessive friction in the counterweight trunnion bearings is often due to improper lubrication. The bearing friction induces a bending moment in the counterweight hanger, which produces repetitive bending stresses for which the hangers were usually not designed. Hangers on some bridges have failed in fatigue, resulting in collapse of the leaves.

3.2.4.2 Vertical Overhead Counterweight

Instead of hanging the counterweight from the bascule girder below the deck, the counterweight may be positioned overhead as shown in Figure 3.2.4-1 (b). This is advantageous at sites where the road profile is located close to the high water level in the channel and the bascule pier cost must be minimized.

Again the balance principle requires that the line joining the center of gravity of the leaf at A with the hinge C should intersect the trunnion axis at B. Line C-E will remain parallel to B-D if the line D-E is parallel to B-C. Lines B-D and C-E need not be vertical; they just need to be parallel.

Vertical overhead counterweight type Strauss bascules were built across small rivers in remote areas where appearance was not a primary consideration. Operation is usually by rack and pinion with drive machinery mounted on the pier or, at sites subject to flooding, at the deck level bracketed from the approach structure.

3.2.4.3 Heel Trunnion

A single-leaf Strauss heel trunnion with overhead rotating counterweight frame (rocker) is shown in Figure 3.2.4-1 (c). The geometrical figure B1DEB2 is a parallelogram and the center of gravity of the counterweight at C is located so that the line B2C is parallel to the line between the center of gravity of the leaf at A and the heel trunnion B1. As a result, the ratio between the leaf dead load moment about B1 and the counterweight moment about B2 remains essentially constant during rotation of the leaf.

The Strauss leaf rotates about the heel trunnion B1 in response to a force transmitted to the leaf by the operating strut (a rack is fastened to the strut) which is hinged at the top chord joint D and engages the output pinion of the span drive machinery mounted on the counterweight support frame. The trunnions B1 B2 and D E are heavily loaded during motion. The reaction at the heel trunnion B1 may reverse direction, depending on the proportions of the structure, and this effect should be considered in evaluation of the heel trunnion bearings.

3.2.5 Articulated Counterweight Bascules – ABT Type

Another type of heel trunnion bridge with an articulated counterweight is the ABT bascule patented by Hugo Abt in the early 1920s. Figure 3.2.5-1 shows an ABT bascule in the closed and the open position.

An ABT bascule is distinguished by a pair of A-frames located at the heel of the leaf. At the apex of these A-frames is an axis about which the counterweight rotates. On each A-frame, between the two legs, is an inclined track. Riding on these two tracks is a carriage that supports the operating machinery. Two links connect the counterweight to the machinery carriage, and two links connect the bascule leaf to the machinery carriage. As the machinery carriage travels along the tracks, it allows the counterweight to swing downward, thereby pulling the bascule leaf upward.

There is only one ABT bascule in Wisconsin. It is a railway bridge over the Manitowoc River in Manitowoc. As of this writing (2013), it is not in active service.

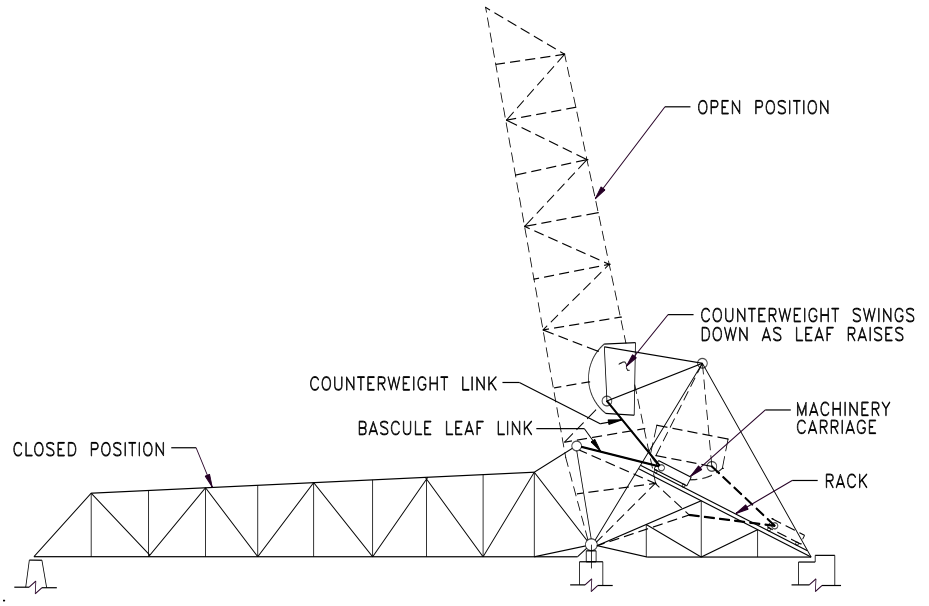


Figure 3.2.5-1: Through-Truss ABT Bascule.

3.2.6 Bascules Without Counterweights

Simple trunnion bascules have been built without counterweights, with only partial counterweights, and with counterweights that are partially or wholly submerged when the leaf is open. For all three cases, the machinery must be able to develop a greater torque to rotate the leaf than if the leaf were balanced by “dry” counterweight. The rationale to use a bascule without a counterweight is that it is more cost-effective to install and operate larger machinery than to build a deep watertight pit to accommodate a counterweight when the leaf is in the fully-open position, or when the aesthetics of an overhead counterweight are deemed to be inappropriate for a particular site.

Because of the larger operating torque required to rotate the leaf, hydraulic cylinders are often used to power the span. One machinery arrangement for a single leaf simple trunnion bascule without a counterweight is shown in Figure 3.2.6-1. It is a simplified diagram of the South 8th Street Bridge in Sheboygan.

Referring to Figure 3.2.6-1, the bascule leaf is comprised of two girders with deck framing. There is no counterweight. Without the stiffness that is normally provided by the massive counterweight another method is required to keep the two bascule girders in the same plane. The required stiffness is provided by a torque tube that extends from the exterior of one girder to the exterior of the other girder. Two crank plates are welded to the torque tube at each bascule girder - one on each side of the web. A trunnion shaft passes through the bottom of each pair of crank plates and rests in trunnion bearings. The bascule girder rotates about these bearings.

Force to raise the leaf during opening and to retard the leaf during closing is provided by two hydraulic cylinders per bascule girder. The cylinders are mounted and rotate about pivots. One of the problems that arises with cylinders that are mounted rearward of the main trunnions, and at a small angle with the horizontal, is the physical interference between the cylinders and the end of the deck in the open position.

The size of the machinery required for this bridge is much greater than would be required for a balanced bascule span of like dimensions.

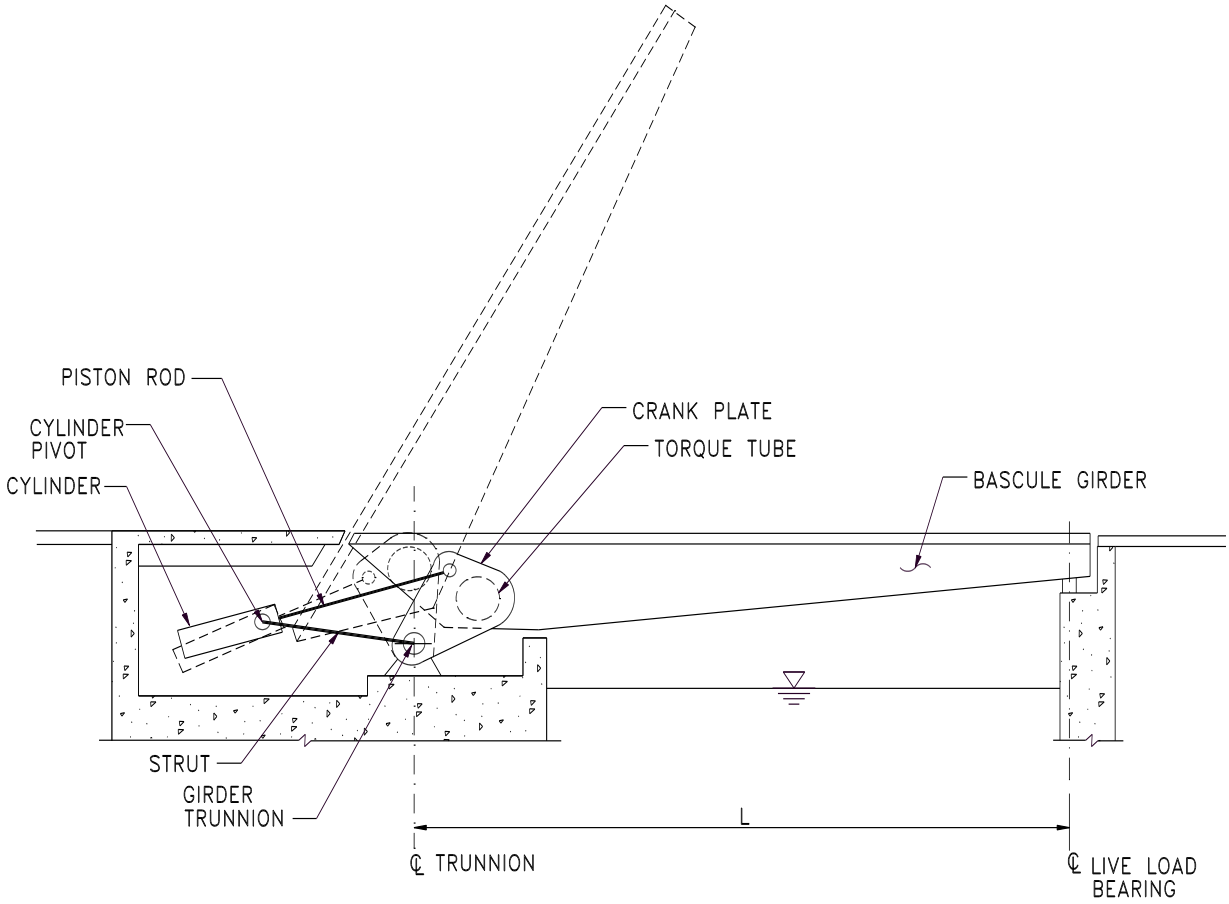


Figure 3.2.6-1: Bascule Without Counterweight – Hydraulic Cylinder Drive.

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3.3 Vertical Lift Bridges

3.3.1 Introduction

The movable span of a vertical lift bridge is raised in order to provide clearance for the passage of vessels. The lift span is usually balanced (nominally) by counterweights connected to wire ropes that pass over sheaves at the piers of the movable span. The sheaves are usually located atop towers but they may be located inside the piers. The purpose of the counterweights is to minimize the force required to move the span. This type of bridge is known as a balanced vertical lift. Vertical lift bridges are categorized by the arrangement of the span drive machinery. Figure 3.3.1-1 shows simplified diagrams of some sub-types of balanced vertical lift bridges: the span drive, the tower drive, the connected tower drive, and the pit drive.

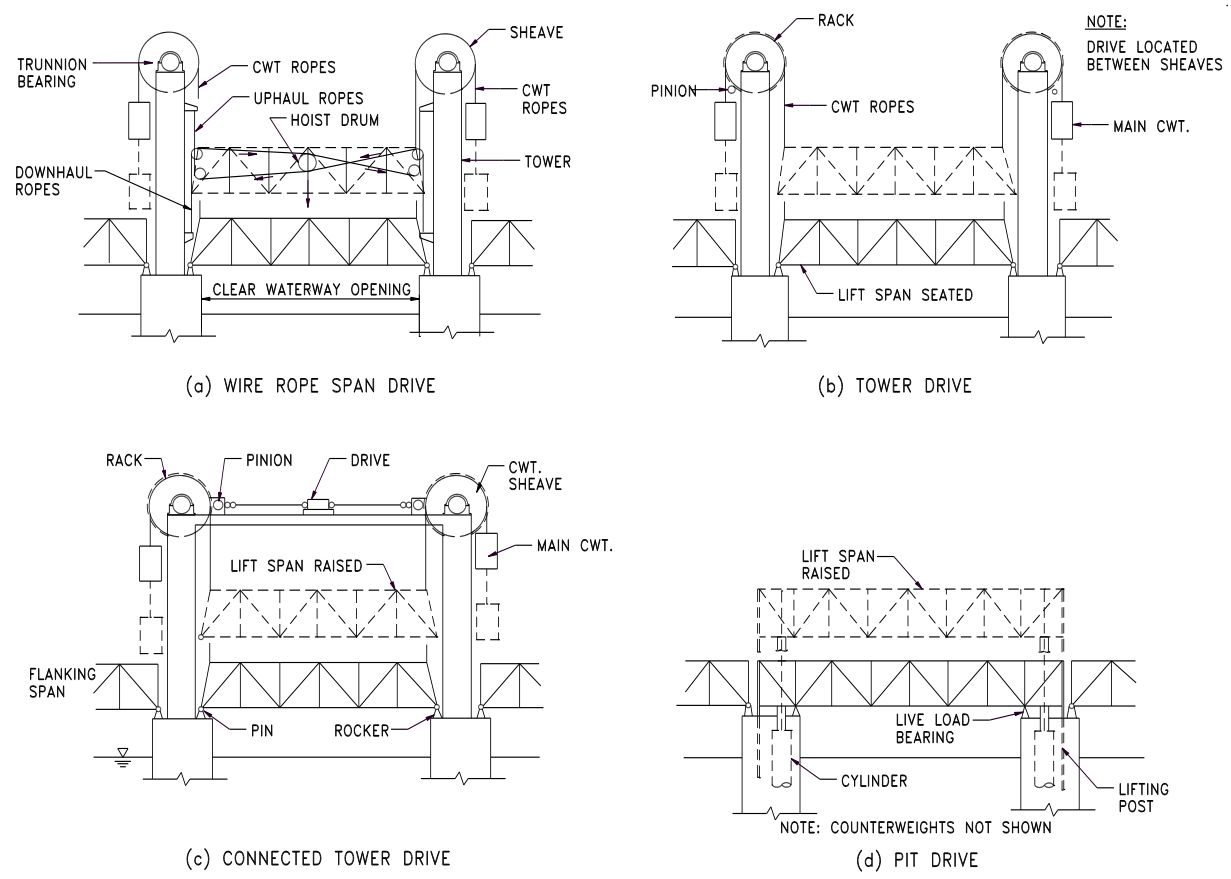

Figure 3.3.1-1: Types of Vertical Lift Bridges.

3.3.2 Span Drive Vertical Lift

The span drive vertical lift bridge is almost always a balanced vertical lift. The ends of the lift span are connected to wire ropes that pass over sheaves at the tops of the towers. The main counterweights are connected to the other ends of the wire ropes. These counterweights balance (or nearly so) the weight of the lift span. For large lift bridges with high lifts the

weight of the counterweight ropes is so large that the ropes themselves are usually counterweighted by an auxiliary system in order to minimize power requirements for operation. The auxiliary counterweight systems are not shown in the figures of this section, for clarity.

3.3.2.1 Wire Rope Span Drive

As shown in Figure 3.3.1-1 (a), the feature defining a span drive bridge is the wire rope drive mounted on the lift span that hauls the span upward or downward. The directions of rope travel shown in the diagram are for the lift span being lowered. An advantage of this type of mechanical drive is that only negligible skewing of the span in the longitudinal direction can occur if the haul rope lengths are properly adjusted. As used here, the word skew refers to the difference in elevation between the two ends of the lift span (for a particular vertical position of the span). In the figure the hoist drum is located at midspan, but span drive bridges exist for which the primary drive machinery is at midspan and the secondary machinery and hoist drums are located at the ends of the span.

3.3.2.2 Rack and Pinion Span Drive

Instead of using haul ropes to raise and lower the balanced lift span, bridges have been built with a rack and pinion drive (not shown in Figure 3.3.1-1). They are balanced lift bridges. The primary machinery is located at the middle of the span and secondary machinery (terminating with pinions) is located at each end. The pinions engage racks mounted vertically on the towers and rotation of the pinions raises or lowers the span. This type of drive prevents significant longitudinal skewing of the lift span.

3.3.3 Tower Drive Vertical Lift

Tower drive vertical lift bridges, as in Figure 3.3.1-1 (b), are usually balanced bridges with the span drive machinery located at the towers, either atop the towers or below the roadway level, but above high water level. There are two basic types; the traction drive and the winch drive. The distinguishing feature of tower drive vertical lifts is that the machinery in one tower is mechanically independent of that in the other tower. Hence, non-uniform raising of the span end-to-end has to be limited, and compensated, by the motor controls.

3.3.3.1 Wire Rope Traction Drive

Figure 3.3.1-1 (b) is a diagram of a tower drive vertical lift bridge with a traction drive. There is drive machinery in each tower that rotates the counterweight sheaves. The forces necessary to raise the span are transmitted from the sheaves to the counterweight ropes by friction. The action is similar to that of a traction drive passenger elevator in a building. Both ends of the lift span should raise and lower at the same rate so that the lift span remains horizontal and does not wedge itself between the towers during motion. There are various electrical/electronic means of controlling the drives in the two towers so that the skew is kept within permissible limits. It should be noted that the force necessary to raise the lift span at each end may differ due to unequal machinery friction, unequal ice loading, etc.

3.3.3.2 Winch Drive

Tower drive lift bridges have been built with winch drives located atop the tower or, more commonly, in the tower base. The lift spans are balanced in the usual fashion but raised and lowered by wire rope winches. The haul ropes are connected to either the lift span framing or to the counterweights. Because the mechanical machinery on one side of the channel is independent of that on the other side, skewing has to be controlled.

Winch drives with the machinery located below roadway level are considered suitable for lift bridges with four independent counterweights.

3.3.4 Connected Tower Drive

A connected tower drive vertical lift bridge is shown in Figure 3.3.1-1 (c). Longitudinal and lateral framing connects the tops of the towers and serves as support for machinery and access walkways. The lift span is balanced by counterweights. However, because this type of lift bridge is most suitable for short spans of low to moderate lift, the counterweight ropes are usually not balanced by auxiliary counterweight systems. The span drive machinery is mounted on the structure connecting the towers. Primary machinery is located in a machine room at midspan from which line shafts extend to speed reducers at the towers, midway between the counterweight sheaves. From the reducers, shafts extend to pinions that engage curved racks fastened to the counterweight sheaves. The force necessary to move or hold the lift span is transmitted between the sheaves and the counterweight ropes by friction.

Because all the span drive machinery is directly connected (normally without any torque equalizers in the system) no major skewing of the lift span in either the longitudinal or transverse direction occurs. However, minor skewing due to differential counterweight rope stretch, different sheave diameters, and rope slip in the counterweight sheave grooves can cause skewing over time. The mechanical system is usually designed with features to permit angular adjustments in order to compensate for these effects. The bridge carrying East Wisconsin Avenue over the U.S. Canal in Kaukauna is a connected tower lift bridge.

3.3.5 Pit Drive Vertical Lift

3.3.5.1 Balanced Pit Drive Vertical Lift

Balanced vertical lift spans are raised and lowered by machinery located within pits that are located inside the rest piers. The pits extend downward below the high water level in the channel. The design objective is, of course, to avoid the need for towers above the roadway. This is accomplished by fastening downward extending legs (lifting posts) to the ends of the lift span as shown in Figure 3.3.1-1 (d). The lifting posts guide the movable span during motion and resist horizontal forces applied to the span when it is not seated on its bearings. One or two lifting posts may be provided at each end of the span, depending on the roadway width. Counterweight ropes (not shown in Figure 3.3.1-1 (d)) are fastened to the lower ends of these lifting posts and looped over sheaves located above and connected to the counterweight. This type of vertical lift bridge is suitable for low lift heights - say less than 40 feet.

The lift span may be raised or lowered by mechanical machinery or hydraulic cylinders. Mechanical drives are usually located in one of the two pits. Pinions engage racks mounted on the lifting posts to move the span. Because the pinions are mounted on the same shaft, the vertical motion is automatically equalized side-to-side. Equalization of vertical motion, end-to-end of the span, is accomplished by a wire rope/tension rope equalizing system.

In hydraulic cylinder drives the span is usually moved by one or two cylinders in each pit. Equalization of piston travel is accomplished either by controlling fluid flow to the individual cylinders, based on measurement of the longitudinal and transverse slope of the lift span, or by tying the cylinders together with hydraulic lines to assure equal pressure is applied to all cylinders, supplemented by mechanical equalization. Mechanical equalization may be accomplished using rotary machinery or wire rope equalizing. An example of a balanced pit drive vertical lift bridge with hydraulic cylinders and a wire rope equalizing system is the Michigan Street Bridge over the Milwaukee River in the City of Milwaukee. See Figure 3.3.5.1-1.

Figure 3.3.5.1-1: Balanced Pit Drive Vertical Lift, Michigan Street Bridge in Milwaukee.

The Wisconsin Avenue Bridge, rehabilitated in 2012, uses internal equalization by means of sensors within the hydraulic system. There is no outside mechanical equalization system such as wire rope equalizing. Prior to the rehabilitation the Wisconsin Avenue Bridge, the structure was raised by means of one hydraulic cylinder in each pier and equalized by means of a wire rope system. Refer to Figures 3.3.5.1-2 and 3 for elevation views of the old and current lifting mechanisms.

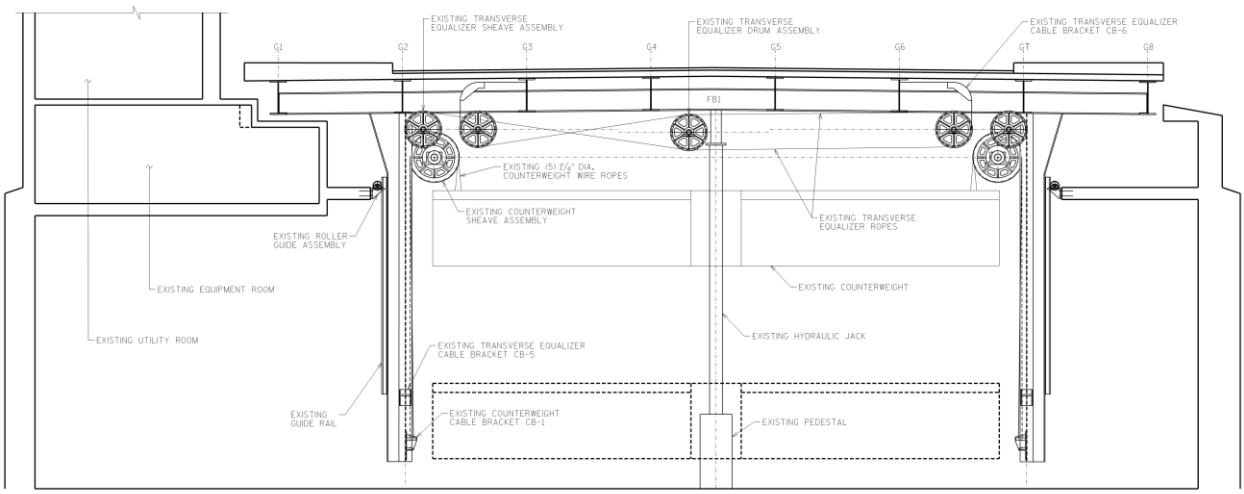

Figure 3.3.5.1-2: Elevation view of previous Wisconsin Avenue Bridge lifting and equalizing components. Note only single hydraulic piston used to lift superstructure.

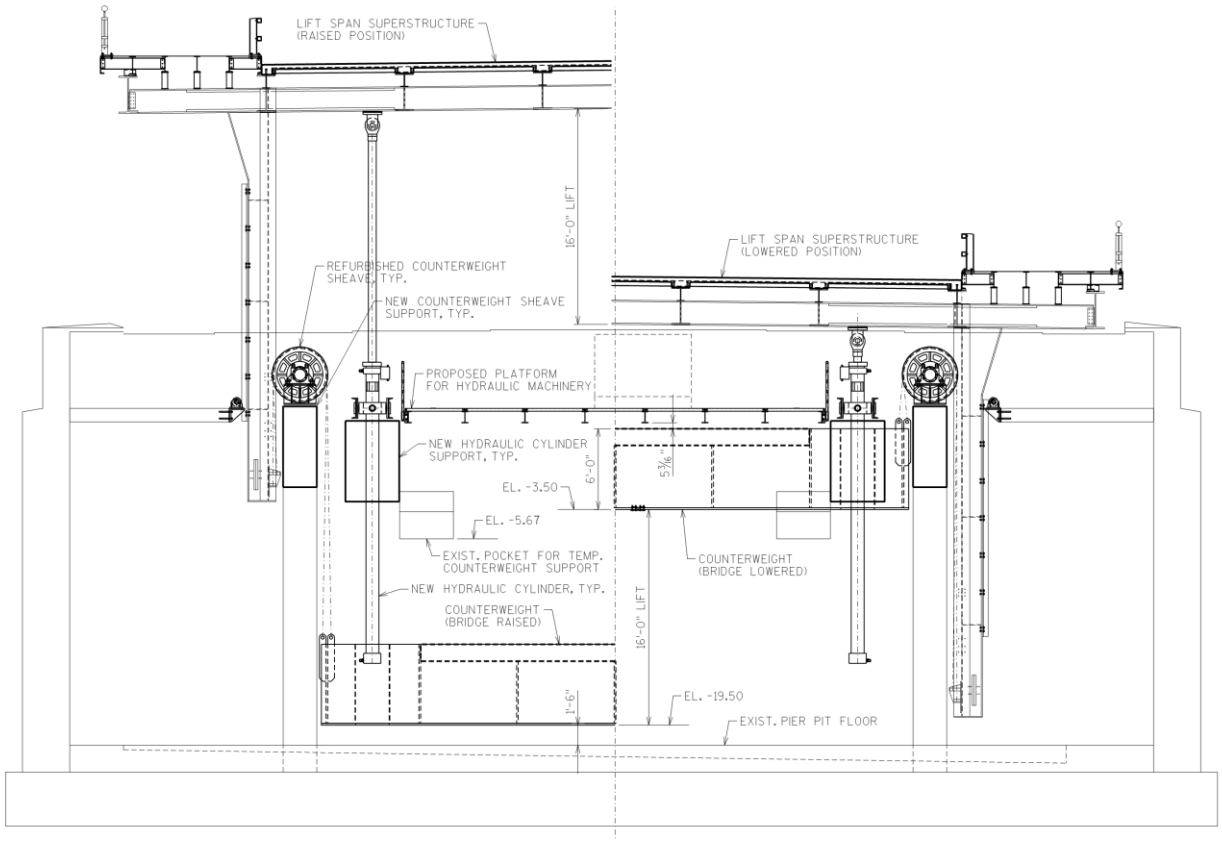

Figure 3.3.5.1- 3: Elevation view of rehabbed Wisconsin Avenue Bridge lifting components. Note two hydraulic pistons used to lift superstructure and absence of equalizing cables.

3.3.5.2 Unbalanced Pit Drive Vertical Lift

Unbalanced vertical lift bridges are constructed when the cost of the greater lifting capacity required (compared to a balanced bridge) is estimated to be less than the cost of counterweights or for reasons of aesthetics. Hydraulic cylinder drives are most suitable for unbalanced lift bridges because large actuating forces can be produced using smaller electric motors than in a mechanical drive. There is less friction between the prime mover and the point of force application in the hydraulic system.

The Highland Avenue Lift Bridge in Milwaukee is an example of an unbalanced pit drive vertical lift bridge. For this structure, there are two hydraulic cylinders in each of the two pier pits which support the full weight of the span when it is lifted up off its supports. As the span is not balanced by a counterweight, a sudden loss of hydraulic pressure when the span is in a raised position could cause the span to drop. As a safety precaution two spring-loaded rail clamps are provided at each lifting post. The design concept is that they clamp onto rails mounted on the lifting posts and thereby retard motion if the span should descend uncontrolled or at an excessive skew.

The Highland Avenue Bridge, in the city of Milwaukee is an unbalanced pit drive vertical lift pedestrian bridge. The bridge is designed to accommodate either a full pedestrian live load or a single H10-44 live load (an emergency vehicle). It should be noted that a vehicle can only approach the span from the east end since steps and a guardrail and sidewalk are on the west end. Refer to Figure 3.3.5.2-2 for a photograph of the Highland Avenue Bridge.

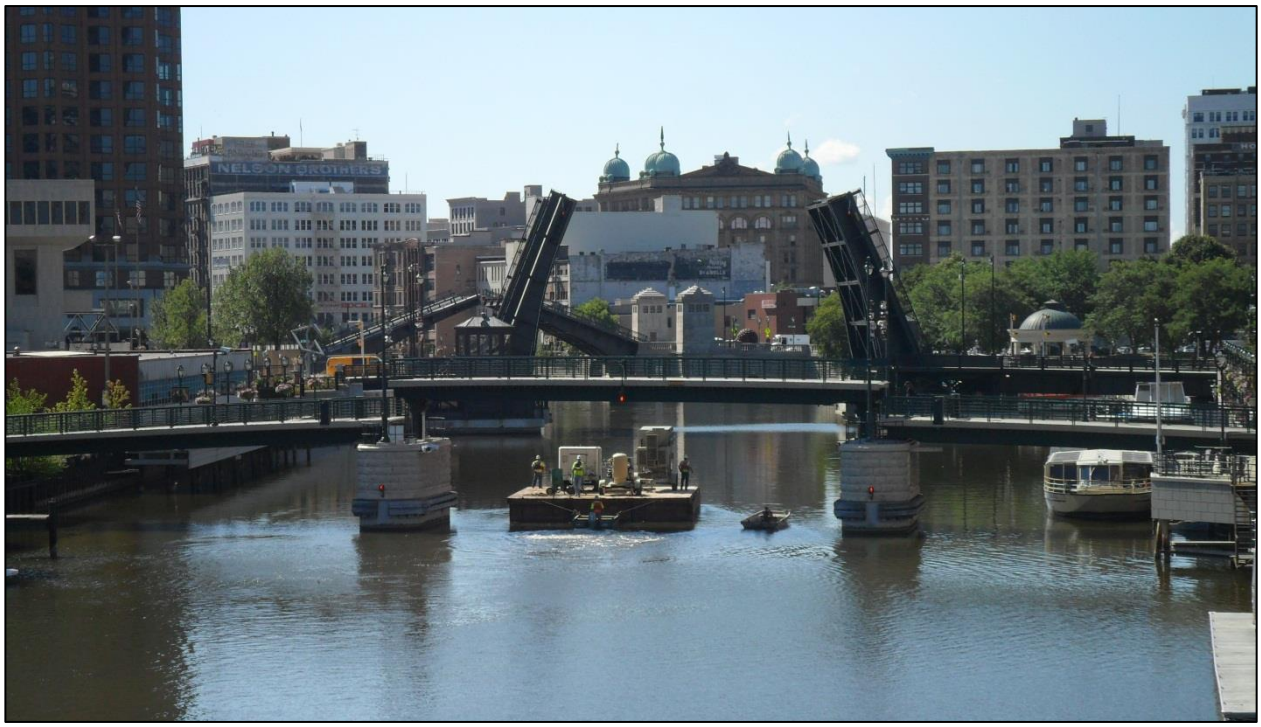

Figure 3.3.5.2-1: Unbalanced Pit Drive Vertical Lift, Highland Avenue Pedestrian Bridge in Milwaukee.

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3.4 SWING BRIDGES

3.4.1 Introduction

The movable span of a swing bridge, also termed the draw, rotates about a vertical axis (pivot axis). If the pivot axis is at mid-length of the draw, the draw is said be symmetrical or to have equal length arms. Sometimes, the arms are not of equal length and the draw is termed unsymmetrical or bobtailed. The dead load (self-weight) of a swing span is usually balanced about the pivot. Hence, bobtailed spans require counterweights at the ends of the shorter arms. Some bobtail draws are only nominally balanced about the pivot, with a definite excess of counterweight. The reason for the tail-heavy condition is to tilt the structure and raise the toe of the channel arm above its rest pier. This condition aids in closing the span. This tilting is sometimes referred to as secondary motion.

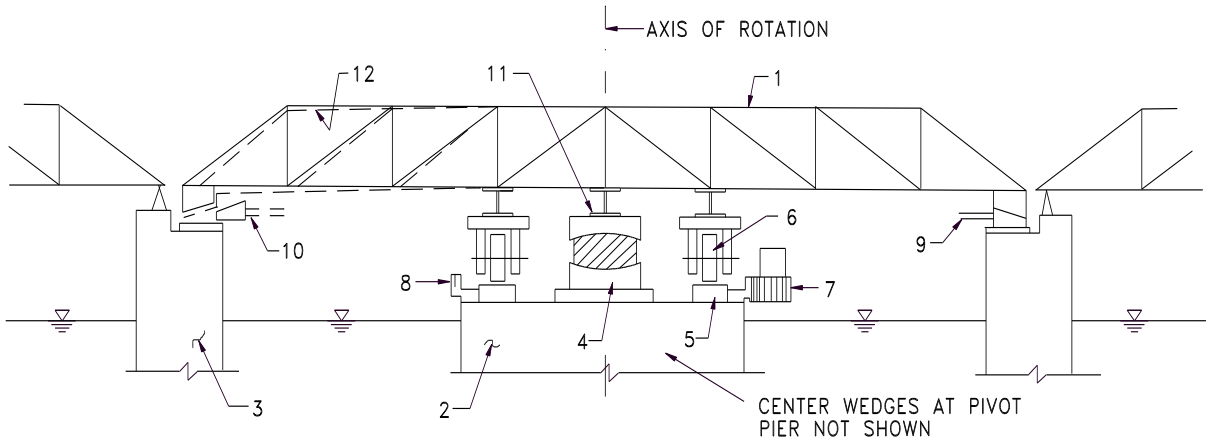
Swing bridges are also categorized according to type of pivot bearing. If all the dead load is supported at the center, the swing span is said to be center bearing. If all, or most, of the dead load is supported by a large-diameter ring of rollers concentric with the pivot axis, the bridge is termed rim bearing.

3.4.2 Center-Bearing Swing Bridges

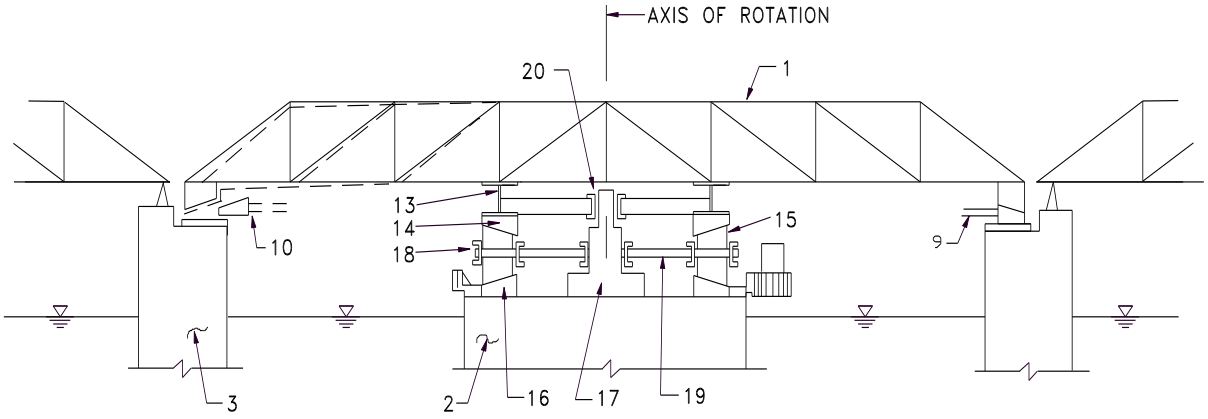
Figure 3.4.2-1 (a) is a diagram of an equal-arm center-bearing swing bridge. The spanning member is shown as a truss; it could as well be a girder. The draw weight is balanced on the pivot bearing, which is mechanical in the figure, but could be hydraulic. To prevent the draw from tipping under unbalanced loads, such as wind, balance wheels are provided that roll on a large-diameter circular track concentric with the pivot bearing. When the draw is balanced these wheels normally clear the track by about 0.2 inch (5 mm). The design intent is that the center bearing support all the dead load when the draw is open. Figure 3.4.2-1 (a) shows a center bearing swing bridge with a plain mechanical pivot bearing, comprised of a lenticular bronze disc between hardened steel concave discs. Sometimes two-part bearings are used. Plain bearings are most common. However, spherical antifriction rolling element bearings are being specified more frequently for new bridges. Hydraulic bearings that support the entire dead load have been used for some center bearing swing bridges.

In the United States live load on center bearing swing bridges is usually supported by center and end lift devices (often wedges) which are actuated when the draw is returned to the closed position. They provide the load path for the free ends of the girders and also provide a firm intermediate live load support for the girders at the pivot pier.

Rotation of the draw is by means of mechanical or hydraulic machinery, or a combination thereof. When the mechanical span drive is mounted on the draw one or more downward extending pinion shafts engage a rack mounted on the pivot pier and rotate the draw, as shown in Figure 3.4.2-1 (a).



(a) CENTER BEARING



(b) RIM BEARING

1. SWING SPAN (DRAW)
2. PIVOT PIER
3. REST PIER
4. CENTER BEARING
5. TRACK
6. BALANCE WHEEL
7. PINION
8. RACK
9. END WEDGES (EXTENDED)
10. END WEDGES (WITHDRAWN)
11. DISTRIBUTION FRAMING
12. DEFLECTED POSITION (WEDGES WITHDRAWN)
13. DRUM GIRDER
14. TREAD PLATE
15. TAPERED ROLLER
16. TRACK PLATE
17. PIVOT POST
18. LIVE RING
19. SPIDER
20. DRAW PIVOT BEARING

Figure 3.4.2-1: Types Of Swing Bridges.

3.4.3 Rim-Bearing Swing Bridges

Swing bridges in which all the dead load of the superstructure is supported by tapered rollers when the draw is in the open position are termed rim bearing. The superstructures of rim bearing swing bridges are supported by a minimum of two longitudinal spanning members (trusses). As shown in Figure 3.4.2-1, (b), the tapered rollers run on a circular track whose diameter is usually about the same as the transverse spacing of the outer swing span trusses or girders. Tapered rollers (the common term) are really frustums of cones. It is necessary that the rollers be conical because the distance traveled by the outer end of a roller is longer than that traveled by the inner end, for the same angle of bridge rotation. When the bridge is closed, the rim bearing supports both dead load and live load. Rim bearings are used for wide heavily-loaded swing bridges. Special load-equalizing framing is provided to transfer the loads from the bridge trusses to the circular drum girder at a number of points around the periphery so that it is uniformly loaded along its length (at least for dead load). The load is transferred through the drum girder to a tapered tread plate supported by tapered rollers. Rotation of the draw may be by the same means as for the center-bearing swing bridge.

Alternatively, the mechanical span drive may be mounted on the pivot pier, in which case the pinion shafts extend upward to engage a rack mounted on the periphery of the drum girder. If the span drive uses hydraulic slewing cylinders, no rack is necessary.

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3.5 FUNCTIONAL SYSTEMS

3.5.1 Introduction

The machinery of a movable bridge may be classified as span drive machinery and span stabilizing machinery. Span drive machinery comprises those mechanisms which move the span. It also serves to stabilize the span when the span is not in the fully-closed position. Stabilizing machinery supports and restrains the span when it is at rest and when in motion, but it does not accelerate the span. Span drive machinery is sometimes called operating machinery but this terminology can be ambiguous because some stabilizing machinery also operates, in the strict sense. These classifications are mainly for convenience of reference in machinery descriptions.

The span drive machinery varies little with movable bridge type because the objective is the same for all types, to convert the low-torque rotation of a motor (electric or internal combustion) to the high torque or force required to move the span. In electro-mechanical drives, gearing is used to convert the low-torque, high-speed, rotation of the motor to high-torque low-speed rotation for moving the span. In hydraulic/mechanical systems the high-speed low-torque output shaft of the motor rotates a pump which pressurizes a suitable hydraulic fluid. Modern hydraulic systems utilize mineral, vegetable, or synthetic fluids – the older systems used water. The pressurized fluid is transmitted to hydraulic cylinders, which move the span directly, or to hydraulic piston motors, which move the span by rotating a rack pinion, sometimes using intermediate gearing.

Stabilizing machinery comprises electro-mechanical and hydraulic components that support the span when it is in motion and when it is at rest. Some stabilizing components do not support the moving span – they may be used to decelerate it under certain conditions.

In this chapter common span drive and stabilizing machinery arrangements are described. The emphasis will be on layouts found on movable bridges in Wisconsin.

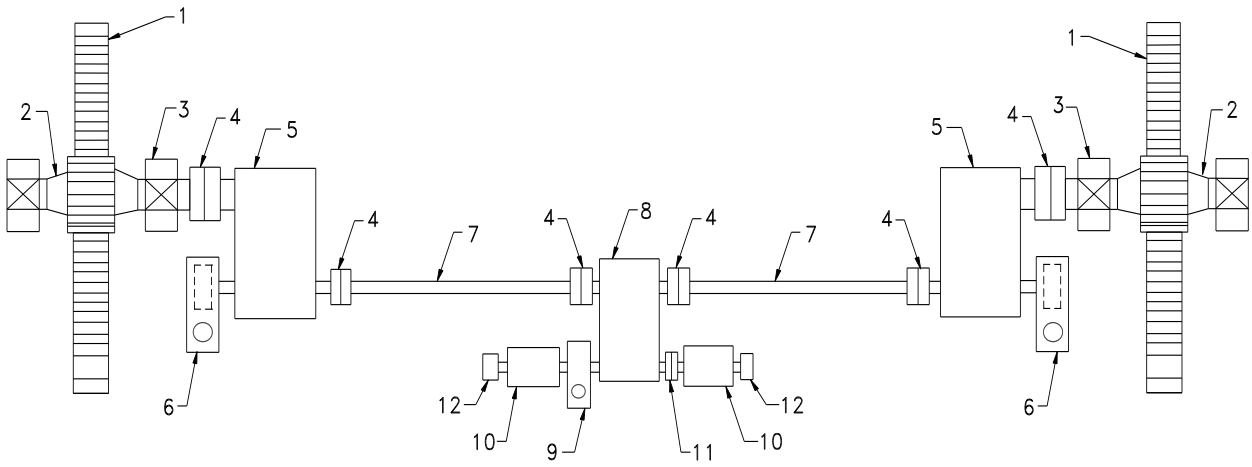
3.5.2 Span Drive Machinery

Many combinations of electrical, mechanical, and hydraulic components may be assembled to form span drives. There is no “right way” to move a span. However, some machinery arrangements have become common because they were part of the original bridge patentee’s designs and, after the patents expired, were regarded as traditional. In this section, we shall consider electro-mechanical and electro-hydraulic-mechanical drives.

3.5.2.1 Type 1 Span Drives

A common arrangement of electrical and mechanical components to form a span drive with two low-speed, high-torque final outputs is shown in Figure 3.5.2.1-1. Span drives in which all the outputs are connected mechanically are denoted as Type 1, and the three subtypes are differentiated by using the suffixes ND, AD, and LD, as explained later. In Type 1 arrangements, a motor is connected to the primary speed reduction gearing, usually by a flexible coupling. This gearing is enclosed in a welded steel housing on most contemporary designs, but cast steel and various kinds of cast iron were and are used in manufacturing these speed reducers. Power is distributed from the primary reduction gearing to two (or

more) sets of secondary reduction gearing located at the sides or ends of the movable span. The secondary speed reductions are often made with enclosed speed reducers as depicted in Figure 3.5.2.1-1, however, open gearing is also very common. On some newer bridges the primary and secondary reduction may be made in one larger speed reducer. Three different variations of Type 1 span drives have been installed on bridges – based on the capability of the drive to equalize the output torques.



1. RACK
2. PINION SHAFT
3. BEARING
4. GEAR COUPLING
5. SECONDARY GEAR SPEED REDUCTION
6. MACHINERY BRAKE
7. FLOATING SHAFT
8. PRIMARY SPEED REDUCTION
   TYPE 1 – ND DRIVE: NO DIFFERENTIAL
   TYPE 1 – AD DRIVE: ACTIVE DIFFERENTIAL
   TYPE 1 – LD DRIVE: LOCKABLE DIFFERENTIAL
9. MOTOR BRAKE (WITH FLEXIBLE COUPLINGS)
10. MOTOR
11. FLEXIBLE COUPLING
12. TACH GENERATOR

Figure 3.5.2.1-1: Type 1 Span Drive.

Type 1-ND Span Drive (No Differential)

In Type 1-ND span drives, the motor is connected to the input shafts of the primary speed reducer, which may have two or more parallel shafts. The number of speed reductions is one less than the number of shafts, except when the reducer contains idler gearing. The output shaft is continuous through the gear box and hence both shaft extensions rotate at the same speed. However, the torques delivered to each extension are not necessarily equal, in fact they usually are not. The distribution of torque between the two extensions is a function of the stiffnesses of the drives outboard of the primary reducer and the rigidity of the structure being driven. Much depends on the simultaneous engagement of both sets of pinion and rack teeth.

Type 1-AD Span Drive (Active Differential)

Under some circumstances, designers consider it important that the primary speed reducer has two output shafts rather than a common output shaft, thus permitting the output shafts to

rotate at slightly different speeds, but transmit equal torques. This distribution of speed and torque is accomplished by including a differential in the primary speed reduction gear train. The principle and function of this differential is the same as that on a rear-wheel drive automobile. If the differential is always active (it is said to be free), we designate the span drive as a Type 1-AD.

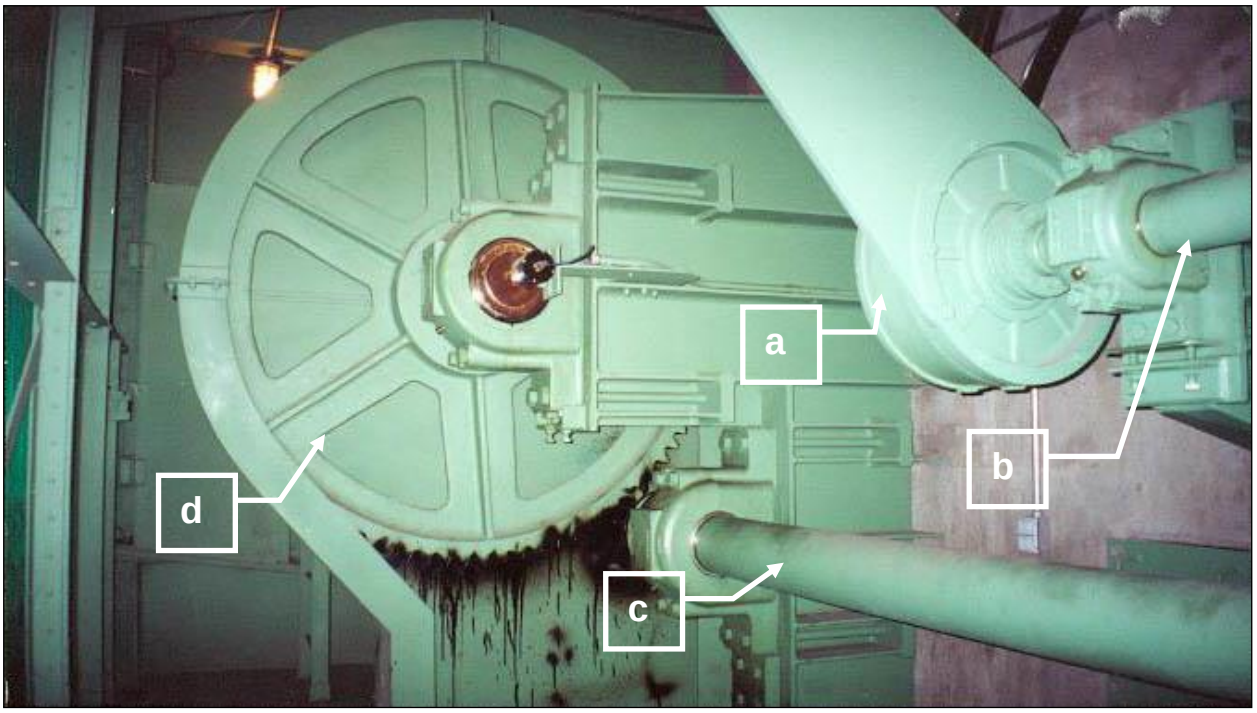
Type 1-LD Span Drive (Lockable Differential)

There are situations for which the designer wants the output shaft extensions of the primary reducer to normally rotate at the same speed but, under special conditions, wants to permit differential action. In order to enable this dual action a primary gear box is provided which incorporates a differential mechanism that can be "locked out", i.e., both output shaft extensions can be forced to rotate at the same speed. When the differential is unlocked, i.e., the differential is "free", it behaves as a normal differential gear box. A drive with a lockable differential in the primary train is denoted as a Type 1-LD Span Drive.

3.5.2.2 Secondary Speed Reductions for Type 1 Span Drives

From the primary reduction gearing, power is transmitted to the secondary reduction gearing by shafting, which is arranged to be "floating" in contemporary designs in order to minimize the effect of superstructure distortion on the action of mechanical components. The floating action is achieved with the aid of flexible shaft couplings that permit only minimal amounts of axial, transverse, and angular motion at the joint while transmitting torque.

If the output shaft of the secondary gearing is connected to only one rack, as shown in Figure 3.5.2.1-1, the secondary gearing does not contain a differential. Such is the case for trunnion and rolling lift bascules and for vertical lift bridges with only one counterweight sheave per corner of the span. If the secondary reducer drives two racks, it is usually equipped with an active differential so that the rack tooth forces are equalized. Figure 3.5.2.2-1 shows a portion of the drive machinery for a typical bridge.



a Hydraulic motor with torque arm
b Input shaft to primary gear reduction
c Input shaft to secondary gear reduction
d Gear

Figure 3.5.2.2-1: Secondary Gear Reduction – 8th Street Bridge, Manitowoc.

3.5.2.3 Span Drive Type Relative to Movable Bridge Type

The Type 1 span drives described in the foregoing have been installed on the following types of movable bridges:

1. Type 1-ND
   - Trunnion bascules
   - Rolling lift bascules
   - Swing bridges (mostly early). Not desirable, in general.
   - Vertical lift bridges [span drive, connected tower drive, tower drive, and pit drive].
2. Type 1-AD
   - Trunnion bascules
   - Rolling lift bascules
   - Swing bridges
3. Type 1-LD
   - Trunnion bascules (differential normally locked)
   - Swing bridges (differential normally unlocked)

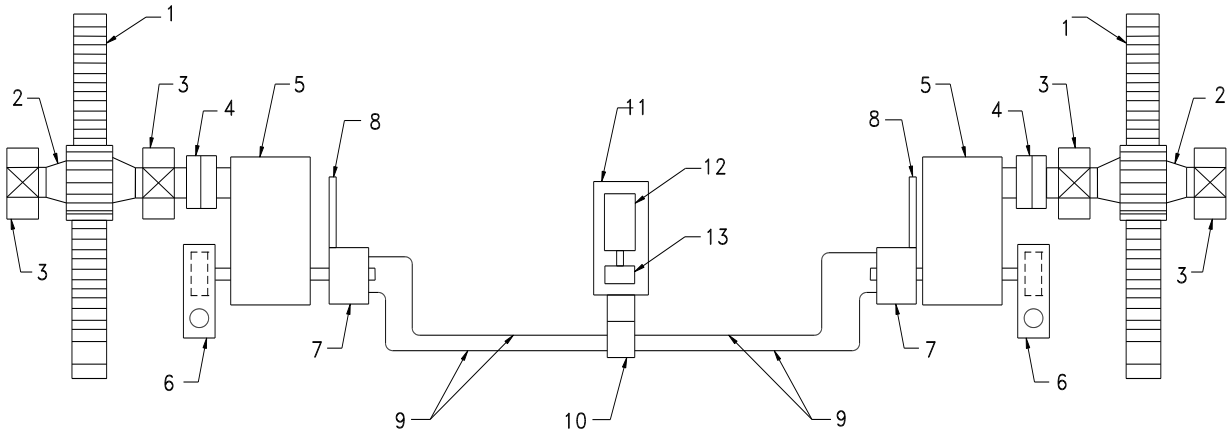
Vertical lift bridges (differential normally locked), [span drive, connected tower drive, and tower drive].

3.5.2.4 Type 2 Span Drives

In the Type 1-ND and Type 1-LD (when locked) mechanical drives, as described previously, the two output pinions rotate at the same velocity. Unless the structure to which the machinery is fastened is very flexible, or the machinery is well aligned, or the machinery is very “springy”, the teeth of both racks will not simultaneously be subjected to equal force. In other words, the racks will not “share the load” equally. The early rolling lift bascules had Type 1-ND span drives which were torsionally soft (springy) and hence tended to distribute the torque to the two racks nearly uniformly if the gearing was properly installed. However, in later bridges stiffer shafting was utilized and hence differentials were installed (Type 1-AD) to make load sharing more reliable.

Not all bridge designers are convinced of the need for differentials to ensure load-sharing between multiple racks. They are willing to forgo uniform loading of racks for the safety advantage of the Type 1-ND or Type 1-LD (locked) drives due to their redundancy outboard of the primary reducer. The two philosophies will not be discussed further here because the emphasis of the Manual is on inspection, not design.

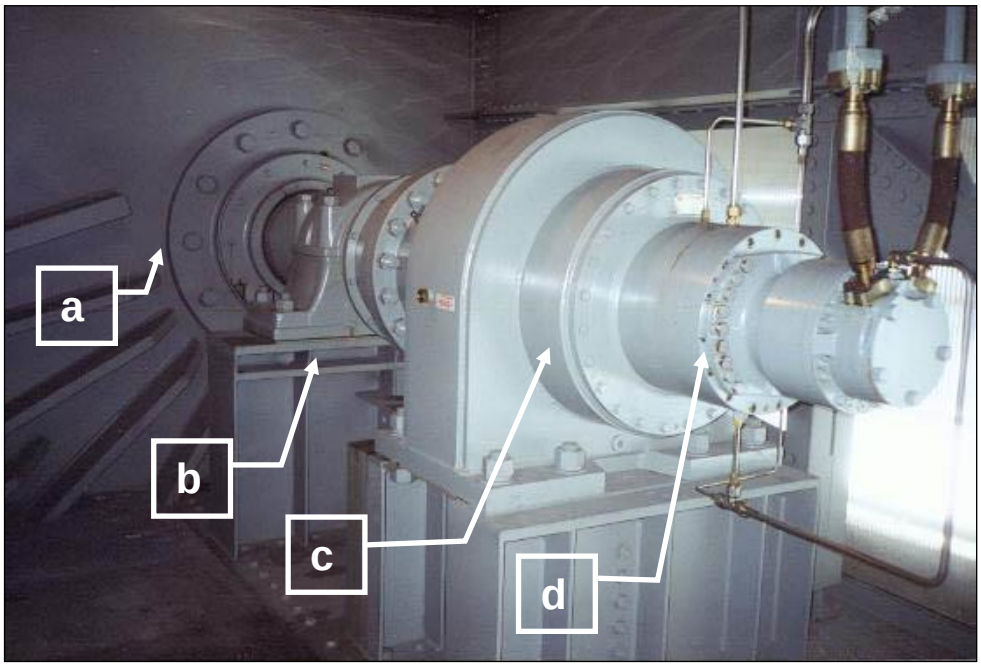
Another approach to load sharing has been adopted by designers of large and stiff simple trunnion bascules and swing bridges. One or more pinions, each separately powered, engage a common rack in the case of swing bridges. For bascules with two racks, each pinion is separately powered. The reasoning is that if all of the drive machinery packages are alike, then the pinion tooth loads should be alike. In practice, this simplistic approach has not proved valid during driving when electric motors are prime movers, especially with modern feed-back controls. The pinion tooth loads are also not alike during retarding if each drive package is equipped with thrustor-released, spring set, friction brakes.



1. RACK
2. PINION SHAFT
3. BEARING
4. GEAR COUPLING
5. SECONDARY GEAR SPEED REDUCTION
6. MACHINERY BRAKE
7. HYDRAULIC MOTOR
8. TORQUE ARM
9. HYDRAULIC LINES
10. MANIFOLD & VALVES
11. TANK
12. MOTOR
13. PUMP

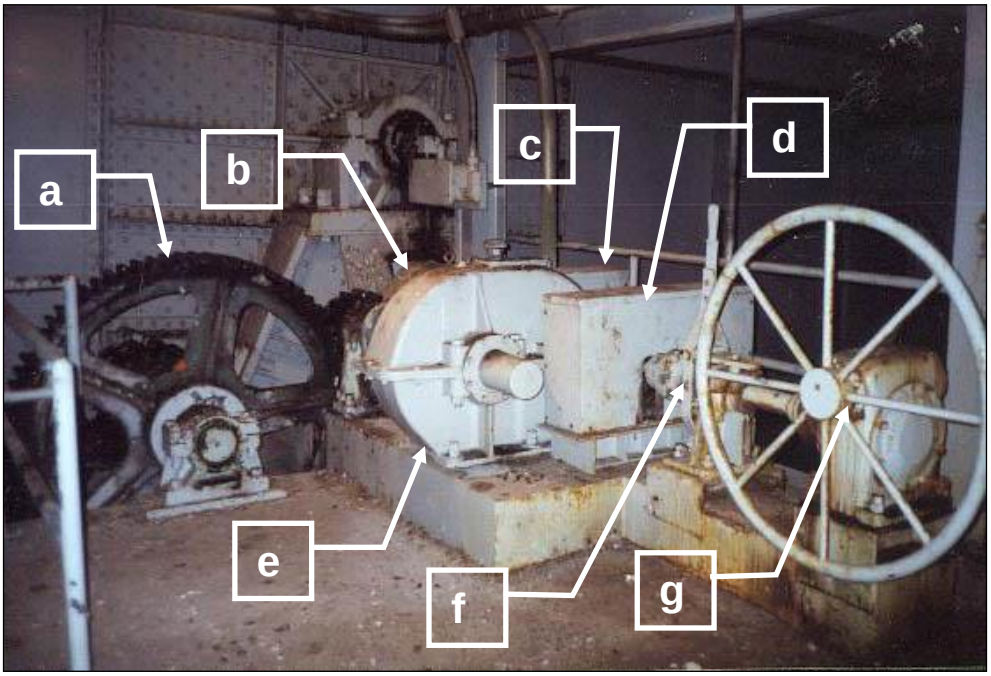
Figure 3.5.2.4-1: Type 2 Span Drive.

Figure 3.5.2.4-1 depicts a Type 2 span drive with two pinions, each powered by a separate hydraulic motor through a geared speed reduction. The fluid input to the two motors comes from a common source; hence the torques applied by the pinions should be nearly equal, assuming that both sets of motors and gearing, and piping, etc., are alike. Parallel shaft gear reducers are shown in Figure 3.5.2.4-1, but planetary gear reducers may be used as well, as were installed at Main Street in Green Bay (see Figure 3.5.2.4-2). Open gear trains are also common.



a Pinion Shaft
b Planetary Speed Reducer
c Hydraulically released spring disk brake
d Hydraulic Motor

Figure 3.5.2.4-2: Type 2 Span Drive – Main Street Bridge, Green Bay.



a Secondary gear reduction train
b Motor (behind speed reducer)
c Motor Brake
d Machinery Brake
e Primary Speed Reduce
f Clutch (behind manual drive wheel)
g Manual Drive (with right angle reducer)

Figure 3.5.2.4-3: Type 2 Span Drive – Oneida Street Bridge, Appleton.

The Oneida Street Bridge in Appleton has a Type 2 span drive with completely independent electro-mechanical drives. Figure 3.5.2.4-3 depicts one of the two units on this simple trunnion bascule bridge. One of the two drive units is fitted with a means for manual operation.

3.5.2.5 Type 3 Span Drives

Span drives whose output is transmitted to the moving span by linear action, as depicted in Figure 3.5.2.5-1, are designated as Type 3 Span Drives. Such drives have been installed on simple trunnion bascules, rolling lift bascules, swing bridges, and vertical lift bridges. The linear action is usually produced by hydraulic cylinders, but chain, wire rope, and screw drives have also been used. The cylinder mountings shown are for a simple trunnion bascule or a pit vertical lift bridge.

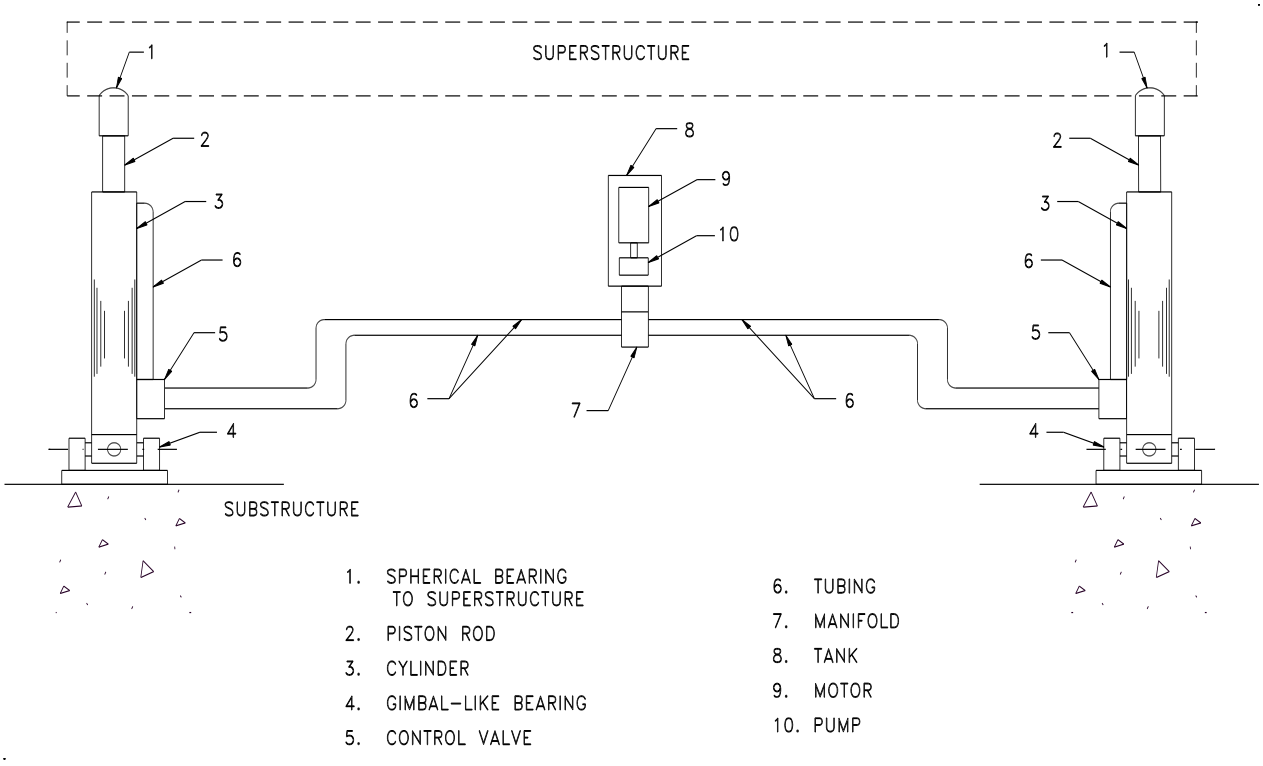
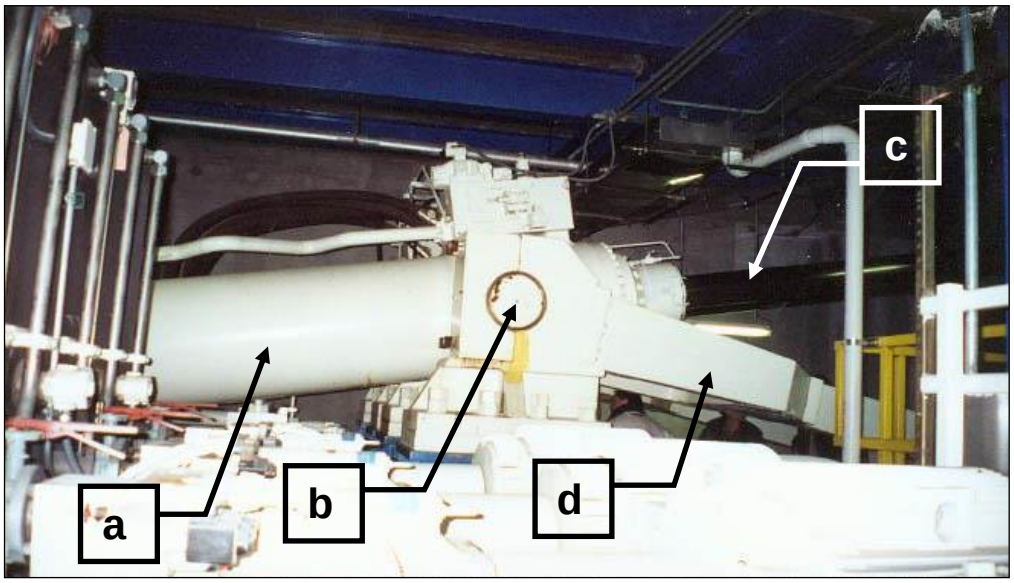


Figure 3.5.2.5-1: Type 3 Span Drive.

Rather than mount the cylinders in an inclined position in the bascule pit, simple trunnion bascules have also been rotated by fastening levers (crank arms) to the ends of the bascule girders and pivot mounting the cylinders in a near horizontal position. The linear action of the cylinders acting on the crank arm produces a torque that rotates the end of the bascule girder. Figure 3.5.2.5-2 shows one of the cylinders driving the Eighth Street Bridge in Sheboygan.



a Cylinder
b Pivot mountings
c Piston rod
d Strut

Figure 3.5.2.5-2: Cylinder of Type 3 Drive, Eighth Street Bridge, Sheboygan.

For rolling lift bascules the cylinder is usually horizontal with the piston rod end usually connected to the bascule girder at the center-of-roll and the cylinder foot fastened to the fixed structure. Extension or retraction of the piston causes the leaf to travel and thus rotate closed or open.

For vertical lift bridges, the cylinders are vertically mounted in pits with the rod end fastened to the lift span.

3.5.2.6 Types of Span Drives on Selected Wisconsin Bridges

Examples of movable bridges in Wisconsin having the types of span drives previously described are listed below.

Type 1-ND Span Drive

1. Wisconsin Avenue, Kaukauna. Connected tower vertical lift.
2. US-10, Prescott. Rolling lift bascule.

Type 1-AD Span Drive

1. Mason Street, Green Bay. Simple trunnion bascule.
2. Walnut Street, Green Bay. Rolling lift bascule.

Type 1-LD Span Drive

1. Unknown

Type 2 Span Drive

1. South Kinnickinic Avenue, Milwaukee. Simple trunnion bascule.
2. Main Street, Green Bay. Rolling lift bascule.

Type 3 Span Drive

1. Highland Avenue, Milwaukee. Unbalanced pit vertical lift.
2. Eight Street, Sheboygan. Unbalanced single-leaf bascule.
3. Main Street, Oshkosh. Rolling lift bascule.

3.5.3 Stabilizing Machinery

Stabilizing machinery supports the span when it is in motion and at rest. The machinery components and assemblies are usually mechanical but fluid power (air and liquid) components are also utilized. General examples of stabilizing components for various types of movable bridges are:

1. **Trunnion Bascule Bridges:** Trunnion shafts and trunnion bearings; live load reactions; toe, mid-span, and tail locks; centering devices; and buffers.
2. **Rolling Lift Bascule Bridges:** Treads mounted on segmental girders; tracks; tail locks; midspan joints or locks; live load reactions; centering devices; and buffers.
3. **Vertical Lift Bridges:** Counterweight sheaves with trunnions and bearings; main counterweight and auxiliary counterweight ropes; live load supports; centering devices; span locks; span guides and guide wheels or shoes; and buffers. Hydraulic cylinders for vertical lift bridges without counterweights.
4. **Swing Bridges:** Center pivot bearings and balance wheels; rim girders, rollers, and spider assemblies; center wedges; end lifts; centering latches or centering mechanisms; buffers; bumpers; and rigid stops.
5. **Retractile Bridges:** Tracks; wheeled bogies; wheel stops and bumpers; and span locks.

The locations of most usual stabilizing machinery components on bascule, vertical lift, and swing bridges are shown in Figures 3.2.2-1, 3.2.3-2, 3.3.1-1, 3.4.2-1, 3.5.3.1-1, and 3.5.3.3-1. The functions of many components are obvious, but the actions and needs for others may not be so obvious. Subsequently, the components usually installed on common bridge types will be briefly explained. Additional details will be found in Chapter 6, Electrical Systems and Chapter 8, Mechanical Systems.

3.5.3.1 Simple Trunnion Bascules

Single Leaf

Figure 3.2.2-1 is a simplified diagram of a single leaf simple trunnion bascule showing stabilizing machinery including:

1. Trunnion Shafts and Bearings**:** They support the full dead load (balanced leaf) and usually support live load when the leaf is closed. In the leaf open position they support the full dead load, wind, and ice.

2. Toe Locks: A toe lock is usually comprised of a lock bar in an operator (electro-mechanical or hydraulic), lock bar guides, and a receiving socket. The arrangement shown in Figure 3.2.2-1 is common but on some bridges the operator is located on the rest pier and the socket on the leaf. Much depends on availability of space and the availability of the submarine cable or other means to transmit power and control signals across the channel. The function of a toe lock is to prevent upward movement of the toe of the leaf due to: live load rearward of trunnion, a counterweight-heavy imbalance condition, or vibration due to moving live load.

3. Live Load Bearing: Besides the obvious live load, the live load bearing supports a positive dead load (from a desired span-heavy imbalance), and the force due to residual torque in the span drive machinery (wind-up on seating).

4. Bumper: Bumpers are installed to stop the moving leaf in the event that the leaf travels beyond the fully open position. Bumpers may be constructed of timber or synthetic rubber.

5. Centering Device: The function of this device is to align the toe of the leaf laterally so that finger joints in the roadway and sidewalk will mate properly during lowering of the leaf. Not all bascules are equipped with centering devices. They are usually installed on highway bridges having long, narrow, leaves and on all bridges having railway tracks.

Double Leaf

Double leaf simple trunnion bascules require a different arrangement of stabilizing machinery because there is no pier at the toe of the leaf.

1. Midspan Lock or Center Lock: Its function is to maintain the adjacent toes of the leaves at the same elevation (or nearly so). In order to accomplish this, the lock bar must be able to transfer live load shear across the open joint between the leaves. They may also be designed to align the leaves laterally.

2. Live Load Bearings: Their function is the same as on the single leaf bascule, but they are not located at the toe of the span. Three possible arrangements are:

    a. **Live load Bearings at Front Wall:** The advantage of the front wall location for a balanced bascule is that the cantilevered length of the bascule girder is shortened; see Figure 3.5.3.1-1 (a). However, an extreme live load applied

near the toe of the leaf when the midspan lock is ineffective can create uplift against the trunnion bearing cap if the ratio B/L is small.

b. **Primary Live Load Bearings at Front Wall, Secondary at Rear of Bascule Girder:** In order to prevent uplift against the trunnion bearing caps, some bridges have primary live load bearings on the front wall and secondary live load bearings at the tails of the bascule girders. There is a small gap above each secondary bearing so that it will only act when the forward part of the leaf is subjected a large live load. There is also a gap or spring arrangement under the bearing cap to preclude loading the cap by uplift.

   If the rear roadway break is located rearward of the trunnions, a secondary live load bearing, or tail lock, may be necessary to stabilize the leaf when live loads are located between the trunnions and the roadway break.

c. **Live Load Bearings at Rear of Bascule Girder:** Locating the live load bearings at the tails of the bascule girders has the advantage that the elevation of the midspan is not as sensitive to live load bearing adjustments as the dimension C in Figure 3.5.3.1-1 is usually larger than B. However, the bascule cantilever span is longer than for the front wall live load bearing arrangement. Shim packs are usually provided for adjustment, but screw adjustments such as on South Kinnickinnic Avenue Bridge in Milwaukee are not uncommon. These simple rear live load bearings are provided on bridges with floor breaks forward of the trunnions. If the breaks are to be behind the trunnions an upward bearing (tail lock) may be provided at the rear end of the bascule girder.

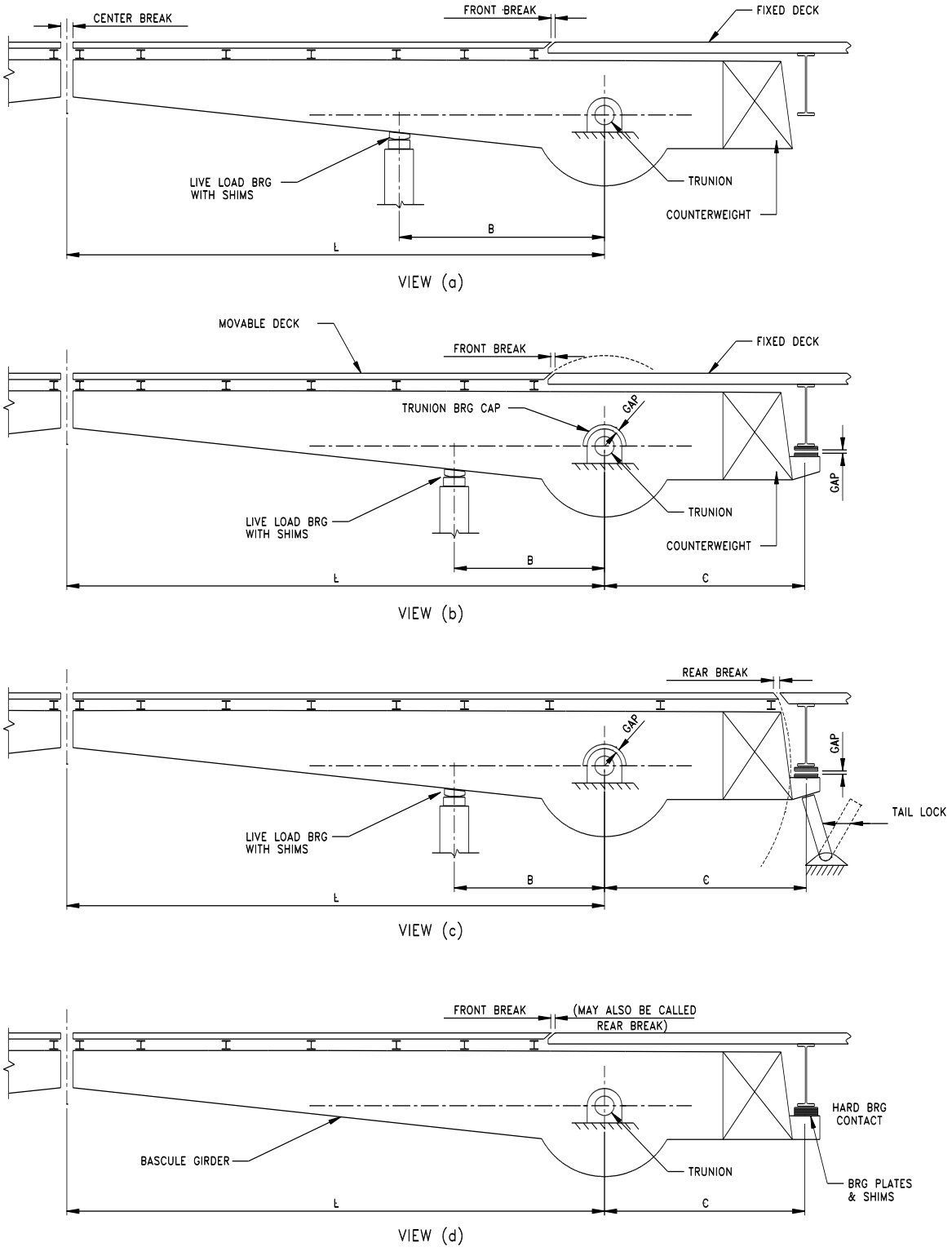


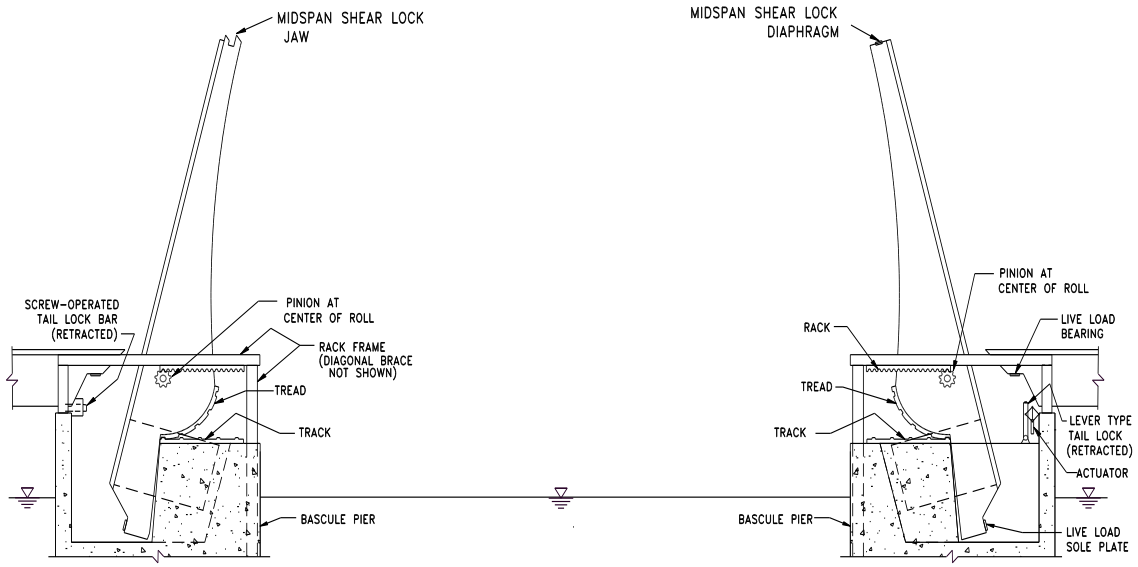
Figure 3.5.3.1-1: Live Load Bearings for Various Types of Double-Leaf Simple Trunnion Bascules

3.5.3.2 Articulated Counterweight Bascules

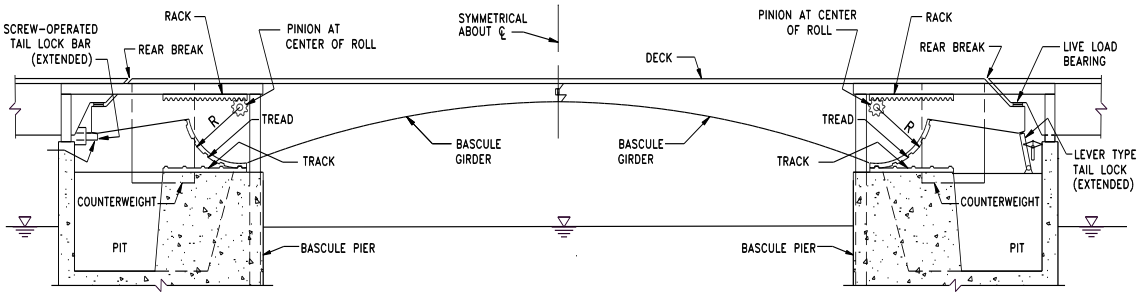
Bascule bridges with articulated counterweights may be single-leaf or double-leaf. All ABT (named after the inventor of the bridge style, Hugo Abt) style bridges are single-leaf. Strauss style bascules may be either single-leaf or double-leaf. The stabilizing machinery is very similar in function to that of the single-leaf or double-leaf simple trunnion bascules. The only additional devices, on some older bridges without automatic deceleration control, are air buffers. Their function is to reduce impact on the structure if the bridge is closed at excessive speed.

3.5.3.3 Rolling Lift Bascules

Stabilizing machinery on rolling lift bascules is illustrated schematically in Figure 3.2.3-2 for a single-leaf and in Figure 3.5.3.3-1 for a double-leaf bascule. The figures are composites; they show features from a number of bridges.



LEAVES FULLY OPEN



LEAVES FULLY CLOSED

Figure 3.5.3.3-1: Double-Leaf Rolling Lift Bascule.

Single-Leaf Rolling Lift Bascule

Some of the essential stabilizing machinery for any rolling lift bascule is depicted in Figure 3.2.3-2 and includes:

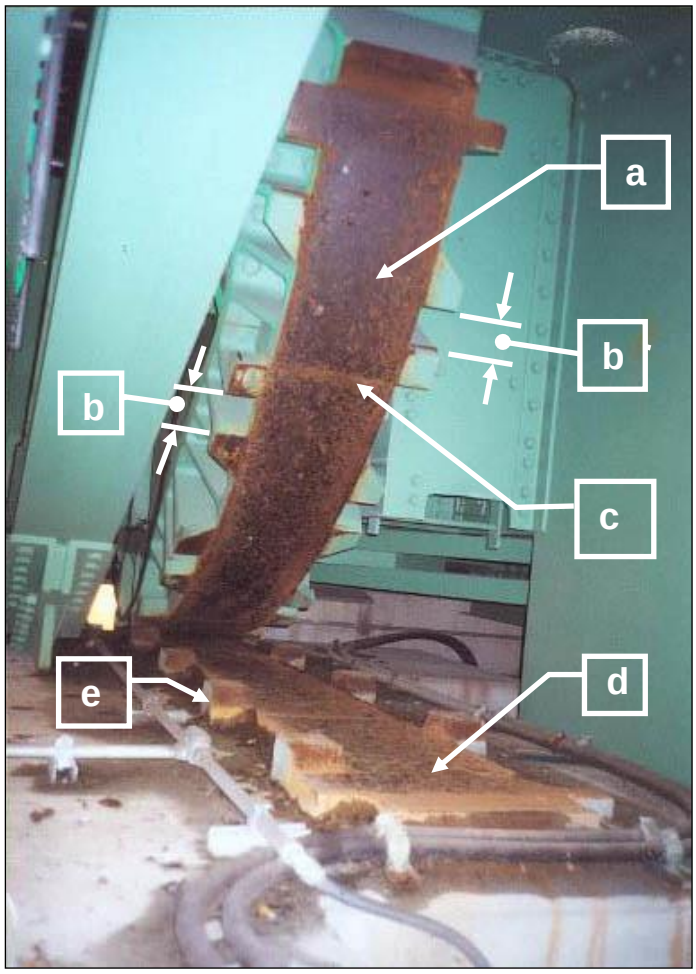
1. Tracks and Treads: Each bascule girder rolls on a track which has upward projecting lugs (they are equivalent to gear teeth). As the bridge may have a multiplicity of bascule girders (from 2 to more than 4) there is a multiplicity of tracks. In the United States, it is typical for all tracks to have lugs; elsewhere it is typical for only the outer two of a leaf to have lugs. The lugs of early rolling lift bridges were protrusions (rectangular in plan) of the steel casting which were spaced from about 10 to 25 inches apart, depending on the radius of roll. Lugs may be found in one line centered on the track, or staggered in two lines along the edges of the track. Figure 3.5.3.3-2 depicts the lugs on the edges of a track of the rolling lift bridge at 8th Street Manitowoc.

   Many tracks were manufactured with lugs made by inserting pins (of circular or rectangular cross section) into vertical holes in the track.

   Another approach to engaging the tracks and treads is to bolt racks to the vertical sides of the track and bolt gear segments to the sides of the tread, as shown in Figure 3.5.3.3-3.

2. Live Load Bearings: Their function is the same as for simple trunnion bascule.

3. Toe Locks: Their function is the same as for simple trunnion bascule. Because the rear floor breaks of rolling lift bascules tend to be proportionately further rearward of the center-of-roll in the closed position than for simple trunnion bascules, greater uplift has to be resisted at the toe. To avoid this situation some single-leaf rolling lift bascules have lever tail locks, as described subsequently for double leaf rolling lift bascules.



a: Tread

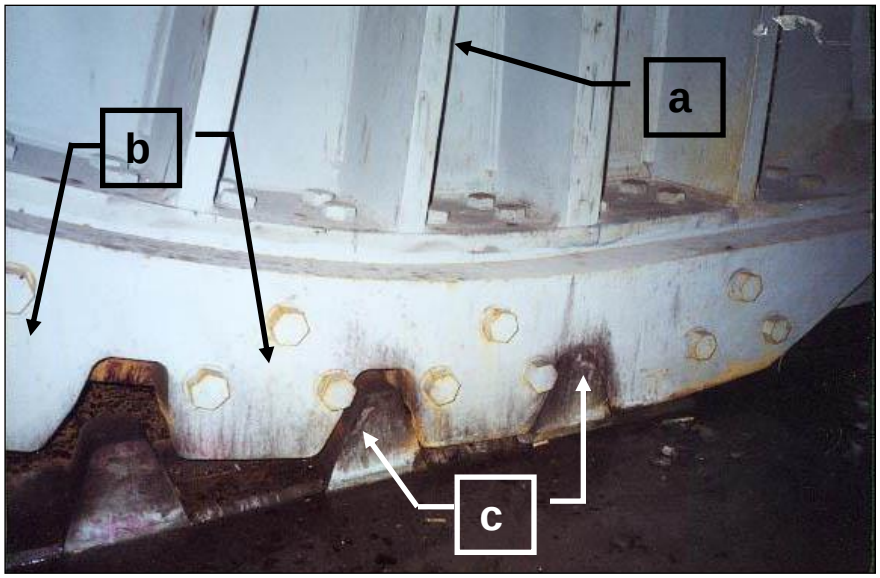
b: Socket

c: Joint Between Segmental Tread Castings

d: Track

e: Lug

Figure 3.5.3.3-2: Track And Tread Castings On Rolling Lift Bascule - Eighth Street Bridge, Manitowoc.



a: Segmental Girder

b: Segmental Tooth Casting (pinion) bolted to Tread

c: Track Tooth Casting (rack) bolted to Track Casting

Figure 3.5.3.3-3: Track And Segmental Girder Tread – Main Street Bridge, Green Bay.

Double Leaf Rolling Lift Bascule

Figure 3.5.3.3-1 shows a double-leaf rolling lift bascule in the open and closed positions with stabilizing machinery noted. It must be emphasized that the design shown supports live load only by bending of the bascule girder. There is no arch action of this double leaf rolling lift bascule because the midspan joint shown cannot resist thrust (horizontal force). The stabilizing machinery includes:

1. Tracks and Treads: Function is same as for single-leaf rolling lift bascule.

2. Midspan Shear Lock (Center Lock): This lock is intended to maintain the toes of both leaves at the same elevation and in proper lateral alignment when the bridge is in the closed position and subjected to live load. In order to accomplish this, it must transfer live load shear between the leaves. The joint shown is known as a Jaw and Diaphragm Detail. On most bridges, a diaphragm casting is bolted to the two side plates at the end of the Diaphragm Leaf. Figure 3.5.3.3-4 depicts a diaphragm casting on the 8th Street Bridge in Manitowoc. As the bridge closes, the jaw shown in Figure 3.5.3.3-5 enters between the diaphragm side plates and the diaphragm casting is engaged between the upper and lower jaws.

   With time, the top and bottom edges of the diaphragm casting and the sliding surfaces of the jaw strike plates wear. The rate of wear depends on the number of bridge openings and the amount of heavy truck traffic passing over the bridge. On the newer rolling lift bascules, shims are provided under the jaw plates in order to permit adjustment. Figure 3.5.3.3-6 is a detail drawing of the mid-span lock parts shown in Figure 5.3.3-4 and Figure 3.5.3.3-5.

   For mid-span locks without provision for adjustment with shims, worn areas may have been built-up by welding, followed by grinding, to achieve the desired reduction in clearance between the diaphragm casting and the strike-plates on the jaw. Regardless of the manner of adjustment, the adjustment should be made so that the roadway joint plates (usually finger-jointed) of both leaves present a flush surface. As a cautionary note, readjustment of rear live load bearings and tail locks may be required in connection with adjustment at mid-span.

3. Tail Locks: Live load on the leaf forward (channel-ward) of the center-of-roll causes uplift at the rear end of the bascule girder, and therefore, structural framing is required to provide a downward reaction. However, when live load is located between the center-of-roll and the rear floor break an upward reaction must be provided. Tail locks provide this function. Because of the motion of the leaf, the tail lock must be retractable.

   Two different tail locks, typical on Wisconsin bridges, are depicted in Figure 3.5.3.3-1. At the left is a retractable lockbar and at the right is a lever that is moved by a toggle. In this 2-dimensional sketch, both mechanisms are shown operating in the longitudinal direction. In reality, the motion of these units on most Wisconsin bridges is in the transverse direction. Figure 3.5.3.3-7 shows a lever tail lock in the extended position (the bridge leaf is fully closed).

4. Live Load Uplift supports: When the bridge is in the closed position each girder's heel top flange should be checked for being firmly seated against the live load uplift support.

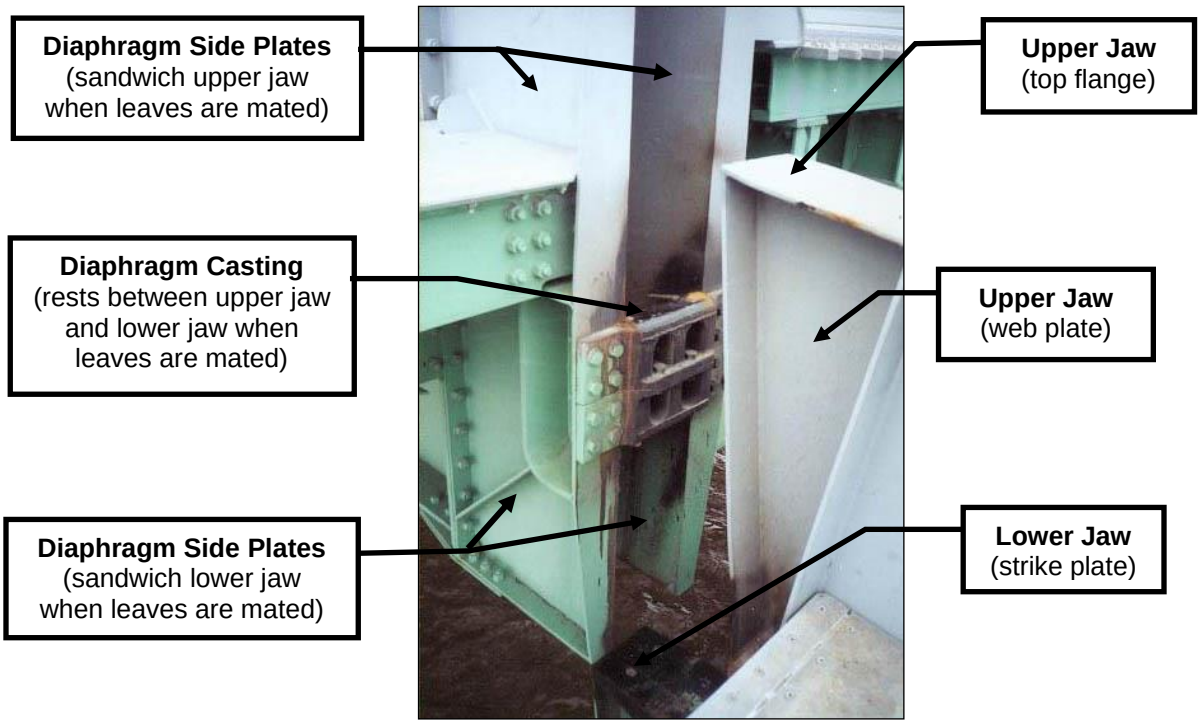


Figure 3.5.3.3-4: Center Lock on Double-Leaf Rolling Lift Bascule, Eighth St. Bridge, Manitowoc.

Note: When leaves begin to mate, diaphragm casting will come to rest on lower jaw strike plate

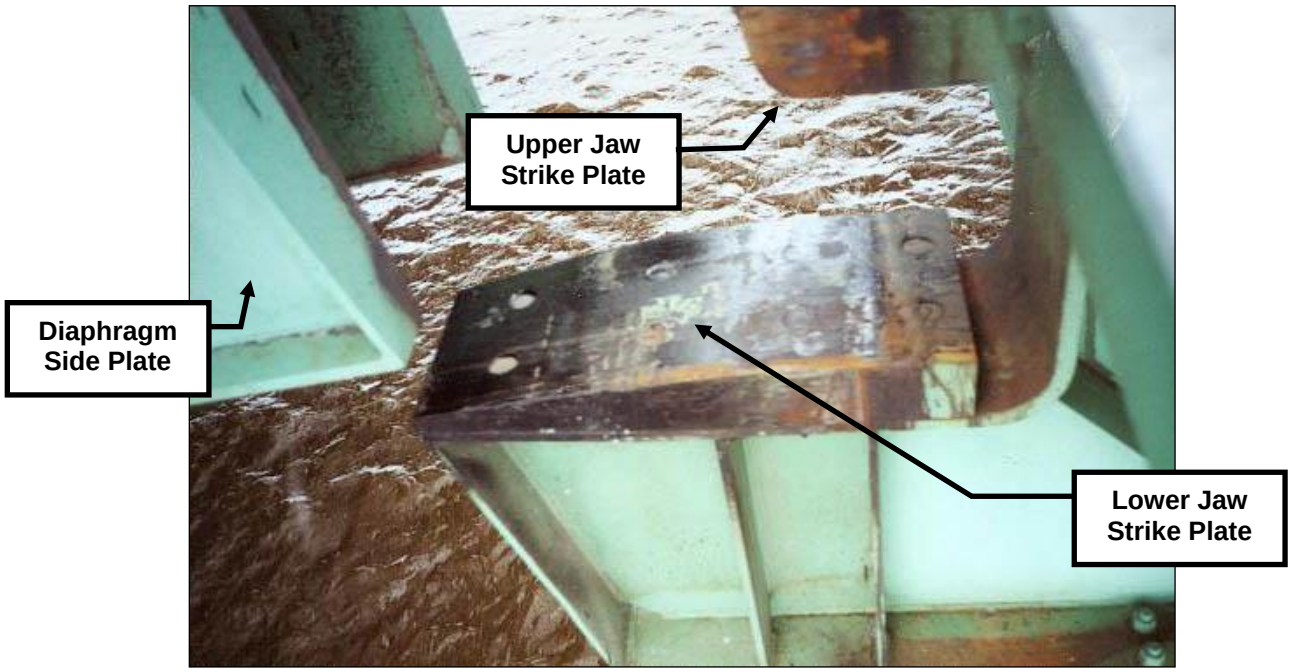


Figure 3.5.3.3-5: Center Lock Jaw on Double-Leaf Rolling Lift Bascule, Eighth St. Bridge, Manitowoc.

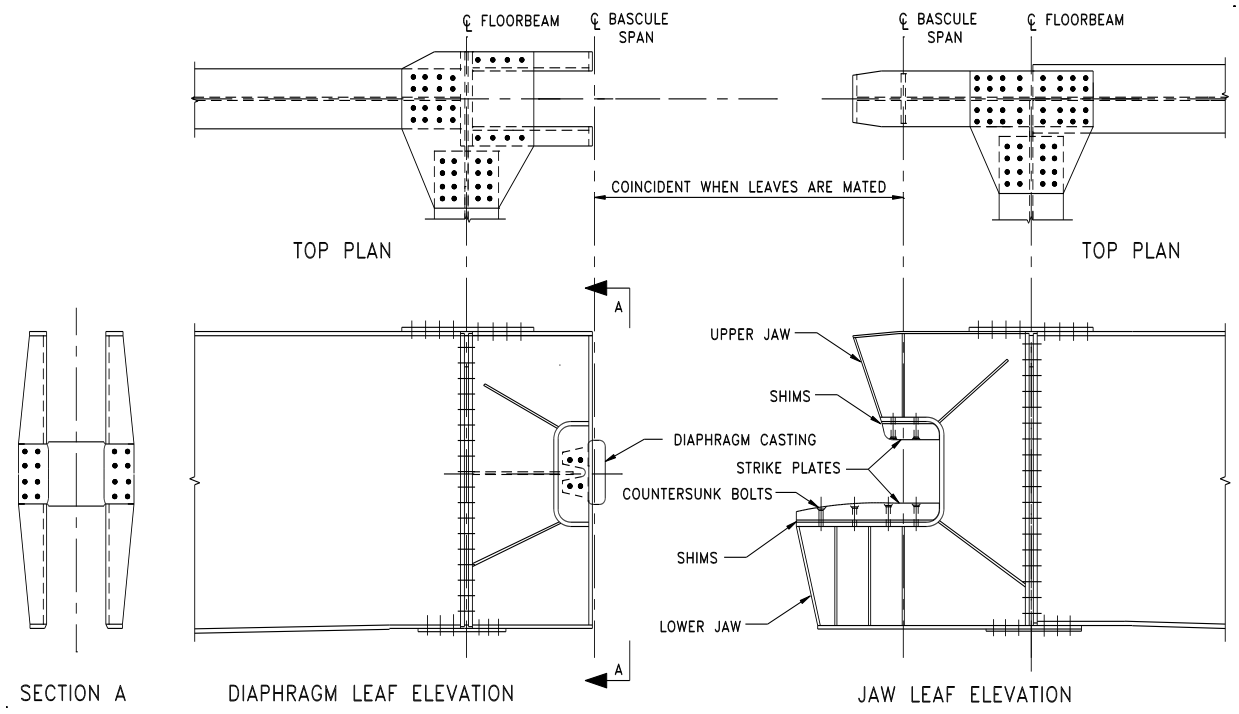
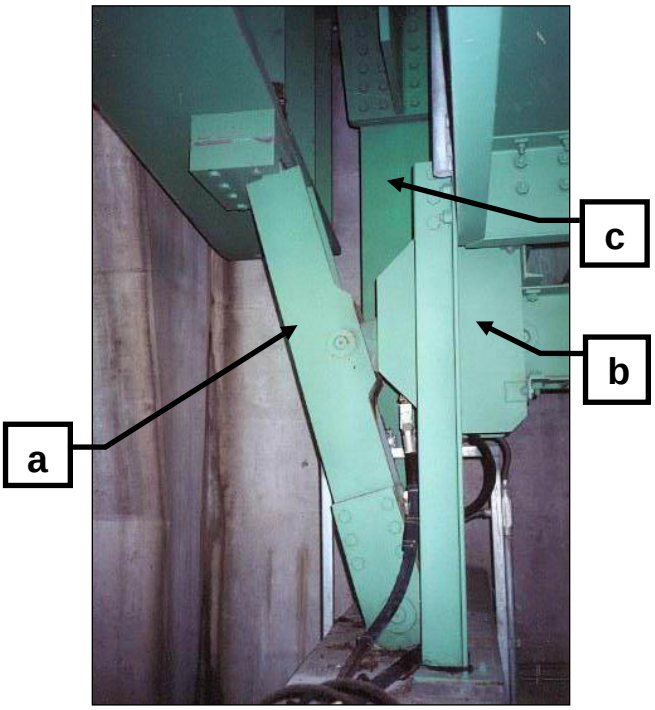
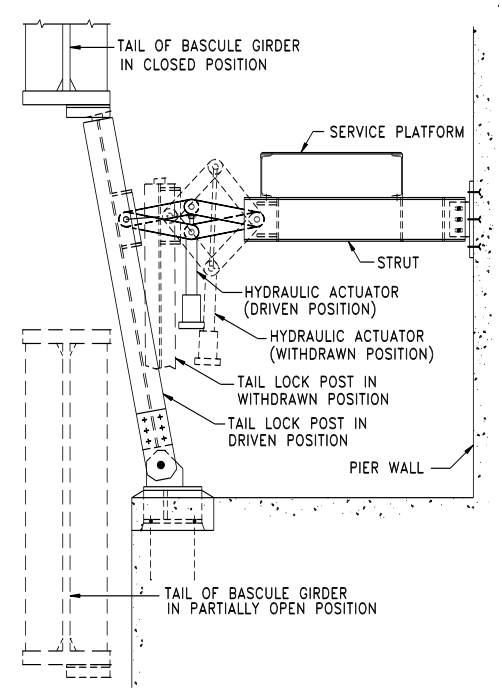


Figure 3.5.3.3-6: Detail of Mid-Span Shear Lock.

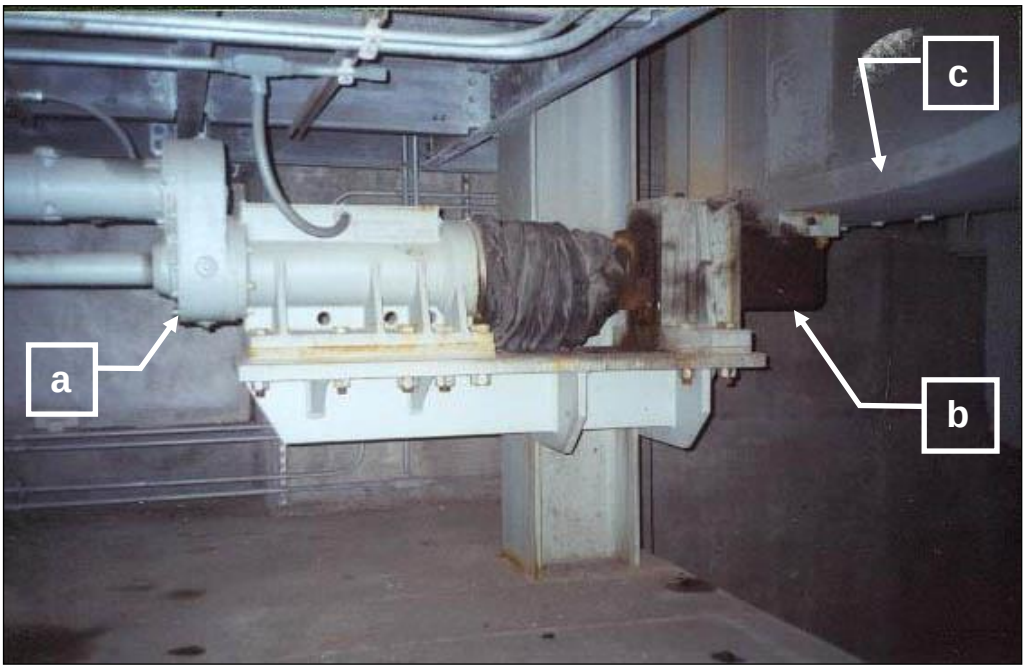




a Tail Lock Post in Driven Position
b Actuator (hidden by enclosure)
c Live Load (Uplift) Anchor Column

Figure 3.5.3.3-7: Lever Tail Lock - Eighth Street Bridge, Manitowoc.

Horizontal lockbars act as tail locks for the Main Street Bridge in Green Bay. Figure 3.5.3.3-8 depicts the installation with the leaf closed. Note that the bar is oriented normal to the bascule girder web.



a Screw-type Operator
b Tail Lock Lockbar
c Bottom Flange of Bascule Girder (closed position)

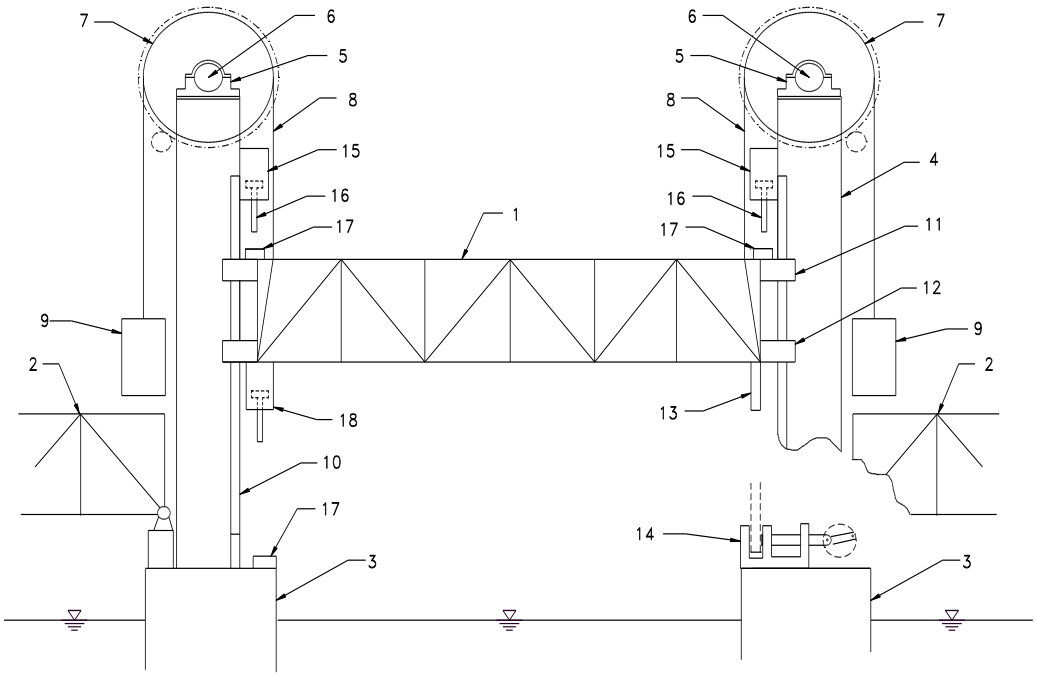
Figure 3.5.3.3-8: Horizontal Lockbar Tail Lock – Main Street Bridge, Green Bay.

3.5.3.4 Vertical Lift Bridges

Various types of vertical lift bridges are described in Chapter 3 and shown in Figure 3.3.1-1. Except for the pit drive type, the stabilizing machinery for vertical lift bridges is quite similar. Figure 3.5.3.4-1 depicts the stabilizing machinery for a tower drive that does not have auxiliary counterweights. The stabilizing machinery components include:

1. **Main Counterweight Ropes:** Although the lift span and the main counterweights are usually considered structural components, the wire ropes connecting them are classified as stabilizing machinery. They are typically 6 x 19 Class (6 x 25 filler wire construction) helical wire rope with a fiber core. Current specifications require right regular lay, but other lays were formerly used. The rope is normally prestretched to about 40 percent of the catalog strength in order to remove some of the internal wire looseness which is a result of the manufacturing process. Rope terminations are made with forged or cast steel sockets using molten zinc to form the wedges.

2. **Counterweight Sheaves and Trunnions:** The counterweight ropes are looped over grooved sheaves of cast steel or welded rolled steel. A trunnion shaft is fitted through the sheave. Because of the rotation of the shaft and sheave, while supporting full dead load, stress reversals occur and these can lead to cracking of the sheave and the shaft. Cracking usually occurs at the step in the shaft nearest the sheave if the fillet radius is not large enough to reduce the stress concentration sufficiently. Such fatigue cracking can be a serious safety concern.



1. LIFT SPAN TRUSS
2. FLANKING SPAN TRUSS
3. PIER
4. TOWER
5. CWT TRUNION BEARING
6. CWT TRUNION
7. CWT SHEAVE
8. MAIN CWT ROPES
9. MAIN CWT
10. GUIDE BAR
11. UPPER SPAN GUIDE
12. LOWER SPAN GUIDE
13. CENTERING BAR & SPAN LOCK SOCKET
14. SPAN LOCK ACTUATOR & LOCKBAR (RETRACTED)
15. UPPER BUFFER
16. PISTON ROD
17. STRIKE PLATE
18. LOWER BUFFER

NOTE: LIVE LOAD REACTIONS FOR LIFT SPAN AND AUXILARY COUNTER WEIGHTS NOT SHOWN

Figure 3.5.3.4-1: Vertical Lift Bridge – Typical Components of Stabilizing Machinery.

3. **Span Guides:** As the lift span is moved vertically by the span drive it is horizontally guided at all four corners. The guides transfer the wind loads to the towers. In order to minimize frictional resistance when the lift span is moving vertically, the guides are equipped with rollers or bronze wear plates. The rollers must be arranged such that longitudinal and lateral thermal expansion of the lift span framing is not restricted.

4. **Guide Bars:** The guide shoes or guide wheels rub or roll on machined plates attached to the tower columns. The contact surfaces should be smooth and the rubbing surfaces lubricated.

5. **Centering Devices:** The purpose of the centering devices, which are usually located along the longitudinal centerline of the lift span, is to laterally center the span as it descends to the seated position. This action is important to ensure that finger joints on highway bridges mate and especially on bridges with railway tracks where the cantilevered extensions of the running rails must be lowered into joint castings having

little lateral clearance. Sometimes, the centering device bar is designed to also serve as the socket for the span lock.

6. **Span Locks:** Span locks are often provided on vertical lift bridges to ensure that the lift span does not rise inadvertently while traffic is permitted on the structure. Such an event can occur due to control malfunction or brake release on an improperly balanced lift span.

7. **Buffers:** Buffers were installed on bridges without automatic deceleration controls in order to reduce impact of the moving mass, if the operator should lower the lift span too fast. Most buffers cushion by compressing air and discharging it slowly.

Stabilizing machinery of other types of lift bridges are similar to that for the tower lift bridge just described. Recall from Part 3 Chapter 3 that Wisconsin does have lift bridges that do not rely on mechanical equalizing systems. The Wisconsin Avenue Bridge in the City of Milwaukee is an example of a structure that uses internal sensors within the hydraulic system to stabilize the hydraulic cylinders during operation. While this reduces the number of mechanical components an inspector needs to be aware of during an inspection, it is important to operate the structure under normal conditions to verify the bridge is operating and functioning as designed.

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3.6 ELECTRICAL SYSTEMS

3.6.1 Introduction

A typical electrical system for a movable bridge comprises four major groups of equipment:

1. Power Distribution Equipment
2. Electrical Machinery
3. Control System
4. Lighting Systems

3.6.2 Electrical Inspection Tools and Instruments

Several tools that are necessary for an electrical inspection are a megohmmeter, a voltmeter, a live power indicator, an ammeter, a thermometer, and a receptacle tester. A voltmeter can be used to check the voltage on equipment and help verify equipment is de-energized. An ammeter can be used to verify the current and direction of phasing to motors, and verify desk indicators. The megohmmeter is a cable voltage insulation tester used to inspect bridge wiring, cables, and specialty cables.

The inspector should note all equipment on the bridge and the state of the equipment. Even if a piece of equipment is new and operating properly, that still has to be noted. Each piece of equipment should have a unique identifier. This name should be used to track the status of the equipment from inspection to inspection.

An electrical inspection should be done in accordance with the recommendations listed in this manual, the American Association of State Transportation and Highway Officials (AASHTO) Movable Bridge Inspection, Evaluation, and Maintenance Manual and the National Electric Code (NEC). This inspection manual is intended to augment the inspector's prior knowledge of the NEC by providing bridge specific equipment information.

3.6.3 Power Distribution Equipment

The power distribution equipment consists of electric power sources, protective devices, and distribution equipment.

The primary power source for movable bridges is a three-phase electric service from a local utility company. Typical three-phase electric service voltages are 120/240-volt, four-wire systems (usually found on older bridges) and 277/480-volt, four-wire systems. Some bridges may be furnished with ungrounded three-wire 480-volt electric services.

The electric service from the utility company is delivered from pole-mounted or pad-mounted transformers typically owned and maintained by the utility. Feeders from the transformers extend to the service disconnect. The service disconnect is a circuit breaker or fused switch, owned and maintained by the Owner, which provides overload and short circuit protection of the bridge electrical system. A utility energy consumption meter is located in the vicinity of the service disconnect or at the utility transformers.

A movable bridge electrical system may be provided with a secondary source of electric power should the primary electric source fail. To provide this redundancy in electric supply, a second electric service derived from a utility source independent of the primary electric service may be provided. The second electric service will be furnished with its own service disconnect and utility meter.

In lieu of a second utility service, a secondary source of electric power may be provided by an engine generator set. An engine generator set utilizes a combustion engine (usually a diesel engine) as a prime mover to operate an electric generator. The generator generates an electric supply with characteristics similar to the utility electric service. An engine generator set may be located permanently at the bridge site or it may be a mobile unit that is brought to the site when needed.

A transfer switch is utilized to select between the primary electric service and the secondary electric service. Typically, this switch automatically starts the engine generator set upon failure of the primary electric service and connects the generator to the bridge electrical system.

Electric power is supplied to the various motors and electrical equipment through protective devices, namely fuses and circuit breakers. Fuses and circuit breakers provide overload and short circuit protection to the electrical equipment they serve. These protective devices are typically housed in Panelboards, Motor Control Centers, and/or Enclosed Panels.

Typically, fuses are cylindrical devices that prevent fault currents by melting and preventing any current flow. They are single use items and must be replaced when they have been used.

Circuit breakers are used to protect the electrical equipment from a fault condition. Circuit breakers have elements that sense the current and are set to open the breaker if a certain limit is reached. Once tripped, the circuit breaker can be reset and used again.

A panelboard contains a group of circuit breakers to distribute power to various electrical devices. Motor control centers house circuit breakers, fuses, motor starters, motor controllers, and other equipment required to control and distribute power to motors and other equipment. Motor control centers are modular in construction. In lieu of panelboards and motor control centers, circuit breakers, fuses, motor starters, motor controllers, and other motor control equipment may be installed on an enclosed panel. Enclosed panels are of custom construction.

Depending on the electric service at a particular bridge, transformers are commonly installed on movable bridges. Transformers convert voltage from one level to another, usually to serve lighting loads or to isolate electrical noise in the electrical system.

Electrical circuits are carried from panelboards, motor control centers, enclosed panels and transformers to the electrical devices they supply power to through a raceway system. A raceway system typically consists of rigid metal conduit and junction boxes. Electrical wires, or conductors, carry electrical current and are installed inside the conduit and boxes that make up the raceway system.

3.6.4 Electrical Machinery

Electrical Machinery refers to electro-mechanical devices that operate the movable span and auxiliary devices such as locks, wedges, and traffic control equipment.

3.6.4.1 Span Motors

The movable span is provided with one or more span motors that serve as the prime mover for the span. Span motors may be of the alternating current (AC) type or direct current (DC) type. Depending on the type motor control equipment employed, the operating speed of the span motors is governed by the bridge operator or by a motor controller. A motor controller provides controlled motor speed and torque to ensure smooth movement of the movable span. The span motor and motor controller combination is commonly referred to as a span drive (electrical part of complete span drive).

The movable span is provided with span brakes to stop and hold the movable span. With modern motor controllers, the majority of braking during operation is accomplished by the span motor and motor controller. Thus, span brakes are typically utilized for holding the movable span and stopping it during emergency conditions.

3.6.4.2 Auxiliary Motors

Some movable spans are equipped with a back-up motor for operation in the event the span motors fail or are out of service. These motors are called auxiliary motors and are generally smaller motors that take longer to open or close the span because of additional speed reduction gearing. The motor controller for an auxiliary motor is typically a simple across-the-line contactor.

The auxiliary motor will either be directly coupled to the main span machinery or separated by a clutch. The auxiliary motor will either be selected by the operator at the control desk or operated locally. The clutch, if present, will then be operated (either manually or electrically) to connect the auxiliary motor.

3.6.4.3 Locks and Wedges

The different types of locks and wedges are described in detail in Section 3.5.3. The electrical equipment is very similar in each type, consisting of a motor directly coupled to the machinery and a series of limit switches to monitor the machinery. The motor controllers for lock and wedge motors are typically simple across-the-line contactors.

3.6.4.4 Warning Gates and Barrier Gates

In order to provide protection to vehicular traffic during a bridge operation, gates are used to warn approaching cars and also provide physical protection when required.

Warning gates provide the function of alerting the traffic that they must stop and indicating beyond which point no vehicle may proceed. The gates are usually equipped with flashing lights that also warn the traffic. Traffic signals, or red flashing lights, are used to initially stop the traffic at a pavement traffic line. Once the traffic has come to a complete stop, the warning gates are lowered to indicate that no vehicles may enter.

Some bridges are equipped with resistance gates or barriers that can physically stop a vehicle that is either out of control or whose driver failed to realize the bridge was opening. Resistance gates are rated for a certain size of car traveling at a certain speed. They may be equipped with flashing lights. Resistance gates are needed for bridges whose moveable spans when open do not provide a barrier to prevent vehicles from falling into the waterway. Because they provide a critical safety function, the integrity of their system when deployed is important.

3.6.4.5 Gongs or Horns

Gongs (bells) or horns are used to alert the vehicular traffic traveling on the bridge to be alert to traffic signals. They are intended to operate at the beginning and end of a bridge operation. The traveling public expects the bridge to be in the closed position. In order to prevent sudden and unexpected stops, the gongs aid in making the traveling public aware that the traffic conditions are changing. They are used in tandem with flashing lights and warning signs.

3.6.5 Control Systems

3.6.5.1 Control Desk

The control desk is where the bridge operator controls the operation of the bridge and its associated equipment. There are pushbuttons, control switches, indicating lights, meters and indicators on the desk and often a foot pedal switch mounted on the floor at the control desk.

3.6.5.2 Motor Controllers

There are many types of motor controllers, ranging from simple contactors to motor drives. The equipment may be installed in a panel or motor control center.

A standard motor controller consists of a motor protector, a contactor, and a motor overload device. A motor circuit protector is either a circuit breaker or a fuse that has a trip setting to protect the motor controller and motor. Contactors are devices that make or break current to the motor. Once the motor is connected to the current it will operate, once the current is removed it will stop. Contactors can operate the motor in a single direction, or forward and reverse directions. An overload device is a sensitive, quick acting device that will sense when the motor is drawing too much current and open the contactor to stop the motor. Motor overloads are intended to be faster in reacting to a motor fault than a circuit breaker or fuse, and more sensitive to minor faults that would not trip a circuit breaker.

There are many specialty controllers, called motor drives that provide the same functions listed above as well as speed, torque, and/or counter torque control of the motor. Motor drives use circuit boards and capacitors to generate a specific current amplitude and/or frequency, to control the motor.

3.6.5.3 Drum Controllers

A very common method of motor control, especially on older bridges, is the use of drum controllers. There are primary drum controllers used with DC motors and secondary drum controllers used with alternating current (AC) wound rotor motors.

The rotation of the drum controller handle changes the motors speed and direction. The drum controller is usually mounted on the control desk. It can either control the motor directly, or use a system of motor secondary resistors to control the motor speed.

3.6.5.4 Limit Switches

Limit switches are devices that determine the limit of travel for machinery and provide an electrical signal to stop or change operation. There are several types of limit switches, lever arm, plunger type, rotary, and proximity.

3.6.5.5 Relays

Relays are low current switching devices that provide logical control of a bridge. They can be used independently or concurrently with a programmable logic controller (PLC) system. In order to provide control for an entire bridge, multiple relays are required. The relays are generally located in a panel or enclosure.

When relays are used with a PLC control system, they are generally interposing relays. These relays are located between the PLC outputs and the equipment being controlled. They serve as a means of isolation between the PLC outputs and any electrical faults.

Relays are also used as part of auxiliary systems, such as traffic gates, for control of local equipment.

Machine tool relays are larger relays that can be repaired and modified for various logical configurations. They are bolted to panel back plates and the terminals of the relay accept wire. “Ice cube” style plug-in relays are smaller relays covered with a clear plastic cover. They cannot be modified or repaired, and must be replaced when damaged. They are mounted by being plugged into a mounting strip. Wires to electrical equipment are terminated on the mounting strip.

3.6.5.6 Programmable Logic Controllers (PLC)

PLC processors are computer controllers that provide the logical control of the bridge. They are generally rack-mounted in cabinets in the control room. There may be multiple processors in the cabinet and multiple input/output (I/O) cards in the rack. There may be multiple remote I/O drops throughout the bridge. A remote I/O drop will consist of I/O cards and communication cards rack mounted in a panel. Since they are computer technology, PLC processors can use communication networks to transmit information from a remote drop to the main processors.

The processors replace the relay logic network with a programmable logical control of the bridge equipment. The PLC generates electrical control signals through the PLC output cards. These output signals interface with the motor electrical controllers and equipment to control bridge equipment. PLC input cards supply the PLC processors with the state of equipment and provide the necessary interlocks for the processors to start and stop the bridge equipment.

3.6.6 Lighting Systems

3.6.6.1 Roadway Lighting

Many bridges are equipped with roadway lighting to illuminate the roadway for vehicular traffic. Illumination levels of the roadway vary depending upon the speed of traffic and local standards. Roadway lights are mostly pole mounted fixtures that are offset with a mast arm to overhang the roadway. On most bridges, the roadway lighting is owned, operated, and maintained by a local agency. Inspection of roadway lighting is not normally included as a part of the bridge inspection. However, if a bridge inspector observes a defect in the roadway lighting system it would be advisable to report it to the appropriate agency.

3.6.6.2 Service Lighting and Receptacles

Service lighting and receptacles are provided throughout the structure to enable work and inspection in dark areas or at night. Lighting fixtures have various types of bulbs. Receptacles that are exposed to the elements should be provided with covers.

The goal of service lighting and receptacles is not to illuminate the entire structure, but to provide light and power to specific areas and equipment.

3.6.6.3 Navigation Lighting and Signals

Navigation lighting and signals are provided to guide and alert the channel water traffic. Red lights mounted on the piers or fenders mark the channel for the boats to pass through. Alternating red and green lights mounted on the span notify the boat operator that the span is either not fully open (when the light is red) or is fully open and it is safe to proceed (when the light is green). Lights are installed in accordance with Coast Guard standards and guidelines. Proper maintenance of the navigation lights is essential for the safety of the waterway traffic.

An air horn or similar audible device is used to warn the water traffic that a bridge operation is about to start. It is sounded before opening the bridge.

3.6.7 Inspection of Power Distribution Equipment

3.6.7.1 Electric Service

Locate all points of electrical service. Most bridges will only have one point of service, while others may have utility service provided to both sides of the bridge. Some bridges may have separate service for special equipment (such as roadway lighting).

Before performing any inspection of the electric service contact the utility and arrange for power to be disconnected. Have the utility verify, in the presence of the inspector, that electric power is removed.

Perform a visual inspection of the utility incoming feeders. If the feeders are from overhead transmission lines, they can be easily viewed. Underground feeders will not be visible except at the point of entry. Check for damaged wires and missing or broken supports. All equipment should be firmly mounted. Check the line jacks to verify that none of them have blown.

Electric service will be terminated to a main incoming protection and disconnect panel. Usually this will consist of a service disconnect switch and fuse or a circuit breaker. Inspect the panels for damage, rust, debris or fluid build up. Check the wiring and terminations as described in Section 3.6.7.9. Schedule the insulation resistance inspection of the cables while they are de-energized. Look for any scorch marks or evidence of faults in the panel.

Inspect the main ground terminal. Request that the utility take a measurement of the resistance to ground to verify that the incoming service is solidly grounded. If the grounding at the utility is not acceptable, then a thorough inspection of the bridge grounding system should be performed.

3.6.7.2 Engine Generator Set

Test the generator by operating the bridge under generator power. Record the voltage of the generator while lightly loaded and when fully loaded. A full-load test should be conducted every three to five years. Observe the generator vibration during operation and compare the amplitude to the manufacturer's specifications. Inspect the airflow in the generator room and verify that the generator exhaust evacuates the room quickly and safely. Verify that the airflow in the generator area is sufficient for cooling the machinery.

Inspect the generator housing for damage, rust and corrosion. Pay close attention to the generator fuel tank and batteries for any leakage or corrosion. Inspect the generator wiring as described in Section 3.6.7.9.

3.6.7.3 Transfer Switch

There are two types of transfer switches, automatic and manual. The automatic transfer switch (ATS) should be operated by simulating the loss of normal power for an automatic switch. Upon sensing the loss of power the ATS should energize the back-up power system and transfer to that source.

The manual transfer switch should be tested in accordance with the bridge operations manual. In a manual system there will be a series of steps required to supply back-up power and operate the switch. Follow the approved procedure and verify the switch operates.

Inspect the panel and wiring as described in those sections.

3.6.7.4 Transformers

All transformers should have their exteriors inspected for damage, corrosion, lost paint or scratches. Panel doors or bolt-on covers should be closely inspected to determine whether the hinges and latches operate properly, are sufficiently lubricated, and make a tight seal when the doors/covers are sealed. There should be a gasket between the door/cover and the panel. The gasket should be continuous, springy, and compressible to the touch. If the gasket is brittle, permanently deformed, or missing in areas, the condition should be noted.

The inspector should determine if the transformer is mounted securely. The panel may be free standing and bolted to the floor, wall mounted and bolted to the wall, or wall mounted and mounted to a metal strut support. The panel mount must be secure and vibration resistant. Loose bolts or other deficiencies should be noted.

The inspector should listen to the transformer for any unusually noises. This should be done during bridge operations when it is most heavily loaded. Transformers normally have a low, quiet, buzzing sound. The temperature of the transformer should be taken and compared to the specified normal range. A record of the transformer's operating temperatures should be kept.

Oil-filled transformers should be inspected for leakage. Older transformers should be tested for polychlorinated biphenyls (PCBs).

3.6.7.5 Motor Control Centers

Motor Control Centers (MCCs) are cabinets where electrical power is distributed to end devices, and where controls for the end devices are located. Equipment is arrayed in units called buckets. Each bucket will contain one or more of the following: an overcurrent protection device, a motor controller, an overload relay, or metering equipment.

Inspect the panel, motor controllers, circuit breakers, fuses and wiring as described in those sections.

3.6.7.6 Panelboards

Panelboards are panels with only distribution circuit breakers and on older bridges, relays. Inspect the panel, circuit breakers, and wiring as described in those sections.

3.6.7.7 Enclosed Panels

All panels should have their exteriors inspected for damage, corrosion, lost paint, or scratches. Panel doors or bolt-on covers should be closely inspected to verify that the hinges and latches operate properly, are sufficiently lubricated, and make a tight seal when the doors/covers are sealed. There should be a gasket between the door/cover and the panel. The gasket should be continuous, springy, and compressible to the touch. If the gasket is brittle, permanently deformed, or missing in areas note the damage.

Verify that the panel is mounted securely. The panel may be free standing and bolted to the floor, wall mounted and bolted to the wall, or wall mounted and mounted to a metal strut support. The panel mount must be secure and vibration resistant. Any loose bolts should be tightened and the deficiencies noted.

The panel may be equipped with temperature control equipment, such as a heater, ventilation grate, and/or fan. The equipment should be operated so that the inspector can determine if it is operating properly. The ventilation grate filter should be clean and free of dust and debris.

Each panel should be solidly grounded by a conduit fitting or ground bar located in the panel. Verify that the panel is solidly grounded.

Other equipment located within the panel should be inspected as described in their respective sections.

3.6.7.8 Raceway System

The raceway system consists of conduits, conduit fittings, junction boxes and terminal boxes.

Conduits typically used on movable bridges are rigid galvanized steel (RGS) conduit, PVC coated RGS conduit and PVC nonmetallic conduit. Conduit is used to protect wire and route it from one location to another. Conduit must be supported regularly. Conduits consist of 10-foot or 20-foot sections coupled together.

The couplings in a conduit run should be checked to determine if the conduit is tightly connected. If a coupling becomes loose, the conduit sections may separate and the wires inside may become damaged.

The rigid galvanized steel conduit should have secure support at intervals not exceeding 10 feet. Nonmetallic conduit should have a support every 3 to 7 feet in accordance with the National Electric Code. Inspectors should determine if all the required supports are present and securely mounted. Each support should be inspected to determine if there are any loose screws or bolts.

Wall mounted conduit runs should be inspected for accumulation of dirt and debris between the conduit and the walls. Dirt and debris should not be allowed to build up on any conduit runs. Note any areas that require cleaning.

Conduit fittings are in-line enclosures in conduit that provide bends or taps in conduit runs. The fittings have removable plates that are screwed in place and sealed with a gasket. The plates should be removed and the gaskets inspected for a tight seal. The conduit fitting should be inspected for any debris or fluid.

Junction and terminal boxes are enclosures for the routing of wires. The boxes will be rated for various conditions, water tight, dust proof, corrosion proof, etc. The boxes must be opened and inspected. Wire and terminals are to be inspected as directed in Section 3.6.7.9. The seals around the access panels must be inspected to verify that they provide a watertight seal. Check boxes for accumulation of debris or fluid. Check that drain and breather valves are operational. Check the exterior of the boxes for rust or chipped paint. Check the conduit bushing and fittings to verify a solid and tight fight. Verify that any grounding fittings are properly installed and the ground wire is bonded to the fitting.

3.6.7.9 Wire and Cable

Bridge wires are copper conductors that carry electrical power and control to the electrical devices. A wire consists of a copper conductor (occasionally aluminum is used in lieu of copper) that is either a solid cylindrical shape or composed of several strands. Conductor sizes are based upon the amount of electrical current, or ampacity, of the load device. The more current required the larger the conductor. The copper wire is covered with insulation, rated for electrical voltage, and a jacket to protect the wire. A cable contains several insulated wires within the same outer jacket. The insulation and jacket are selected based upon the electrical voltage of the system and the environmental conditions the wire/cable will be exposed to. Refer to the National Electric Code for information concerning jacket and insulation types, wire ampacity, and conductor sizing.

The bridge should be wired in accordance with the as-built documents of the electrical system. Each wire should be designated with a wire number that is referenced on the as-built drawings.

Warning: High and medium voltage cables require special protection and must be de-energized before inspecting. Only personnel trained on such equipment should perform the inspection. See Section 3.1.3.

The primary concern with cable is insulation failure. Insulation failure is when the jacket and insulation of the conductor wears away. The copper conductors in the cable become exposed, and this can cause electrical faults or the wire to fail. Insulation failure may be caused by overloading, physical wear and tear, exposure to water or corrosive materials, or age.

While inspecting electrical cables, the inspector should visually inspect the entire length of cable for the following:

1. signs of abrasion or cracking on the cable
2. discoloration and over-heating
3. excessive bends or kinks in the wire
4. water or other moisture on the cables

Any of these conditions should be noted as they will reduce the useful life of the material.

Wire and cable are usually installed in conduits. The conduits provide additional protection for the cables. When the cable is inside conduit, it cannot be visually inspected for the entire run. The wire and cable should then be inspected at accessible points, such as conduit fittings, terminal and junction boxes, and equipment panels.

Wire and cables are terminated at devices. These terminations are to terminal strips in panels and lugs on equipment. There are three types of terminations to terminal strips: compression, fork tongue, and ring tongue. A compression terminal is simply a screw that presses onto the bare wire to make a contact. A fork-tongue or ring-tongue terminal is a device that is compression-clamped onto the wire, and the screw in the terminal strip will compress onto the tongue portion. Vibrations that occur on a bridge will cause the terminals to loosen over time. Compression terminals traditionally have the least resistance to vibration and the wires will sometimes become loose and even fall off the terminal strips. Ring-tongue terminals provide the best resistance to vibration, as the compression screw travels through the ring on the cable. If the screw becomes loose, the ring will still maintain contact.

The inspector should inspect the terminations to verify:

1. All terminals are tight and no wires are loose. Note loose terminals and terminals with problems.
2. All wires are tagged with a wire number.
3. Spot check the wiring with the as-built drawings to verify accuracy.

4. All terminals are marked with the wire number of the wires terminated on it.

5. Any movement or vibration between the panel and wires damaging the wires.

6. Corrosion or rust on the terminals.

Loose terminals will cause heating in the wires and reduce the useful life of the equipment.

When a wire is terminated, the insulation is removed from the bare copper. Verify that all wires do not have so much insulation removed that the exposed copper conductor will come into contact with exposed metal.

During each inspection, the insulation of the wiring should be tested. The method to perform the test is to isolate the wire from power and sensitive equipment, and have an insulation megohm resistance to ground test performed on each individual wire. Testing the cables in this manner, or "meggering" the cables will indicate the state of the insulation. A table should be made recording the wire number and the phase to ground resistance, and a phase to phase resistance with an adjacent disconnected wire. If the resistance value is below one kilo-ohm, the wire is close to failing. If the resistance is zero, the wire has failed. The results of the testing should be compared to previous results to determine if there are any trends in the insulation resistance or the cable test results. This may indicate a problem that has not yet been identified in the run of wire.

3.6.7.10 Specialty Cables: Flexible and Submarine Cables

Specialty cables are installed in areas that cannot be serviced by wire in conduit. These include flexible cables and submarine cables. All specialty cable should be inspected as described in the above sections and with the following additional inspections:

Flexible cables are cables routed between fixed portions to movable portions of the structure. Examples are cables from the pivot pier to the swing span, cables from the rest pier to the bascule leaves, and cables from the tower to the lift span. Flexible cables are designed to be very flexible. Flexibility comes at the cost of reduced jacket protection. In order for the cables to bend and move with the bridge, the jacket must be softer and more flexible. This means that they may wear more quickly from rubbing and abrasion.

Submarine cables are cables that are routed into the channel through the water from the near side to the far side of the bridge. The cables are usually trenched into the riverbed. These cables are exposed to a much harsher environment than regular cables. The portions of the cables that remain continually underwater, or continually out of the water, usually remain undamaged. The portions of the cables that are exposed to fluctuations in water level due to wet and dry periods, and the wear and tear of moving in response to the changing water level, require the closest inspection. Submarine cable is typically manufactured with a steel armor wire wrapping and polyethylene covering to protect it from the harsh conditions.

Submarine cables are usually terminated in panels where the wiring is transitioned to normal wire and conduit. The armor should be terminated and grounded at the submarine cable terminal panels in special fittings.

Flexible cables should be inspected during an operation of the bridge to verify their range of motion. The cables should swing freely and move freely during the entire operation of the bridge. There should be no sharp bends or kinks in the cables during operation. If the cables seem to snag on or rub against the structure or equipment, this should be noted. Megohm resistance test the insulation of the individual wires of the flexible cables and record the values.

During the operation, note the effects of wind on the cables. Some installations will be protected from the effects of wind, but in areas where the cables are exposed, consider how the wind will likely blow the cables and assess potential problems.

Check the cable grips and supports at each end of the cables. They must have a firm grip on the cables and be solidly attached to the structure.

Submarine cables should be visually inspected, where they are accessible. The inspector should coordinate inspection with low water conditions to see the portion of the cables at the water line. Note any deterioration of the cables. Verify panel terminations and cable supports. When the submarine cable is armored, inspect the armor clamps to make sure the cable is supported and grounded. Megohm resistance test the insulation of the individual wires of the submarine cables and record the values.

3.6.8 Inspection of Electrical Machinery

3.6.8.1 Motors

Motors have external features and internal features to inspect. Inspection should begin by verifying the motor shaft is free from oil and grease from the bearings. Leaking oil can indicate poor seal or misalignment of the shaft. All keys, bolts, and pins should be inspected to verify they are in their proper position. All bolts along the motor housing should be checked for proper tightness. Where applicable, space heaters should be checked for proper operation. This can be accomplished by touching the motor to determine if it is warm before operation. Painted surfaces should be checked for signs of corrosion.

The operation of each motor should be observed during opening. Motor shafts should be checked to determine if there is normal end play. Motors should be examined to verify that they are smooth running and free from vibration. Motors and bearings should be checked for overheating. Any unusual noises heard during operation should be noted. If the motor is fan cooled, check for proper operation of the fan and that the motor is being properly cooled.

Each motor should be wired in accordance with the National Electric Code (NEC) and there should be a disconnect switch within sight of each motor. If a disconnect switch is not located within sight of each motor, this condition should be noted. After disconnecting the motor from its power supply, check the internal equipment. Electrical connections on the motor should be checked for proper attachment.

A dielectric test should be performed by using a megohm meter test insulation resistance values on all motors. Megohm measurements should be taken from phase to ground and between phases for all AC, three phase motors. Megohm measurements should be taken at collector-rings to detect cracked or otherwise defective bushings and collars for all DC motors; this is done by testing the insulation resistance of ring to shaft and studs (leads) to

rings. Readings should be taken using a 500 Volt DC hand crank or battery operated megger. Results of the megohm meter tests should be recorded and compared to prior inspection findings. Any large changes may indicate motor deterioration. The inspector should recommend overhauling a motor when megohm values for phase to ground values are projected to reach 2.0 or less before the next scheduled inspection. If the megohm values are 1.0 or less, the inspector should recommend that the motor be overhauled as soon as possible. The inspector should check the phase currents flowing in motors under loaded conditions with a clamp-on ammeter for motors 1 horsepower or larger. The results should be recorded and should be compared to the nameplate data and prior inspection results.

With the power disconnected, the interior of the motor should be inspected by opening the inspection ports of the motor. The collector-rings (slip-rings) should be free of carbon, metal dust, discoloration, and deformation. The wearing surface of the collector-rings should be smooth, highly polished, and free of dirt, oil, grease, and moisture. If any of these detrimental conditions exist, the inspector should try to determine the source(s).

Wound rotor alternating current (AC) motors, synchronous AC motors, and direct current (DC) motors use brushes to carry current to rotating parts of the motor. The brushes should be inspected. The brushes should have free movement within their holders. Each brush holder should be set so the face of the holder is approximately 1/8 inch from the collector-ring. When removing brushes for inspection, each brush must be reinserted into its original holder and in its original orientation. It may be helpful to scratch a mark on one side of the brush when removing it to indicate its proper location and alignment. All brushes should be inspected for wear. If the remaining portion of any brush within its holder is ¼ inch or less, the inspector should recommend that all brushes on the motor be replaced. The entire surface of the brush that rides on the collector-ring should display a polished finish – this indicates full surface contact. If evidence is observed that indicates that a brush is not making full contact over its entire surface, the inspection report should recommend that the brush be re-seated. The springs that push the brushes against the collector-rings should be inspected. The brushes should be held firmly on the collector-ring and all brushes should be held with approximately the same pressure. Improper spring pressure may lead to collector-ring wear or excessive sparking. If this condition is found the inspector should recommend that the springs be replaced. Excessive heat may anneal brush springs. If this condition is observed, the inspector should try to determine the cause of the overheating.

3.6.8.2 Span Brakes

Brakes should be equipped with covers, to prevent debris or grease from affecting brake operation. Check the mounting and location of limit switches on the brake. Generally a brake will have a set, released and a hand released limit switch. Follow the inspection methods listed in Section 3.6.9.7.

Limit Switches: Check the wiring in accordance with 0, Section 3.6.7.9.

Manually operate the hand release arm to verify that the linkages work properly. Check the clearances between the brake shoes and the drum when the brake is released. Observe the drum to note the wear pattern. If the drum is wearing evenly, the entire drum should be shiny from the wear. Uneven wear will allow sections of the drum to rust and corrode while other

sections do not. Make sure no grease, oil, water, or dirt is on the brake drum, as this will reduce braking capacity.

During a bridge operation, time the length of operation for the brake to fully release and the brake to set. Monitor the brake shoe and drum during operation. If the shoe and drum are not aligned, they will come into contact during operation. This contact could produce smoke and damage the brake.

The insulation of the brake motor should be tested with a megohmmeter and the results recorded.

3.6.8.3 Auxiliary Motors

The auxiliary motor(s) should be inspected in accordance with Section 3.6.8.1 Motors.

Equipment associated with auxiliary motors should be inspected. The main component of the auxiliary equipment is the motor clutch. The clutch should be operated to verify proper electrical control and range of motion. Verify that interlocking limit switches are triggered by clutch operation.

3.6.8.4 Warning and Barrier Gates

Check the exterior gate housing for any damage and the access panels for proper operation. Open the housing and inspect for fluid or debris accumulation. Closely inspect conduits entering the base of the housing. Oil leaks may flow into the conduits and damage the wiring and environment.

Inspect the internal equipment per their respective sections; the wire and terminations in accordance with Section 3.6.7.9, the limit switches in accordance with Section 3.6.9.7.

Limit Switches: The motors in accordance with Section 3.6.8.1, and other applicable sections.

Observe the gate arm or barrier during an operation. Verify that the gong operates and the flashing lights blink for the duration of the arm's movement. The lights should start operating when the warning signals are activated and operate until the locks, jacks and wedges are released when stopping traffic. They should operate from the time the locks, jacks, and wedges are engaged until the gates are raised. Observe the cables powering the flashing lights on the arms. Verify the cable is not rubbing against or catching on the gate housing during movement. Inspect the arm for any frayed wire or exposed terminal on the flashing lights. This could pose a danger to a pedestrian.

3.6.8.5 Gongs

Gongs are mounted on traffic gates for oncoming traffic. Gongs should be observed during a bridge operation. The gongs should start operating when the warning signals are activated to stop traffic and should continue to operate until the locks, jacks, and wedges are released. They should operate again from the time the locks, jacks, and wedges are engaged until the gates are raised. The gongs should make a loud, audible sound. The cables powering the gongs should be inspected for any abrasion or tears.

3.6.9 Inspection of Control Systems

3.6.9.1 Control Desk

Usually there is a switch or pushbutton that will test all the indicator lights on the desk. The switch or pushbutton should be activated to test the lights and verify they all work. Any light that operates improperly should be noted in the report.

The bridge should be operated several times to verify that all pushbuttons, control switches, indicating lights, meters, and indicators operate properly. A functional checkout will indicate whether the equipment is working properly. Voltmeter, ammeter, and kilowatt meter readings should be recorded as the bridge is operated. Readings should be compared to the records from previous inspections. Dramatic changes in readings may indicate problems and aid with the inspection.

As part of the inspection, several of the bridge operators should be interviewed to determine if they have experienced any problems with the controls. The operators may also relate any other problems that they have experienced on the bridge that may aid in the overall inspection.

The interior of the control desk should be examined. Verify that the interior light is working, and note if the light is inoperable or missing. Check for any loose wires and tighten terminations, and inspect the wiring in accordance with the section Wire and Cable. Look for any scorching or discoloration that could indicate a faulty piece of equipment. Inspect all interior equipment. Check all relays to verify that they are firmly installed, especially plug-in type relays. Check for a strip heater and verify that it is operational.

3.6.9.2 Interlocks

Interlocks are a mechanism that provides protection to both the bridge users and bridge operating equipment by ensuring various elements of bridge operation occur in a specific sequence, order and condition. A series of tests should be made to verify that the interlocks in the control system are operating properly. Extreme care must be taken while verifying the interlocks. Vehicular traffic must be stopped by flagmen while testing roadway equipment. River traffic must be made aware of the testing and any potential delays. Testing must be reviewed and made applicable to the particular bridge inspection. The testing shall be performed in accordance with the American Association of State Transportation and Highway Officials (AASHTO) Movable Bridge Inspection, Evaluation, and Maintenance Manual, as follows:

With the bridge in the closed position:

1. Prior to sounding the horn or activating the warning lights attempt to lower the traffic gates. The gates should not lower.

2. With the traffic gate and barrier/resistance gate arms open to vehicular traffic, insert gate arm hand crank into the traffic gate housing and try to operate the gates from the console. The gate should not operate. Record results and repeat for all gates.

3. With the traffic gates open to roadway traffic, attempt to operate the barrier/resistance gates. The barrier/resistance gate should not operate if the associated traffic gate(s) are not closed to traffic. Repeat for all gates.

4. With resistance gates open to roadway traffic, attempt to disengage locks, jacks, and/or wedges. They should not operate. Repeat for all locks, jacks, and/or wedges.

5. With locks, jacks, and/or wedges in place, attempt to operate the bridge span. The span should not operate, record the results.

At the appropriate point in bridge operation:

1. Confirm that if a hand crank is inserted into any device (lock, jack, wedge, etc.) the motor for that device will not operate.

2. Confirm that the main motors cannot be started prior to the release of the brakes. The main drive motor starters should not engage – record the results.

3. Test limit switches at fully open.

Additional recommended tests are:

1. Attempt to raise the gates and turn the traffic signals to green before the locks, jacks, and/or wedges are fully driven and the bridge span secure.

2. Verify that the traffic signals cannot be changed to green until all gates are raised.

Note any problems in the interlocking and clearly notify the operators of the problems. The operators must be aware of any issues in control of the bridge.

3.6.9.3 Fuses

Inspect fuses by verifying the fuses are the proper current rating. The fuse ampacity should be printed on the side of the fuse and the correct ampacity of the system should be documented on the as-built wiring diagrams. If the fuse ratings are different from the as-built documentation, check the load equipment. If the equipment protected by the fuses has changed, the new equipment may require a different fuse size. Verify that the fuse ratings are accurately documented. Inspect the fuse terminals for a tight and good electrical fit. Look for corrosion or scorch marks on the fuse blocks. Occasionally a fuse will blow and no replacement fuse is available. The inspector may find that a piece of wire has been used to jumper a fuse, leaving the equipment unprotected but operational. This condition is not acceptable and must be reported. Note any missing fuses. It would be good practice for the bridge owner to maintain a supply of extra fuses.

3.6.9.4 Circuit Breakers

Verify that the trip settings on the breakers are accurate. Compare settings to as-built documentation and equipment ratings.

Molded-case circuit breakers are not accessible, due to their plastic cover. Air circuit breakers should have their arc chutes inspected for debris, missing hardware, damaged

chutes. Check the contact surface for corrosion, pitting, and damage. Operate the circuit breaker to determine whether the contacts make and break contact.

Check wiring and terminations in accordance with 0, Section 3.6.7.9.

3.6.9.5 Motor Controllers

There are many types of motor controllers ranging from simple contactors to motor drives. The equipment may be installed in a panel or motor control center.

There are many specialty controllers, called motor drives that provide the same functions listed above as well as speed, torque, and/or counter torque control of the motor. Given the diversity of this equipment and the varied manufacturers that supply motor drives, the inspector should review manufacturer's specific written information on the drives on the specific bridge and follow their inspection recommendations.

The inspector should first inspect the panel that the motor controls are mounted on for any fluid or debris build up. Any scratches or damage to the panel exterior should be noted.

The wiring and terminations should be checked as described in Section 3.6.7.9. The circuit breakers and fuses should be inspected as described in their respective sections.

Motor contactors should have their wiring inspected as described in Section 3.6.7.9. The individual contacts should be inspected for corrosion and scorching.

3.6.9.6 Drum Controllers

The drum switch should be inspected for any corrosion or debris. The wires and terminations should also be inspected as described in Section 3.6.7.9. Inspect the drum switch contacts for good solid contact while being operated and any scorching or corrosion.

3.6.9.7 Limit Switches

All limit switches should have their wiring and enclosures inspected. Note any scratches or damage to the switch exterior. Where accessible, open the limit switch and inspect the wiring as describe in Section 3.6.7.9. (Some limit switches are factory sealed and should not be opened.) Inspect the seal of the limit switch and verify that no fluid or debris have accumulated in the housing. The limit switches should all be securely mounted and have little movement or play.

Lever arm limit switches and plunger type limit switches have arms that move to trigger the electrical contact. Lever arms rotate around a pivot point in the housing and plunger type are pushed into the housing. Inspect the arms for debris, corrosion and a buildup of dirt. Verify that the arms move freely and do not stick in place. While testing the arm, the making and breaking of the limit switch contacts should be audible. During a bridge operation watch the limit switch. When safe, manually operate the switch to test whether the operation will stop or the appropriate indicating light energizes.

Proximity limit switches are generally magnetic sensors that make electrical contact in the presence of a metal trigger. The limit switch magnetic sensor should be inspected for a

buildup of magnetic filings that may provide a false indication. When safe, manually operate the switch to test whether the operation will stop or the appropriate indicating light energizes.

Rotary limit switches should be opened and the contacts inspected. Any corrosion or scorching should be noted. Bearings should be checked for proper lubrication. The rotary limit switch is coupled to the span drive gearing and these couplings should be inspected for proper connection.

Position indicators, selsyn transmitters, resolvers, and tachometers are all feedback devices that provide position or speed information to the operator or to the motor drives. The enclosures, wiring, and mounting should be inspected as described above.

3.6.9.8 Relays

When inspecting relays, the inspector should verify that they are securely mounted; especially plug-in relays. All wires and terminals should be tagged and identified. Check for any “jumper” wires that are not part of the logical control system. These wires are added to bypass logical control temporarily and should be removed when the equipment has been repaired. Note any wiring without tags that is not documented on the as-built drawings. Determine what equipment the relays with jumpers control, and pay close attention to the interlock testing on the control desk when testing that equipment. Relay identifiers should be on nameplates mounted adjacent to the relays, should cross reference the as-built wiring diagram.

Inspect the individual relays for contamination, scorch marks or discoloration and record any relays with these problems.

The relays should be monitored during a bridge operation to verify proper operation. The inspector will be able to hear the relays “pull in” or become engaged. This should be a short sharp click sound. Chattering relays indicate a faulty relay and the relay should be noted. Use a clock to determine whether timing relays are operating properly.

3.6.9.9 Programmable Logic Controller (PLC)

Care should be taken whenever inspecting a PLC processor. Small switches on the processor and I/O cards called dip switches are configured to allow proper operation. Changing these switches can interfere with proper PLC operation. The inspector should be familiar with the type of processor and not change anything during the inspection. As part of the familiarization process, the inspector should review any manufacturer’s manuals for specific maintenance issues with the particular type of PLC installed on the bridge.

The inspector should inspect the processors, I/O cards, and remote racks for any dust, dirt, debris, corrosion, or fluid on the equipment. The inspector should check the PLC diagnostic lights to see if there are any failures in the equipment. All terminals and wires should be inspected as described in Section 3.6.7.9. Inspect the cabinet for debris, fluid, and clean air filters on the fans. Check the PLC batteries and make sure they are fully charged.

Other equipment in a PLC panel may include fuses, circuit breakers, heater unit, lights, fans, and relays. Verify proper fan, heater, and light operation. Inspect the filter on the fan for

accumulation of dirt and debris. Inspect the other equipment as described in their various sections.

3.6.10 Inspection of Lighting Systems

3.6.10.1 Roadway Lighting

The roadway lighting system may be owned and operated by the owner of the bridge or it may be owned and operated by another agency. If the lighting system is to be inspected as a part of the bridge inspection, the following items should be inspected.

The inspector should inspect the roadway lights at night in order to determine whether the lights operate properly. The inspector should measure and record the intensity of illumination on the roadway at points on a grid.

The inspector should open the roadway light's transformer bases to inspect the wire and cable and the enclosure. The enclosure should be free of debris and fluids. Any damage to the base or pole should be noted.

3.6.10.2 Service Lighting and Receptacles

As a part of the bridge inspection, the inspector should test the service lighting throughout the whole structure. Note any damaged or inoperative light. Lights in machinery areas should be equipped with guards and globes. While inspecting, attempt to determine whether the fixture or the bulb is inoperative. A simple method is to carry a typical light bulb during the inspection to test the fixture.

Use a receptacle tester to verify that the receptacles work and are wired properly. Note any damaged receptacles or any exposed receptacles lacking covers.

3.6.10.3 Navigation Lighting and Signals

Navigation lights will be located along the piers or fender system of the bridge and on the span. The fender navigation lights are red fixtures, while the lights on the span may vary between red only and red and green alternating fixtures.

Each navigation light should be checked for damage, broken lenses, loose mountings, and corrosion. The navigation lights should be tested for functionality. This may require a control switch on the desk be used to activate the lights or a night inspection. Each light should be clearly visible when lit. Note any fixture that is inoperative or damaged. The navigation span lights may rotate along a hinge and this should be inspected for proper range of motion.

Signals consist of horns or public address equipment used to alert waterway traffic. The equipment should be visually inspected for any damages or corrosion, enclosures opened and the wiring inspected as described in Section 3.6.7.9, and subjected to a test to verify operation. Bridges may have multiple signal devices and all must be inspected.

3.6.11 Inspection of Remote Operation System

Some of the Moveable Bridges in Wisconsin have been provided with the ability to remotely operate from an operator house located on another nearby moveable bridge. The remote operations provide for a significant savings in operator costs at locations where multiple roadways cross a navigational waterway in near proximity to one another. Remote operation requires US Coast Guard permission. Equipment for these systems include control console at the remote location; camera and camera monitors for providing comprehensive views of the roadway, sidewalks and waterway; bridge–to-bridge communication signals at the bridges for remote operations; and PA systems for communication to bridge users and mariners.

Bridges equipped for remote operation should be operated in all modes from the remote site. Camera monitors should be inspected for providing clear unobstructed views without blind spots. For camera systems with recorders, those devices should be verified for proper recording of camera images. PA systems should be tested to confirm their operation.

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3.7 HYDRAULIC SYSTEMS

3.7.1 Introduction

There are two general types of hydraulic systems used in movable machinery applications. These are the “Open” and “Closed” loop hydraulic circuits, which are described subsequently. More detailed descriptions may be found in “Using Industrial Hydraulics” by T. C. Freudenthal (1984).

3.7.1.1 Open Loop Circuit

In open loop circuit applications, the pump draws fluid from a reservoir and pushes this fluid into the hydraulic system. After passing through the control valve circuitry and the actuator, the fluid returns to the storage reservoir. Typically, the reservoir is sized so that it will hold a minimum of three times the volume that can be displaced by the pump in one minute. For example, a 30-gallon per minute pump would be mated with a 90-gallon reservoir.

An open loop pump only pumps fluid in one direction. For this reason, an open loop pump is normally supplied with a large diameter low pressure inlet port and a smaller high pressure outlet port. In open loop circuit design, the direction of the actuator’s motion must therefore be accomplished by the use of directional control valves.

Open loop hydraulic circuits are the most common in movable bridge applications. This is because most hydraulic movable bridges are actuated by hydraulic cylinders, which require large volumes of fluid as well as differential flow rates in and out of the hydraulic cylinders. Another advantage of the open loop circuit is that several different actuator functions can be performed simultaneously from a single pump. A disadvantage of the open loop circuit design is its relatively large size and weight due to the large volume of oil required.

3.7.1.2 Closed Loop Circuit

In a closed loop circuit design a single hydraulic pump is used to drive one or more hydraulic motors. The closed loop circuit is not viable for hydraulic cylinder applications because of the different fluid volume displacements during extension and retraction. The fluid that passes through the actuator is returned directly to the low pressure side of the pump. For proper operation, the pump must receive the same quantity of oil at its inlet as it is pumping from its outlet.

A charge pump is always used in a closed loop hydraulic circuit. The charge pump is usually a small fixed displacement pump (usually about 15% of the displacement of the main pump). The charge pump always works on the low pressure leg of the main loop pumping filtered fluid into the loop. The pressure in the low pressure leg is maintained at a value of usually between 100 to 300 psi by a relief valve.

During operation, the main pump control can cause the pump’s displacement to “go over center,” which means that the main pump can pump high pressure oil from either of its two main ports. In other, words, it can cause a clockwise or counterclockwise flow of fluid through the main loop plumbing. This will allow the actuator to operate in either direction of rotation.

In closed loop systems, pressure, flow, and directional control are all achieved by the controlling elements of the pump. Cross port relief valves are incorporated to protect the actuators from load induced pressure peaks.

The advantages of the closed loop system are that high horsepower systems are compact, they operate with a minimum amount of excess fluid storage, and they are highly efficient. The pump controls direction, acceleration, deceleration, speed, and torque of the motor actuator, and hence pressure and flow control components are not needed. These systems also provide excellent dynamic braking control, which is highly desirable in most movable bridge applications.

3.7.2 Hydraulic Control Systems

There are many types of hydraulic system controls. Hydraulic system control refers to the way in which flow and pressure is regulated from the system's pumps. In general, the three types of hydraulic system controls used in movable bridge applications are "constant horsepower control", "electronic proportional control", and "hydraulic cylinder control". Each is described below.

3.7.2.1 Constant Horsepower Control

Constant horsepower control (also known as horsepower limiting) uses a prime mover (electric motor) which drives the pump at a constant speed. The intent is to keep the motor working at a constant horsepower level. To maintain constant horsepower, the mathematical product of flow and pressure must be a constant value. Therefore, if the flow is high, the operating pressure must be low. Since the operating pressure level of a system is dictated by the load conditions, the flow must vary with changes in load induced pressure to maintain the product of flow and pressure at a constant value.

Constant horsepower controls sense the load induced pressure in the systems and regulate pump flow accordingly. The pump control holds the pump at its maximum displacement until the pressure reaches the point at which regulation or "compensation" begins. This type of system always uses a system pressure relief valve. Once the end of regulation is achieved, the slightest increase in system pressure will open the relief valve and bypass the minimum pump flow back to the tank.

3.7.2.2 Electronic Proportional Control

Electronic proportional control utilizes a proportional solenoid to vary the pump displacement. The pump varies from minimum to maximum displacement proportionally to the current of a 24 volt DC command signal. As DC current is applied to the proportional solenoid, the solenoid pushes the pilot spool with a specific force. When the current, and therefore the force, is high enough the pump begins stroking and producing flow. Further increases in signal current will increase the pump's output flow proportionally.

3.7.2.3 Hydraulic Cylinder Control

Hydraulic cylinder control utilizes a hydraulic adjusting cylinder and pilot system to control pump flow. The pilot circuit is designed to meet the requirements at hand. This type of control can be used in both open and closed loop hydraulic circuits.

3.7.3 Interlocking

Control systems on movable bridges are provided with interlocking controls that cause the bridge to be operated in a manner that provides safety to the traveling public while protecting the equipment. The requirements for interlocking controls between the hydraulic span drive and other equipment such as; span locks and traffic gates and signals are discussed in Section 3.6.9.2.

3.7.4 Inspection of Hydraulic Components

3.7.4.1 Accumulators

The purpose of an accumulator is to store energy in the form of pressurized hydraulic fluid. Although there are several different types of accumulators, gas charged accumulators are the most common.

A gas charged accumulator stores hydraulic fluid under pressure by compressing an inert gas (usually nitrogen). A rubber bladder separates the gas chamber and the oil chamber. Initially, the oil chamber is vented to atmospheric pressure and the gas chamber is pre-charged with nitrogen to a known setting through a gas valve.

Generally, accumulators are maintenance free. The nitrogen pre-charge should be checked at least every 6 months as follows:

> Close the isolation valve between the system and the accumulator. Slowly vent the accumulator using the venting needle valve. By carefully watching the pressure gauge, a gradual decay in pressure will be seen as the fluid empties. However, the moment the accumulator rids itself of all the oil, the needle on the pressure gauge will immediately drop to zero. The pressure from which the needle drops is the pre-charge pressure.

If the nitrogen pre-charge is below that value required by the system design, then the accumulator should be recharged. Recharging an accumulator can be highly dangerous and should only be performed by a qualified technician who is fully trained to properly and safely perform this procedure!

3.7.4.2 Valves

Modern hydraulic system valves are very reliable. Considering the relatively low usage of movable bridge hydraulic systems, problems with system valves should be minimal. However, dirty system fluid or unintentional maladjustment could lead to problems with hydraulic valves. The inspector and/or maintenance staff should periodically perform the following for the system valves:

Pressure Relief Valves

1. Verify that pressure settings on all relief valves are correct. Adjust as necessary.

2. Check for leaks with manifold interface. Replace seals/O-rings as necessary. Re-torque valve body mounting bolts or valve cartridge.

Directional Control Valves

1. Verify that the spool moves freely. Verify that manual override pushbuttons are functional.

2. If spools are sticking, disassemble valve and inspect internal components for wear, scoring, and fluid debris.

3. Check for leaks with manifold interface. Replace seals/O-rings as necessary. Re-torque valve body mounting bolts.

4. Check condition of pilot lines, if applicable.

Shut Off Valves

1. Inspect all shut off valves and their connections to the system piping for leaks.

2. Verify that these valves are fully open or fully closed as required by the system.

3.7.4.3 Hydraulic Cylinders

Hydraulic cylinders are used for span motion or actuation of locking devices. Cylinders should be periodically inspected to ensure proper, safe and long term operation. The inspector and/or maintenance staff should:

1. Observe extension and retraction of hydraulic cylinder rods. Span or locking device movement should be smooth.

2. Inspect condition of cylinder rod coating. Look for scoring, nicks, or other surface imperfections that could damage the rod seals.

3. Inspect area around rod seals for fluid leakage. A small amount of fluid is not cause for alarm. If leakage is excessive, the rod seals should be replaced.

4. Inspect cylinder valve manifold blocks and piping attachments for leaks. Correct as necessary. Verify that flexible hoses do not scrape anything due to cylinder movement.

5. Inspect all connections of the cylinders to the structure or locking device. Verify that all bolted connections are tight. Verify that cylinder end pin connections have freedom of movement.

6. Verify that cylinder attachments have freedom of movement throughout the entire operating range.

7. Lubricate cylinder end bearing/pin connections monthly. Wipe off excess lubrication.

3.7.4.4 Pumps

Periodically inspect system pumps and observe/listen during operation. Operation should be smooth. If equipped with a flow meter, verify that the pumps provide the flow as required by the design. The inspector and/or maintenance staff should:

1. Inspect pump suction, high pressure lines, and case drain lines for leaks. Tighten as necessary. Verify that suction shut off valves are fully open, if equipped.

2. Check tightness of bell housing to pump and motor to bell housing connections. Check tightness of shaft coupling assemblies. Check tightness of other pump/motor mountings.

3.7.4.5 Rotary Motors

Periodically inspect the system's hydraulic motors and observe/listen during operation. Operation should be smooth. The inspector and/or maintenance staff should:

1. Inspect pressure line and case drain line connections for leaks. If found to be loose, the inspector should note the condition and recommend that they be tightened. Check all housing joints for leaks. Verify that suction shut off valves are fully open, if equipped.

2. Check tightness of hydraulic motor to its support and mating equipment. Check tightness of shaft coupling assemblies, if applicable.

3.7.4.6 Filters

Adequate and proper system filtration is the most important aspect of maintaining any hydraulic system. The environment that movable bridge hydraulic systems must operate in makes proper system filtering even more critical. Without proper system filtration, degradation as well as catastrophic failure could result.

Degradation failure of system components results from the abrasiveness of tiny particles wearing the close tolerance surfaces of internal components. Degradation failure spreads throughout the system, and is usually not detected until the damage is irreversible. Indication that degradation failure has occurred include sluggish system response, loss of speed adjustment accuracy, inability of the system to produce full load, and/or overheating.

Catastrophic failure is the immediate failure of a system component and is usually related to large particles causing moving parts to jam or stick. In pumps, dirt can block lubrication passages and cause pump failure. Large debris can collect in orifices which supply oil to the pilot circuit of pilot operated relief valve, pressure compensated flow controls, etc.

Considering the fact that contaminants cause failure, positive steps must be taken to ensure proper system filtration.

Suction Filter

Suction filtration is achieved by locating a filtering element between the reservoir and the pump. Generally, the strainer is well below the minimum oil level within the reservoir. This however, makes servicing suction filters very inconvenient. Consequently, it is not unusual for these filters to go unserviced until they starve the pumps and cause cavitation damage.

Hopefully, a system is equipped with suction filtration access holes that are present in the reservoir to allow servicing of the suction filters without having to drain the reservoirs. Biannual cleaning/replacement of these elements is recommended.

Pressure Filter

A pressure filter is an element contained in a housing that is subjected to the full system pressure and flow. Pressure filters are commonly used to protect high pressure components such as directional spool valves and piston type hydraulic motors.

Proper inspection/maintenance/replacement of pressure filters is critical because if a clogged filter element was to rupture in service, a large concentration of contamination would dump directly into the components it was supposed to protect.

The following should be regularly performed on system pressure filters:

1. Visual clogging indicators should not be relied upon. Pressure filter elements should be removed monthly and inspected. Filter elements should be replaced as necessary.

2. If equipped with electronic clogging warnings, filter element should be replaced immediately upon receiving a system warning of a clogged filter.

3. As a minimum, all system pressure filters should be replaced annually.

Return Line Filters

Return line filtration is based on the assumption that a clean hydraulic system will remain clean if the contamination is filtered out of the fluid soon after it is ingested or created by the system.

The following procedures should be performed regularly on system return line filters:

1. Visual clogging indicators should not be relied upon. Remove and visually inspect return line filter elements monthly. Filter elements should be replaced as necessary.

2. If equipped with electronic clogging warnings, filter element should be replaced immediately upon receiving a system warning of a clogged filter.

3. As a minimum, all system return line filters should be replaced annually.

3.7.4.7 Piping, Tubing, and Hoses

Plumbing systems for hydraulic movable bridge machinery often consist of complex arrangements of high pressure piping, stainless steel tubing, and hoses. Piping runs usually

contain many bends, elbows, fittings, and mountings due to the complex nature of the bridge structure. Vibration from operation of the equipment, vibrations from vehicular traffic, as well as movement of the equipment itself has the tendency to loosen piping fittings and piping supports. This in turn can cause system leaks as well as damage to the plumbing from not being adequately supported.

The following should be regularly performed on hydraulic system plumbing:

1. Visually inspect the condition of all flexible hoses. Any damaged, nicked, or worn hoses should be replaced immediately.

2. Visually inspect all plumbing fittings for signs of leakage. Any loose fittings should be tightened as required. O-ring or other seals should be replaced as required.

3. Inspect piping support systems. Check tightness of hangers and mounting hardware.

4. Visually inspect all pump and control valve pilot lines for leakage. Loose pilot lines should be tightened.

5. Visually inspect inlet and outlet plumbing to main pumps. Loose connections should be tightened as necessary.

3.7.4.8 Hydraulic Fluids

Water has been used as the working fluid in some hydraulic systems, including the original operating system of the Tower Bridge in London. However, most modern movable bridge hydraulic systems utilize either standard mineral oil or synthetic environmentally friendly vegetable oil as their working fluid. The useful life of these fluids is not infinite. Several factors influence the life expectance of hydraulic fluid including system usage, operating temperature, system cleanliness, water intrusion, etc. Synthetic vegetable oils tend to oxidize after several years and require replacement.

Laboratory analysis on system fluid should be performed at least on an annual basis. Samples should be analyzed for viscosity, contamination levels, wear elements, and water content. Annual fluid sample analysis will greatly assist in determining when system fluid requires replacement.

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3.8 MECHANICAL SYSTEMS

3.8.1 Introduction

Mechanical components discussed in this chapter include machinery for bascule, swing, vertical lift, and rolling lift bridges. The machinery includes the span drive machinery and the stabilizing machinery.

The purpose of this chapter is to discuss basic inspection of mechanical components that should be performed as part of a scheduled maintenance program. This manual often references the American Association of State Transportation and Highway Officials (AASHTO) Movable Bridge Inspection, Evaluation, and Maintenance Manual, as well as other text. This manual is to be used as a “field guide”, which shall be used with the other referenced material if a deficiency is found that may not be corrected with normal maintenance practices.

All machinery components are related to and function as a system with the electrical and structural components. This means that a deficiency found at one item would most likely affect other elements. The elements are categorized for ease of reference, but often relate to the other elements in that assembly.

3.8.2 Open Gearing

Open gearing refers to gears that are not contained in a sealed housing. The gears are supported by shafting and bearings that are mounted onto fabricated or cast metal structural supports or framing. During initial installation, they were most likely mounted in the field and aligned by positioning the bearings with respect to the supports with shim material. Most likely the gear mesh will be as installed, but distortion of the supports, deterioration of the fasteners, deterioration of the shims may result in an abnormal alignment and/or wear of the open gears. Wear of open gearing is compounded by the constant exposure to weather and the presence of abrasive, foreign materials that lodge in the gear mesh.

The most common type of open gearing found on movable bridges is spur gearing. Spur gears transmit power and regulate the speeds of parallel shafts. This is commonly found for rack and pinions (see Figure 3.5.2.1-1). In order to transmit power between two shafts set at an angle (not in parallel), bevel gear sets are commonly used. These are often utilized on swing bridges. On some older bridges, open worm gearing was utilized, which transmits power between perpendicular shafting, and often was designed to be self-locking. This is appropriate for use at end-wedges for swing bridges and span locks for bascules.

3.8.2.1 Gear Alignment

As discussed previously, gears may be misaligned due to incorrect installation, or deterioration of the elements that support the gearing, that has resulted in a “shift” in the alignment. If any misalignment is found during inspection, the proper remedy should be established and implemented or the misalignment may result in accelerated wear, and undo stress on the gear teeth.

Maintenance inspection for gear alignment should include:

1. Viewing the patterns in the grease that are left behind during operation. The pattern should show even contact across the full tooth width at the pitch line. If the pattern is heavier on one edge of the tooth, or the pattern is not along the pitch line. This indicates that the shafts are not parallel to one another.

2. If the grease patterns indicate misalignment, the second step is to determine the amount of misalignment. This is achieved by measuring the backlash between the mating gears. Backlash is the space between adjacent non-contacting teeth. This can be measured using feeler gages and should be measured to ± 0.003 inch. These measurements can be used to calculate the required shaft realignment in order to correct the gear misalignment.

3. Bevel gear alignment is more commonly found to be misaligned. The same procedure should be followed as for spur gears. Common misalignment can be heavier contact at the heel portion of the tooth, or at the toe portion of the tooth, which most likely indicated that the shafting is not aligned at a true 90-degree angle with respect to each other.

3.8.2.2 Gear Wear

Detailed examples of types of wear are described and pictured within the American Association of State Transportation and Highway Officials (AASHTO) Movable Bridge Inspection, Evaluation, and Maintenance Manual.

Maintenance inspection for gear wear should include:

1. Inspection of the tooth surface, which should be smooth at the contact area. Scoring or deep gouges are sure evidence of deterioration of the tooth surface.

2. Inspection of the tooth roots for cracks (this is the area of highest bending stress)

3. Inspection of the tooth for “fins” that may form due to plastic flow of the steel.

Refer to the AASHTO Movable Bridge Inspection, Evaluation, and Maintenance Manual for methods to measure wear.

3.8.3 Enclosed Gearing

Gear sets that are mounted in dust-proof, oil-tight, housings are generally referred to as enclosed gearing. On bridges these assemblies are usually speed reducers. In most cases, they are engineered and manufactured by companies that specialize in mechanical power transmission equipment.

Older custom reducer housings were commonly made of cast iron or cast steel. Shafting was usually supported within sleeve type bronze or Babbitt bearing bushings. Newer custom reducer housings are usually fabricated with welded steel. They are most likely equipped with anti-friction roller or ball bearings. Housings of mass-produced standard reducers are still cast from various grades of iron and steel.

Types of gear reducers include:

1. Parallel Shaft Reducers
2. Bevel Reducers
3. Differential Reducers
4. Planetary Reducers
5. Worm Gearings
6. Gear Motors

The most common type of lubrication for enclosed gearing is splash lubrication. The action of the gearing lifts the oil at the bottom of the housing and splashes it over the gear teeth and bearings. Many bearings are provided with grease fittings, where splash lubrication is not practical, e.g., vertically mounted shafts. Manuals specific for each bridge should be referenced for actual lubrication type.

Maintenance inspection of enclosed gearing should include the following items:

1. Visually inspect gearing through inspection ports in speed reducer housings.
2. Visually inspect reducer supports for cracks or deterioration.
3. Inspect each mounting bolt for tightness, damage, and deterioration.
4. Inspect the reducer housing, in particular at the mounting feet or flanges for cracks or damage.
5. Inspect each bolt that connects the top and bottom housing halves for proper tightness.
6. For sleeve-type bearings – check seals and packing for oil leaks. Measure clearances between shaft journal and bushing, if accessible. Watch during operation for any excessive movement of the shafts.
7. For anti-friction type bearings – check seals for oil leaks. Watch during operation for any excessive axial movements of the shafts.
8. Determine oil level by external level gage. Fill as required.
9. During operation – listen for any abnormal noises. Feel the bearing housings with the fingers; they should not become hot to the touch. No evidence of housing or fastener movement should be present.

3.8.4 Bearings

3.8.4.1 Sleeve Bearings

A sleeve bearing is a fixed cylinder (also referred to as sleeve or bushing) in which a shaft journal rotates. The sleeve is usually made of bronze (or on older bridges, Babbitt) and is

held to a fixed point within a steel housing. Housings are usually split in order to remove the shaft for repairs. The top half is bolted down to the base, and the base bolted to the steel structure. Often the sleeve bearing is provided with a flange that acts as a thrust surface to hold the shaft horizontally positioned.

A sleeve bearing requires lubrication between the sliding surfaces of the journal and bushing. Normally one or more grease fittings are located at the top of the housing. A path is drilled through the housing, and bushing, and meets with grooves machined into the surface of the bushing. The groove is usually in a spiral pattern, which helps to lubricate the entire surface of the journal.

Maintenance inspection of sleeve bearings should include the following items:

1. Inspect bearing supports, mounting bolts, and grout pads for cracks, damage, and deterioration.

2. Inspect mounting bolts and cap bolts for tightness.

3. Inspect bearing housing for cracks and damage.

4. Inspect bushing and flange for cracks and damage.

5. Confirm that during lubrication, old grease exits from the space between the journal and bushing.

6. During operation, note any movement of the bearing or support. This will indicate damage to the system that may need repair.

7. During operation, note any movement of the shaft within the bushing. Any excessive radial movement indicates wear to the bushing. If excessive movement is found, other parts of the system may be adversely affected. They should be closely inspected and evaluated.

8. Clearance between the shaft and the bushing should be measured, using feeler gages, and measurement recorded.

9. Feel the exterior housing of the bearing after operation. The bearing should remain cool to the touch. Any heat generation may indicate improper lubrication, or damage to the bearing.

3.8.4.2 Anti-Friction Bearings

Anti-friction bearings include roller bearings and ball bearings. Typically heavy-duty spherical roller bearings are used to transmit power. Lighter duty ball bearings are commonly used for instrumentation that drives electrical control feedback devices. In general, the clearances of the bearing are set during installation, and the unit is sealed for operation. Little wear occurs at these bearings, so wear measurements are not required. Indications of potential problems or failure of an anti-friction bearing are overheating, unusual noises and shaft or bearing vibration. Some contributing factors include too much or inadequate lubrication, dirt, rust or foreign materials in the bearing, a faulty ball or roller, seal failure, and loss of clearance or preloading.

Maintenance inspection of the anti-friction bearings should include the following items:

1. Examine bearing supports, mounting bolts, and grout pads for cracks, damage, and deterioration.

2. Inspect mounting bolts and cap bolts for tightness.

3. Inspect bearing housing for cracks and damage.

4. Lubricate bearing and confirm grease enters the bearing and purges through the exit port.

5. During operation, note any movement of the bearing or support. This will indicate damage to the system that may need repair.

6. During operation, listen to each bearing for any unusual noises. Anti-friction bearings should operate smoothly and quietly.

7. Inspect the seals for damage and proper sealing. Excessive lubricant is a warning flag.

8. Feel the exterior housing of the bearing after operation. The bearing should remain cool to the touch. Any heat generation may indicate improper lubrication, or damage to the bearing.

9. If possible, open housing and visually inspect accessible portions of rollers and races. Check for internal contamination.

3.8.4.3 Trunnion Bearings

Trunnion bearings refer to the rotation point for bascule bridges and the main bearings for the sheaves on the vertical lift bridges. The trunnion bearings can be either sleeve type or anti-friction type bearings. Maintenance inspection of the trunnion bearings is the same as for bearings in general.

In-depth inspection of the trunnion bearings for vertical lift bridges is recommended due to a history of fatigue crack formation. This should be scheduled based on the design and use of the bridge. This type of inspection would involve disassembly of the bearing caps and visual, dye-penetration, magnetic particle, and ultrasonic testing of the shaft fillet.

3.8.5 Shafts & Couplings

Shafts transmit torque from one rotating part to another. Shafts ends are usually connected by couplings, which are secured to the shaft by an interference fit, and key.

3.8.5.1 Shafts

Maintenance inspection of shafts should include the following items:

1. Inspect keyways and shoulders for cracks. Suspect cracks should be further inspected by non-destructive testing methods.

2. During operation, inspect shafts for excessive radial movements and vibration.

3.8.5.2 Couplings

Couplings are manufactured in large variety, but can be categorized into three general groups: rigid, flexible, and adjustable. Rigid couplings are commonly found on older bridges, and basically clamp shaft ends together. Flexible type couplings are designed with internal elements that allow for misalignment during operation due to distortion of the bridge structure. The intent is to avoid bending the shafts. They also simplify shaft installation by allowing slight misalignment at the joints. Adjustable couplings are in span drives of connected tower lift bridges and other drives that require angular adjustment with time because of wire rope stretch, etc. They are also used to index electrical control devices. The AASHTO Movable Bridge Inspection, Evaluation, and Maintenance Manual gives detailed descriptions of the different types of couplings.

Maintenance inspection of couplings should include the following items:

1. Inspect keyways for cracks.

2. Visually inspect for corrosive deterioration and cracks.

3. Check flange bolts for tightness.

4. Flexible couplings may fail without proper lubrication. Check for adequate lubrication.

5. For flexible couplings, inspect all seals and gaskets for leakage of lubricant. It is important to lubricate the couplings in accordance with the manufacturer's recommendations, so as not to damage the seals from over-lubricating.

6. During operation, make sure couplings rotate smoothly and free of noise. Noise would indicate inadequate lubrication or misalignment of the shafts greater than the tolerances of the coupling.

7. Disassemble housings or covers. Inspect condition of internal flex grids and coupling hub teeth.

3.8.6 Buffer Cylinders

The purpose of a buffer cylinder is to provide a mechanical means to control the deceleration of the span at the fully closed and/or the fully open positions. Usually the buffers are large pneumatic cylinders, although hydraulic energy absorbers are used. The major components of the air buffer cylinder include: the cylinder housing, the piston, the piston rod, inlet and outlet piping, and the strike plate. Gravity (or spring pressure for horizontally mounted buffers) extends the piston rod, filling the void in the cylinder with air. Upon operation, the piston end hits the strike plate, and the air is forced out of the valving of the cylinder thus dampening the span movement.

Maintenance inspection of the buffer cylinders should include the following items:

1. Inspect for damage or deterioration to the external housing and components.

2. Inspect for damage or deterioration of the mounting bolts.

3. During operation, observe the piston and note if it does not fully extend from the cylinder when the span is moved.

4. During seating, listen for the sound of air being pushed past the outlet valving, it should be evident. Listen for air leakage through the inlet piping. Air leakage indicates the check valve is faulty.

5. During seating the buffers should provide controlled seating of the span. Observe the seating of the span and record any abnormalities.

If the buffer does not function properly with the guide bushing adequately lubricated, the buffer assembly should be serviced by a mechanical contractor.

3.8.7 Live Load Bearings

Live load shoes are found on trunnion bascule, rolling lift bascule, and vertical lift bridges. The function of the live load shoe assemblies is to transfer vehicular live load from the span to the pier/approach span when the span is in the closed position. The locations of the live load shoes are typically at all four corners of a lift span, and can be found in front of or behind the center of rotation for bascule bridges. The assembly consists of a shoe with a rounded surface (mounted to the span) and a flat strike plate mounted to the pier (or other fixed structure). Both are secured with mounting bolts and are provided with shims for adjustment of the span position.

Maintenance inspection of the live load reactions (bearings) should include the following items:

1. Inspect mounting bolts to determine if they are tight.

2. Inspect the bolts and shims for deterioration.

3. Inspect contact surfaces of shoe and strike plate for deformations and wear.

4. Inspect that firm contact exists between shoe and strike plate. If a gap exists, it should be measured with feeler gages and re-shimmed to make it tight.

3.8.8 Span Locks

Span locks are used to transfer vehicular live load between the leaves of a double-leaf bascule bridge, and to ensure the span is in the closed position for single-leaf bascule and vertical lift bridges. The assembly typically consists of a forged steel lock bar that is driven into a receiving socket. On the drive end are additional sockets that guide and support the lock bar.

The sockets are typically welded or cast housings equipped with adjustable wear shoes. Adjustment is made by adjusting the shim material. The lock bar is operated within the

sockets by a crank mechanism, which is driven by gearing and a motor. Linear lock bar operators are also common.

Maintenance inspection of the span locks should include the following items:

1. Inspect sockets for adequate lubrication.
2. Inspect sockets and shoes for cracks and damage.
3. Inspect mounting bolts for tightness and deterioration.
4. Inspect locks under traffic. There should be little, if any, relative span movement. If movement is observed, gaps between bar and sockets should be measured with feeler gages and recorded.
5. Inspect operation of the lock bar mechanism. Individual elements should be inspected as described in individual component sections.

3.8.9 Rolling Lift Bascules

3.8.9.1 Treads and Tracks

On rolling lift bridges, the weight of the leaf during operation is transferred from the curved segmental girders to the flat tracks through the treads. The treads are constructed with sockets that engage mating pintles or lugs on the tracks as the leaf rolls. These serve to position the span during operation and provide resistance against wind forces.

Maintenance inspection of the tread plates should include the following items:

1. Inspect the mounting bolts that attach the treads to the segmental girder.
2. Inspect the contact surfaces for even wear across the width of the tread and track.
3. Inspect the treads and tracks for cracks.
4. Inspect the treads and tracks for surface deformation.
5. Inspect the lugs or pintles and sockets for wear and interference.
6. Inspect the segmental girders and track girders for cracks. Pay particular attention to the angles between web and flange.

3.8.9.2 Centering Devices

To ensure roadway alignment of a rolling lift bridge, a centering device is usually located at the toe of each span. The two-part device consists of a tapered male piece and female pieces that gradually align the span horizontally during seating.

Maintenance inspection of the centering device should include the following items:

1. Inspect for adequate lubrication.

2. Inspect for uneven wear that may indicate span misalignment.

3. Inspect fasteners for tightness and deterioration.

4. Inspect housing for cracks and damage.

5. Observe device during operation. Under normal conditions, the device should not encounter much sideward force.

3.8.10 Vertical Lift Bridges

3.8.10.1 Wire Ropes and Terminations

Wire ropes are used to connect the vertical lift span to the counterweights located in the towers. Wire ropes are also used to operate span drive vertical lift bridges. In both cases, the type of inspection is the same. Termination points include open spelter sockets affixed to each rope end. Rope clips and wedge sockets are often used for operating rope terminations at the operating drums.

Maintenance inspection for wire ropes should include the following items:

1. Inspect for surface cracks on the outer strands of the rope. Note any cracks or broken wires.

2. Inspect for rust due to inadequate lubrication. “Rust-bound” ropes are particularly dangerous and most likely will cause a failure.

3. Inspect wire ropes for abrasive wear in the form of flat spots on the outer wires of the strands. Flat spots greater than ¼ inch long for ropes under 1-inch diameter and greater than ½ inch long for ropes over 1-inch diameter should be brought to the attention of the engineering staff.

4. Inspect for damage due to abuse. Any large debris that may hit the wire rope will most likely nick the surface, weakening the wire rope. Any damage should be noted and reported.

5. Inspect lubrication coverage. If surface rust starts on the wire rope, the lubrication frequency should be re-evaluated by the engineering staff.

6. Observe the wire ropes during operation. Note any interference that may be rubbing the wire rope.

Maintenance inspection for wire rope terminations should include the following items:

1. Inspect point of contact with wire rope. This is the place most likely to collect moisture and corrode the wire strands.

2. Inspect the socket for cracks and damage.

3. Inspect the socket pins for full engagement and excessive clearance.

4. Inspect cotter pins used to hold the socket pins in place. Note any deterioration.

5. Inspect integrity of zinc in end of each socket.

3.8.10.2 Counterweight Sheaves and Drums

Counterweight sheaves are large diameter, angular-grooved drums, located at the top of the towers, over which the counterweight ropes are draped, supporting the entire weight of the span and counterweight during operation. Operating drums are located on the span, and are grooved in a spiral fashion in order to store several wraps of wire rope. These operating ropes are paid-out and taken-in during operation.

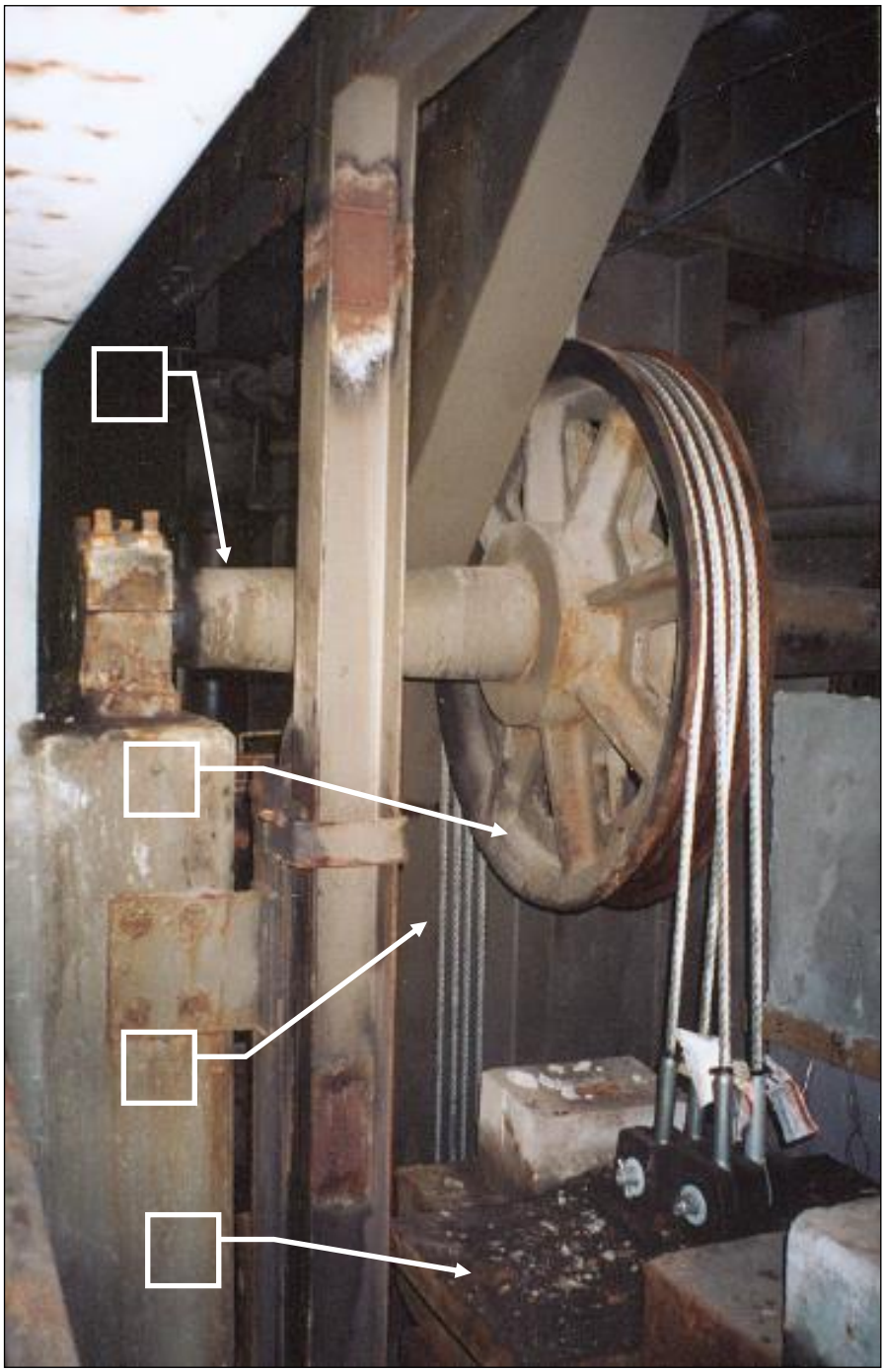
For both types, the grooves are cut to a specific dimension in order to properly space adjacent ropes and support the rope without crushing it. Both drums and sheaves are supported by shafts, and within bearings as discussed in Section 3.8.4.

Maintenance inspection for sheaves and drums should include the following items:

1. Inspect rope grooves for adequate lubrication, signs of rust or corrosion, and the presence of debris.

2. Inspect for indications of rubbing between ropes and between ropes and grooves.

Figure 3.8.10.2-1 depicts the counterweight system at one corner of the St. Paul Avenue Bridge in Milwaukee. This is a balanced pit-drive vertical lift. The significant components are flagged in the figure. The vertical wide-flange steel shape shown is part of the traffic resistance gate located overhead.



a Trunnion
b Counterweight Sheave
c Counterweight Ropes
d Counterweight

Figure 3.8.10.2-1: Counterweight System, St. Paul Avenue Bridge in Milwaukee.

3.8.10.3 Equalizing Systems

A number of pit-drive vertical lift bridges are operational in Milwaukee. Some are balanced pit-drive vertical lift spans as described in Section 3.3.5.1. Milwaukee's lift span bridges of this type are raised and lowered by hydraulic cylinders. The spans are guided vertically at each corner of the span by "lifting posts" (which do not actually lift the spans). Lifting posts typically are rigidly connected to the end floorbeams and fascia girders. Skewing of the lift span during raising and lowering is typically restrained (in both the longitudinal and transverse directions) by wire rope equalizing systems in older structures. More modern structures use position transducers to replace the transverse and longitudinal equalizing cables and sheaves.

The equalizing systems on three Milwaukee bridges (St. Paul Avenue, Michigan Street, and Wells Street), will be described with the aid of reeving diagrams and photographs. The reeving diagrams are schematic and are not drawn to scale. When viewing these diagrams, the reader should recall that these rope systems are not counterbalancing systems (the counterbalance ropes are a separate system) and they are not span drive ropes (the span is lifted by hydraulic cylinders). Because there is no standard designation for these systems, they will be denoted as moving-drum, stationary-drum, and sans-drum. These terms are most descriptive of the Milwaukee systems.

Moving Drum Equalizing System

Figure 3.8.10.3-1 is a schematic of a moving-drum equalizing system. It is denoted as moving-drum because the drum is attached to the end floorbeam or fascia girder of the lift span (the movable part of the bridge). The system nearly equalizes the vertical motion of the two lifting posts (subject to changes in rope length due to elastic deformations and temperature). This is necessary because the center of gravity of the span may not coincide with the center of effort of the lift cylinders and because of differential friction between the two sets of lifting post guide rollers due to wind and other forces. Such effects tend to cause the span to rise canted.

In the planar system, as shown in Figure 3.8.10.3-1, there are two wire ropes. One rope is fastened to the pier wall by the adjustable anchorage at "1" and then passes under the sheave "2" which is mounted on the bottom of the lifting post. From there it rises to sheave "3" and is wrapped around the drum at "4" before passing over a sheave at "5" and descending to an anchorage on the fixed structure at "6". Sheaves "2", "3", "5", and drum "4" are mounted on the moving structure. For clarity, the drum is shown as being in two parts, which are connected by a shaft. A single drum, properly grooved, would serve as well. Also, the cables need not be continuous about the drum; they can be discontinuous with terminations at the drum.

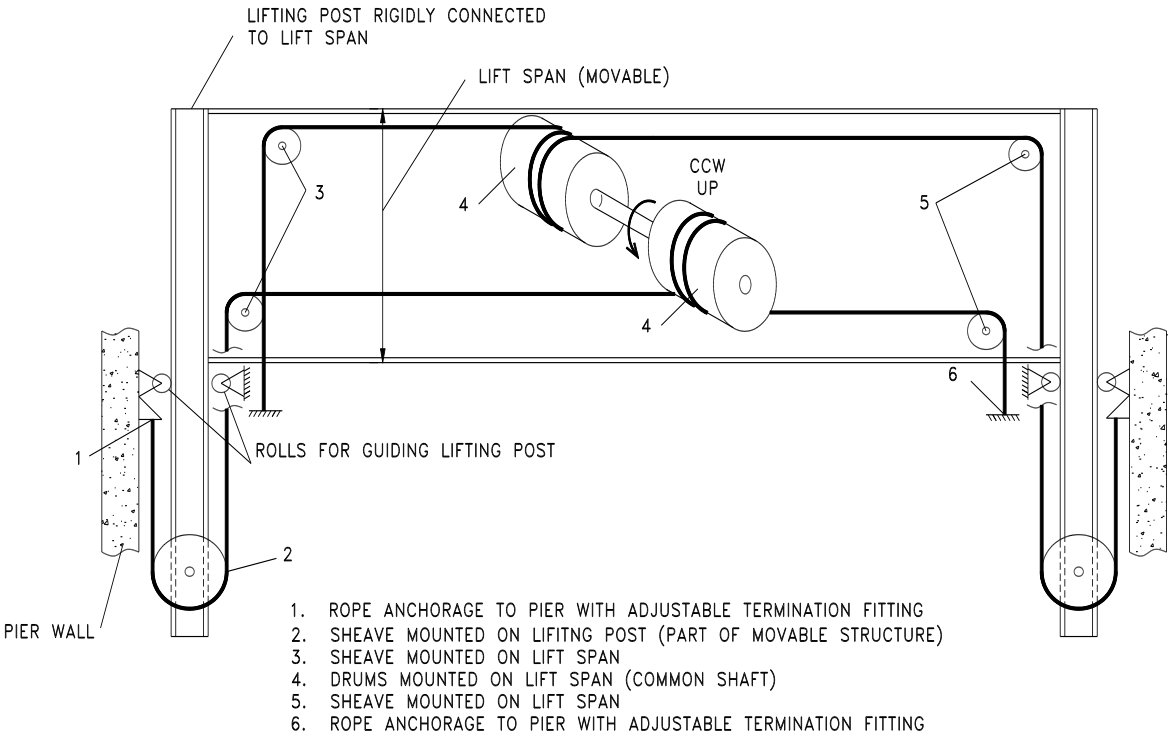
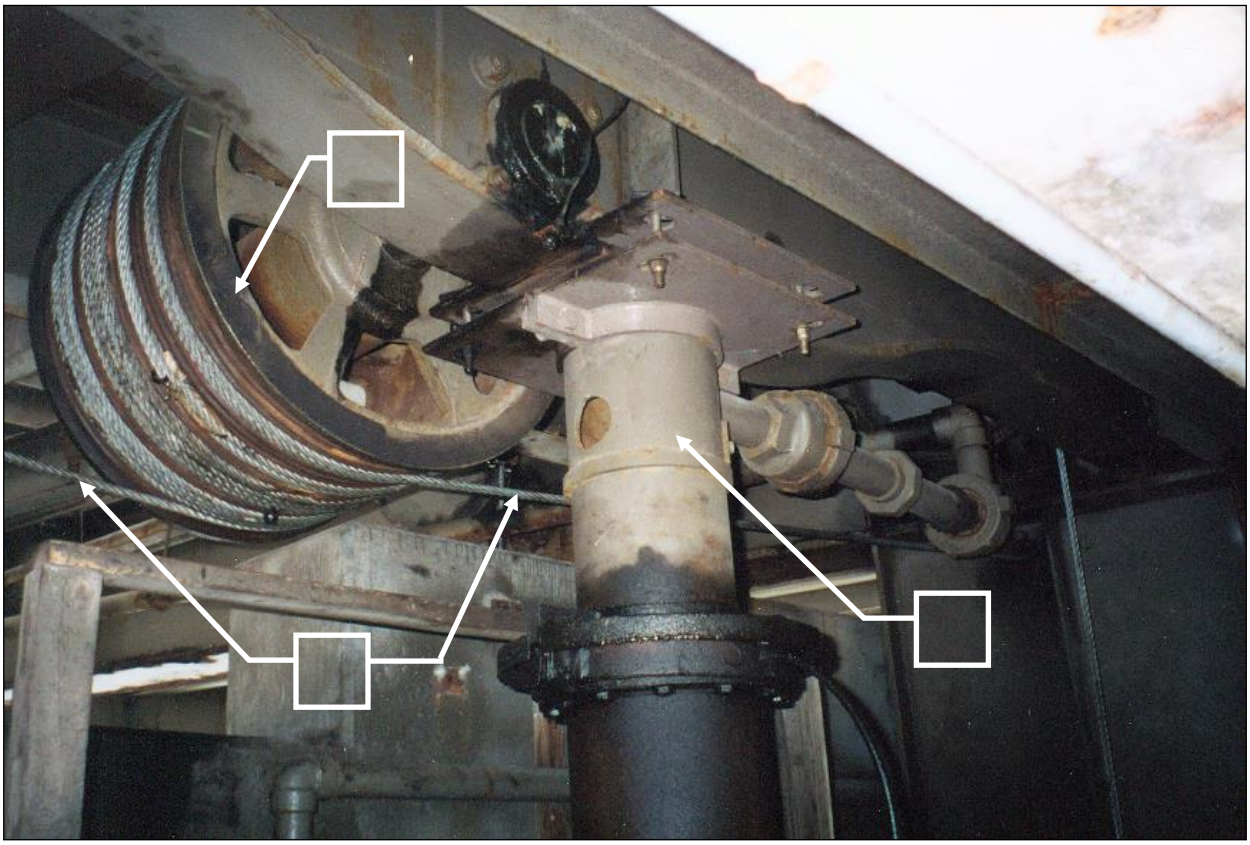


Figure 3.8.10.3-1: Moving-Drum Equalizing System.

St. Paul Avenue Bridge is equipped with four moving-drum equalizers; one at each end of the floorbeam and one parallel to each fascia girder. Figure 3.8.10.3-2 depicts one of the drums mounted to the westerly end floorbeam.



a Drum b Transverse equalizer rope c Lift cylinder

Figure 3.8.10.3-2: Transverse Equalizer Drum, St. Paul Avenue Bridge in Milwaukee.

Stationary-Drum Equalizing System

In the stationary-drum wire rope equalizing system the drum and the sheaves over which the horizontal segments of rope stretch are mounted on the pier (the stationary part of the bridge). Figure 3.8.10.3-3 depicts the system used on the Michigan Street Bridge to equalize vertical motion of the lifting posts and minimize transverse skew. There are two transverse systems, one on each pier. At this bridge, skew control in the longitudinal direction (span-wise) is accomplished using sans-drum wire rope equalizing systems, which are described subsequently.

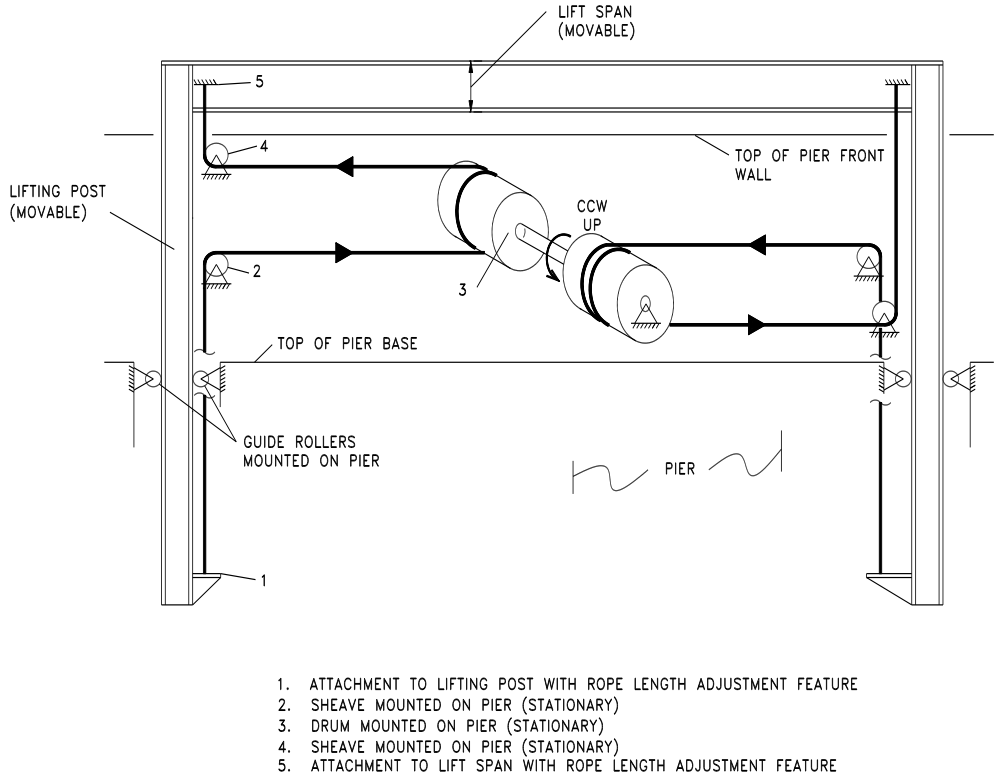


Figure 3.8.10.3-3: Stationary-Drum Equalizing System.

Sans-Drum Equalizing System

Wire-rope equalizers can be reeved without the use of wrappings around a drum. In fact, the equalizers of earlier pit-drive vertical lift bridges did not have drums. A schematic diagram of a drum-less, or sans-drum, equalizer is shown in Figure 3.8.10.3-4. All sheaves are mounted on the moving structure. The arrangement of the parts on the lower end of a lifting post of the Michigan Street Bridge is depicted in Figure 3.8.10.3-5. As mentioned previously, the sans-drum system is used for longitudinal (span-wise) equalization on the Michigan Street Bridge. Stationary drum systems are in place for transverse equalization on this bridge.

The Wells Street Bridge has two sets of sans-drum equalizer systems that are oriented diagonally in plan; each set connects the lifting posts in the opposite corners of the left span. The effectiveness of this arrangement, especially in the transverse direction, is dependent on the torsional rigidity of the lift span (framing plus decking).

In general, the effectiveness of wire rope equalizers is dependent on their axial stiffness; i.e., the elongation of the rope when it is stressed by the skewing action of the bridge. Hence, it is important that the ropes have adequate pretension. Turnbuckles or threaded fittings are provided at the anchorages for adjustment. The rope anchorage with take-up adjustment fittings in place at the Wells Street Bridge is depicted in Figure 3.8.10.4-1.

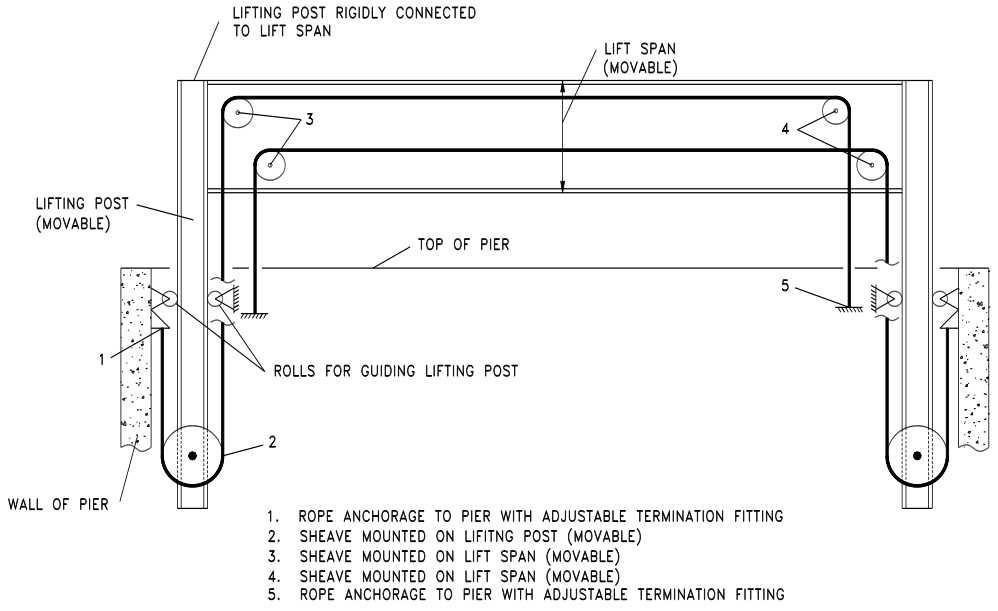
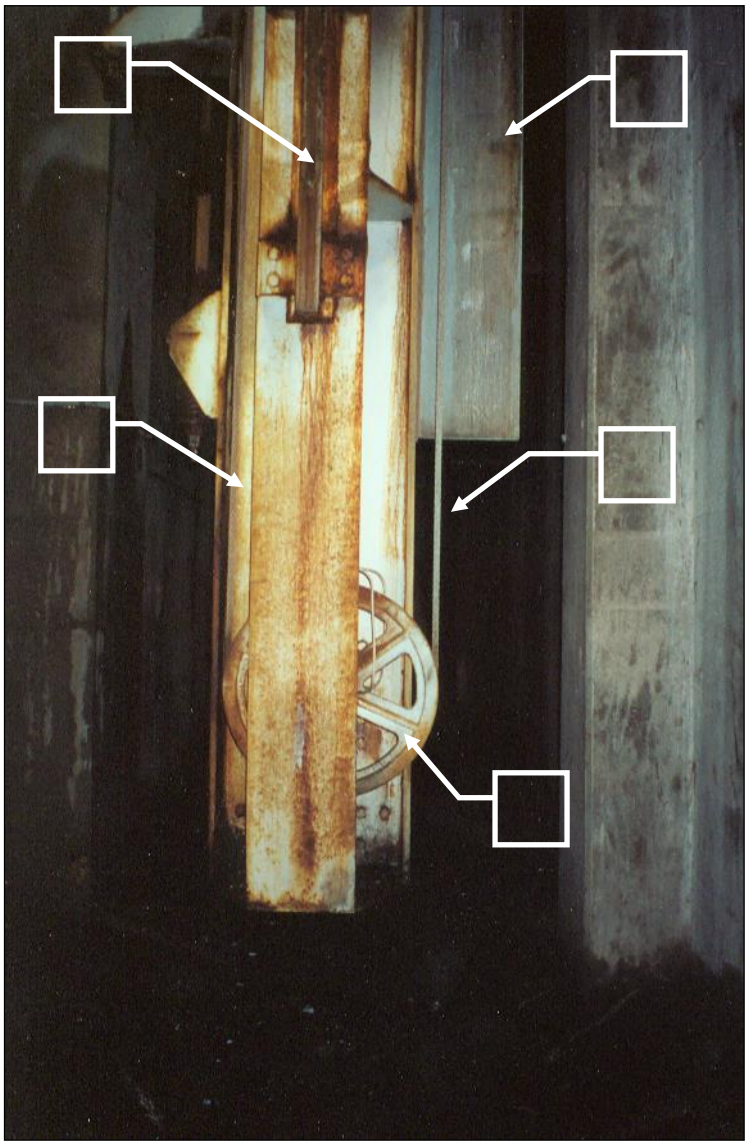


Figure 3.8.10.3-4: Sans-Drum Equalizing System.



a Plate clamp rail
b Lifting post
c Guide rail
d Equalizer rope for longitudinal system
e Lower sheave

Figure 3.8.10.3-5: Bottom of Lifting Post, Michigan Street Bridge in Milwaukee

Position Transducer Equalizing System

Older structures use mechanical methods to ensure the lift span of a moveable structure opens and closes uniformly. As described in the previous sections this is performed through the use of transverse and longitudinal equalizing cables and sheaves. Newer structures are now relying more on technology to replace this system. The Wisconsin Avenue structure was rehabilitated in 2012 and its mechanical equalizing system was replaced with a position transducer equalizing system. Transducers are placed on each hydraulic cylinder within the structure and networked together by wires. During the opening and closing of the lift span, the transducers “communicate” with each other to determine where the bridge is positioned.

The transducers then self-correct by altering the rate of cylinder movement. The benefit of this system is the removal of hardware from the bridge and the need for continual maintenance on the moveable parts.

3.8.10.4 Span and Counterweight Guides

Span guides are used to restrict lateral and longitudinal movement of the span during operation. Counterweight guides are used to restrict movement of the counterweight as they travel within the towers. Typically roller guides are used as span guides, and sliding guides are used as counterweight guides. Guide rails are located along the tower legs, and are for contact with the guides against any side wind loads.

Maintenance inspection for span and counterweight guides should include the following items:

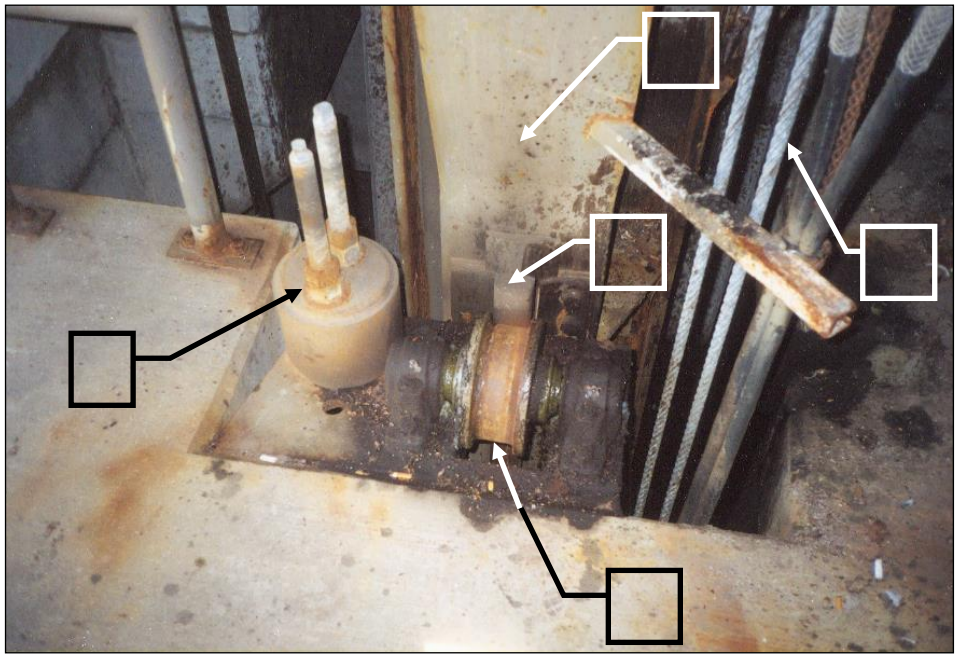
1. Inspect guide clearance the entire length of lift. This should reveal any damage to the rail.

2. Rotate rollers to ensure proper operation.

3. For sliding guides, be sure rails are adequately lubricated the entire length.

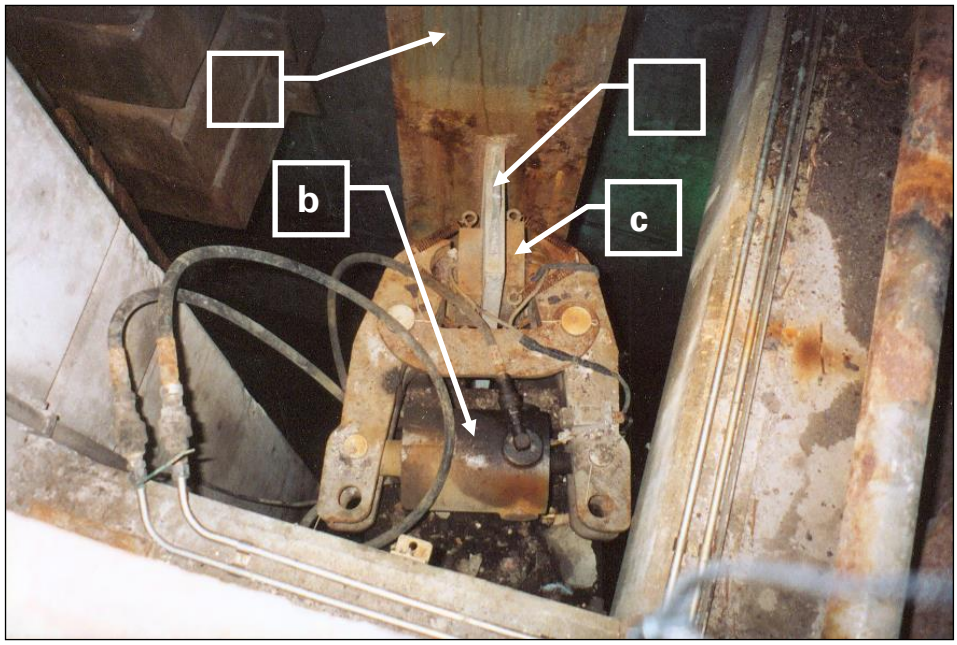
Figure 3.8.10.4-1 depicts the guide rail and guide rollers for the Wells Street Bridge. For this bridge the guide rail should not be lubricated because the sides of the rail head serve as the “brake drum” for the rail clamp brake.

A typical rail clamp brake is shown in Figure 3.8.10.4-2 for the St. Paul Avenue Bridge in Milwaukee. Here the “brake drum”, a long rectangular bar, was not part of the original construction. It was welded to the lifting post sometime thereafter and then the rail clamp was installed.



a Guide roller
b Guide rail
c Lifting post
d Equalizer ropes
e Rope take-up

Figure 3.8.10.4-1: Guide rail detail, Wells Street Bridge in Milwaukee.



a Lifting post
b Hydraulic cylinder
c Rail clamp shoes
d Plate clamp rail

Figure 3.8.10.4-2: Rail clamp, St. Paul Avenue Bridge in Milwaukee.

3.8.10.5 Auxiliary Counterweight and Balance Chains

Auxiliary Counterweights

During operation, auxiliary counterweights are used to offset the shifting weight of main counterweight ropes as they travel from one side of the sheave to the opposite side. The systems consist of counterweights, with guides and rails, sheaves, wire ropes, and termination points. All sections for inspection of individual elements apply to the auxiliary counterweight systems.

Balance Chains

Balance Chains are an alternative method to offset the differential of the main counterweight rope weight during operation. The system consists of large chains that terminate at the towers at one end, and the bottom of the counterweight at the other. As the span is lifted, the chain weight is transferred from the counterweight to the tower.

Maintenance inspection of the balance chains should include the following items:

1. Observe during operation. Note any frozen links that kink the chain during operation.

2. Inspect cotter pins normally used to secure chain pins. These commonly wear during operation. Excessive wear should be noted and reported.

3. Inspect termination points for any loose or deteriorated fasteners.

3.8.11 Swing Bridges

3.8.11.1 Center Pivot Bearings

For center pivot swing bridges, the entire weight of the span rotates about and is supported by the center pivot bearing. The center pivot bearing may be a bronze disc type or an anti-friction type spherical thrust bearing.

Maintenance inspection for both types of bearings should include the following items:

1. Inspect exterior of housing for cracks and damage.

2. Inspect mounting bolts for tightness and deterioration.

3. Note any loud or unusual noises during operation, bearings should operate quietly and smoothly.

4. Inspect oil level at site gage.

3.8.11.2 Balance Wheels and Track

For center pivot type swing bridges, balance wheels are utilized to stabilize the span from any side wind force on the span. The wheels are set to a small clearance (approximately 0.1 inch) with tapered track, and only make light contact due to imbalance and/or wind loads.

Maintenance inspection of balance wheels should include the following items:

1. Check for clearance between wheel and track with wedges driven.

2. Rotate each wheel to check for freedom.

3. Lubricate bearing and check purge points.

4. During operation, note which wheels make contact. Only two adjacent wheels should make contact at one time. Any additional contact indicates an uneven track, improper installation of balance wheel, or worn balance wheel bearing, and/or worn pivot bearing.

5. Inspect track and fasteners for deterioration and cracks.

3.8.11.3 Rim Bearings

A rim bearing swing span has a large number of large tapered rollers that rotate between an upper and lower track. This system supports the entire weight of the span during operation.

The tapered rollers are held in alignment with the center of pivot, by radial tension rods that connect to a central hub. This hub rotates about a vertical pin mounted to the pier.

Maintenance inspection for rim bearings should include the following items:

1. Check radial tension rods for relative tension. Note any excessively loose or tight rods.

2. Observe rollers during operation. All rollers should rotate with no sliding. No abnormal noises should be heard.

3.8.11.4 End Lifts and Center Wedges

End Lifts

The ends of a swing bridge deflect during operation, and must be raised to meet the approach roadway level. This acts to stabilize the span under vehicular traffic. Several types of end lift machinery are used including: end wedges, end toggles, and eccentric cams. Most systems utilize crank mechanisms to lock the devices in place. The mechanisms are usually driven by a system of gears and a motor.

Maintenance inspection for the end lifts should include the following items:

1. Inspect contact of shoe with base (mounted on rest pier).

2. Inspect housings and components for cracks and damage.

3. Inspect mechanisms for damage, in particular in the form of twists.

4. Inspect drive machinery as described under sections for individual elements.

5. Observe operation of system. Note any hesitations, or interference.

Center Wedges

The function of center wedges is to relieve the center pivot bearing for live load. Wedges are driven in place between a base casting mounted on the pier and an upper guide casting mounted to the underside of the span. The operation of the wedge is similar to that of the end lift devices.

Maintenance inspection for the center wedges should include the following items:

1. Inspect contact of wedge with base (mounted on rest pier).

2. Inspect contact surface with wedge withdrawn. Note any scoring or gouging of the surface.

3. Inspect housings and components for cracks and damage.

4. Inspect mechanisms for damage, in particular in the form of twists.

5. Inspect drive machinery as described under sections for individual elements.

6. Observe operation of system. Note any hesitations, or interference.

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3.9 ELEMENT LEVEL INSPECTION

3.9.1 Introduction

Moveable bridges are complex structures and contain components that, for a significant portion of time, do not see structural loading. Examples of these components would be lifting legs, hydraulic cylinders, equalizer cables, etc. However, because these components do experience the structural dead load of the bridge, they need to be inspected and assessed thoroughly to ensure the proper operation and safety of the structure.

While most moveable bridges are individually unique, there are several shared structural components that can be identified and assessed under the NBE, BME and ADE's discussed in Part 2 of this manual.

This chapter will aid the inspector in identifying common moveable bridge components and assigning them under the most appropriate nationally or locally recognized element.

3.9.2 Inspection Safety - Lock Out / Tag Out Procedure

Prior to the inspection of a moveable bridge inspection, it is imperative that the Bridge Operator is contacted and acknowledges that inspectors will be assessing the condition of all the structural and mechanical components. If the bridge controls for the structure on not located on site, it is recommended that the control box for that structure undergo the lock out/ tag out procedure to ensure that a third party unaware of inspectors within the structure do not accidently operate the machinery creating a potentially life threatening situation.

More information on this program can be viewed at:

https://www.osha.gov/dts/osta/lototraining/

The operation of the moveable components of the bridge during the inspection should only occur after confirming with the inspectors to do so. Similarly, the inspectors may require the bridge to be operated in order to see and hear any potential defects during the opening/closing procedure. Clear communication between the Bridge Operator and Inspectors is required.

3.9.3 Element Level Inspection

Although moveable bridges possess mechanical and electrical components as well as unique structural components, the inspector should view the bridge as a fixed structure. This visualization should be done in both the open and closed settings of the bridge. Components may become loaded in one setting versus the other. It is these components that the bridge inspection should aware of and use his/her engineering judgement to assign the most appropriate element if possible.

WisDOT is aware of the uniqueness each moveable bridge possesses. However, there is a limited inventory of moveable structures within the State of Wisconsin. Consequently it is problematic to the development of agency defined elements (ADE's). Due to the rarity of moveable bridge components within the management system, it is not practical to create individual elements within the data management system. Thus, Wisconsin has chosen to

assess the overall condition of the entire systems unique to Moveable bridges (Electrical, Mechanical, Hydraulic, and Structural)

3.9.3.1 Movable Bridge Member Assessments

WisDOT has two moveable bridge components located under the assessments portion of the inspection report to cover the most typical components of moveable bridges. Through HSI these assessments are found through a drop down menu. Assessments are similar to elements in the method they are assessed. Each assessment is quantified and the total quantity or portion thereof, depending on the unit of measurement for each assessment, is assigned a condition rating of Good, Fair, Poor or Severe.

The assessments allow the bridge owner to track the condition of the bridge components with the same practicality as an element. The following table describes the two assessments currently located within the assessments drop down box in HSI. Refer to Part 2 of this manual for more discussion on assessments and inspection methods.

Assessment	Unit	Description
Moveable Bridge Counterweight	EA	This assessment includes all forms of counterweights. The assessment is assessed in the same manner as an element.
Moveable Bridge Cables	EA	This assessment includes all counterweight and equalizer cables within the moveable bridge system. Elements 147 Steel Cable – Primary, 148 Steel Cable – Secondary and 149 Other Cable – Secondary should not be used to assess moveable bridge cables. These elements are specifically called out in the AASHTO manual for use in suspension, cable stay or suspender cables.

Table 3.9.3- 1: Movable Bridge Components Located in Miscellaneous Assessments

3.9.3.2 Moveable Bridge Elements

It is recommended the inspector assess the other unique components of the bridge under common NBE's, when applicable. For instance, purlins, which are typically found supporting steel grid decking, run perpendicular to stringers. These members are not girders, floorbeams, nor stringers. Purlins could accurately be assessed under the deck element they support. However, purlins are typically repaired or replaced at a different rate than the steel decking, therefore purlins would be most suitably assessed under Element 8170 Other Structural Members.

It is acceptable for Other Structural Members to be used more than once on an inspection report for more than one type of member. For management purposes, it may be in the inspector's best interest to document the number of members under each type in each condition state within the structural notes for the element. This will allow for more efficient

updating and inspection during future inspections. Refer to Figure 3.9.3-1 for an example of documenting several different members under Other Structural Members.

Check	Elem	Sub	Description	Unit	Total Qty.	Quantity in Condition State			
						1	2	3	4
			Other Primary Structural Members	LF	2960	2312	72	576	
X	8170		Purlins Total = 60@48'=2880 LF: 12 purlins, 6 West Abutment/ 6 East Abutment, exhibited 25% section loss over the bottom flange surface area. Remaining in good condition with no surface corrosion noted CS1.= 12x48'=576 LF CS3 Floor Beam Cantilever Total = 20@4'=80 LF: 18 floor beam cantilevers , 4 @ NW corner, 5 @ SW sidewalk, 4 @ NE sidewalk, 5 @ SE sidewalk, exhibited moderate corrosion along bottom flange and lower web with no loss of section -18x4'=72 LF CS2. Remaining in good condition with no surface corrosion noted CS1.						

Figure 3.9.3- 1: Sample Breakdown of Movable Bridge Components in the Element Notes under a Common Element.

It is important to note that whenever a moveable component is used under a more generic element that the inspector specifically quantify the component within the structural notes under the element on the inspection report. This is done to clearly separate the moveable component from the fixed portions. This allows for easier editing of notes in future inspections and allows the program manager look at moveable components as necessary.

Whenever possible, the inspector should attempt to evaluate moveable components under appropriate elements. This should be done to more accurately reflect the actual condition of all structural components, moveable or fixed, and to prioritize funding for repairs or maintenance.

There are numerous moveable bridge systems and components that will not be able to be categorized under a common element. The electrical motors, hydraulic lines, counterweight and cable sheaves are examples of components that need to be evaluated and comment upon in the Movable Bridge Inspection Report forms provided separately for these unique systems for moveable bridges.

The inspection report is a legal document and should not be edited inaccurately or embellished upon so as to unfairly allocate potential funding.

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4.1 GENERAL

4.1.1 Introduction

Ancillary structures are those assets within the highway right-of-way that are not specifically denoted as bridges defined by the National Bridge Inventory (NBI), but have a structural design. Ancillary structural assets include small bridges (C), retaining walls (R), noise walls (N), high mast lights (L), sign and signal structures (S, G), and miscellaneous structures (M).

All types of structures deteriorate over time. These assets have exhibited similar patterns of deterioration as the structures previously discussed in Parts 2 and 3.

The methods to qualify as a Team Leader for Sign/Signal/HML inspections are as follows:

1. NHI Ancillary Structures Inspection Training
2. NHI Nonredundant Steel Tension Member (NSTM) Inspection training
3. One of the following:
    a. WI P.E. Registration
    b. 5 years Structure Inspection Experience
    c. Certified Welding Inspector (CWI)

4.1.2 Inspection Intervals

All ancillary structures covered in Part 4 shall be inspected at regular intervals to ensure their intended function in a proper and safe manner. Figure 4.1.2-1 provides the maximum inspection interval for each specific type of structure.

There are several scenarios where the maximum interval for inspection no longer applies and a more frequent inspection is warranted. Note that these are statewide requirements, but Regional Ancillary Program Managers have the authority to set more restrictive policies for their inventory as they see necessary. These statewide requirements shall be, but are not limited to, the following:

- For small bridges (C)
    - If the structure is load posted at 20 tons or up to but not including 40 tons and above, or the NBI value for Superstructure, Substructure, or Culvert is equal to 4, then the routine inspection interval shall be two (2) calendar years.
    - If the structure is load posted at less than 20 tons, or Superstructure, Substructure, or Culvert are 3 or less, the routine inspection interval shall be every calendar year.
- For sign and signal structures (S, G), and high mast lighting structures (L)
    - If the overall condition rating is severe, then the inspection interval shall be every calendar year.
    - If the overall condition rating is poor, then the inspection interval shall be every two (2) years.

- Steel full span truss structures are only eligible for the six (6) year interval if they are less than 30 years old, and the overall condition rating is good. Otherwise, the interval shall be four (4) years unless conditions of the previous two bullet points exist.

- For noise and retaining walls (N, R), and miscellaneous structures (M)
  - If the overall condition rating is severe then the inspection interval shall be every calendar year, if not more frequent at the discretion of the Regional Ancillary Inspection Program Manager. If the overall condition rating is poor, then it is up to the discretion of the Regional Ancillary Inspection Program Managers responsibility to review and reduce the inspection interval for the structure.

ANCILLARY STRUCTURES			
Structure Type	**Inspection Type**	**Max. Inspection Interval (Calendar Years)***	**Comments**
Sign and Signal Structures (S, G)	Routine	6 Years (Steel Truss Full Spans) 4 Years (Steel Cantilever)	Cantilever structures will have anchor rods ultrasonically tested during each inspection.
		4 Years (Aluminum Full Spans) 2 Years (Aluminum Cantilever)	
	NDT	As needed	Should be performed anytime the inspector notes a defect that needs to be further explored.
High Mast Lighting Structures (L)	Routine	4 Years (Steel)	All anchor rods will be ultrasonically tested during each inspection.
	NDT	As needed	Should be performed anytime the inspector notes a defect that needs to be further explored.
Retaining Walls (R)	Routine	6 Years	
Noise Barriers (N)	Routine	6 Years	
	NDT	As needed	Valid only for noise barriers with anchor bolts, such as a barrier mounted to the side of a bridge.
Miscellaneous Structures (M)	Routine	8 Years	Structure types vary widely; interval to be set by Regional Ancillary Program Manager
Small Bridges (C)	Routine	4 Years	
	UW-Dive	6 Years	If the structure is inaccessible for the Routine inspections due to typical water levels, this interval shall be reduced to every 4 years to match the Routine Inspection.

* Inspections will be scheduled based on the month/year, but performance will be measured on based on completing the inspection in the appropriate calendar year. For this reason, it is highly encouraged that no routine inspections be scheduled for the month of December, unless they are emergencies or initial inspections of a structure recently installed. This is to prevent weather related delays for the inspection that would cause the program to fall into non-compliance.

Figure 4.1.2-1: Recommended Inspection Intervals for Ancillary Structures.

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4.2 OVERHEAD SIGN/SIGNAL STRUCTURES

4.2.1 Introduction

Sign structures, per Chapter 39.5 of the WisDOT Bridge Manual, are composed of "sign bridges" and "overhead sign supports". Sign bridges are typically designed by the Bureau of Structures or a consultant prior to bid letting. The typical use for these structures is to support large signs, or Dynamic Message Signs (DMS).

Overhead sign supports are smaller structures that are typically designed by the contractor, or their representative. These structures include mast arms, trombone arms, full span monotubes, etc. These structures typically carry signs indicating detour routes, upcoming lane changes, highway directional information and a variety of additional information for the traveling public.

Signal Mast Arms, also referred to as signal monotubes, are quite similar to the mast arms that carry signs in the above paragraph. The primary difference is they carry signal heads and have electrical conduit to power the signals. They are extremely common in urban settings and are responsible for the efficient conveyance of traffic information on a lane by lane basis.

These structures are comprised of either steel or aluminum. Currently, all new structures are fabricated of steel. Aluminum structures are being phased out in Wisconsin. The steel is typically galvanized, although it may be painted. Standard overhead sign structure members are either round structural tubes/pipes, or angles and are connected to the structures via welded and bolted gusset plates. Wide-flange shapes and angles are used in sign connections and other places where a flat surface is required. The new, galvanized steel structures being fabricated are typically comprised of angle members in lieu of tubular members. Overhead sign supports are typically comprised of galvanized structural tubes.

The sheer number of structures that need to be inspected and their typical location in high density traffic settings place importance on having well-planned and well-managed inspection procedures to eliminate errors and inconsistencies. Proper traffic control and access equipment is paramount for the inspection crew's and traffic's safety during inspection. Some of the factors that will affect the sequence of the inspection are the type and size of the structure, the time of day (rush hour), traffic density at a particular location, the availability of special equipment, and requirements for traffic control.

The following sections cover the various types of overhead structures mentioned above, proper inspection methods, documentation requirements, and the basic elements and assessments associated with these structures.

4.2.1.1 Overhead Sign Bridges

There are several types of overhead sign bridges used in the State of Wisconsin: the simple-span (or full-span) sign bridge (including four-chord, tri-chord, two chord and monotube configurations), the cantilever sign bridge, the single pole or "Butterfly" sign bridge, and the structure-mounted sign bridge.

Four-Chord Overhead Sign Bridge

The standard four-chord simple-span sign bridge consists of an all-welded or combination welded-bolted three-dimensional space frame of four chords. Diagonal members run along all four faces of the space frame with intermediate diagonals running between the front and back faces of the frame, alternatively. A camber is typically built into the space frame. The space frame is simply supported at each end by a single post, a two-column support frame, or a reinforced concrete column. Refer to Figure 4.2.1-1 for an overall view of an aluminum sign structure. Refer to Figure 4.2.1-2 for a modified galvanized steel structure.

Figure 4.2.1-1: Typical Four-Chord Aluminum Sign Bridge Structure.

Figure 4.2.1-2: Modified Galvanized Steel Sign Bridge Structure.

Tri-Chord Overhead Sign Bridge

The Wisconsin Department of Transportation (WisDOT) does not have a standard design for the tri-chord simple-span sign bridge. In the past, design engineers have designed many of these structures on a job-specific basis.

The tri-chord simple-span sign bridge is similar in basic concept to the four-chord simple-span sign structure. The notable difference between the four-chord and the tri-chord designs is that instead of having a second parallel truss to form a space frame, the tri-chord design utilizes a single chord behind the front vertical truss, which is connected with diagonals to the top and bottom chords of the truss to form the space frame. In cross section, the tri-chord space frame looks like an equilateral triangle, with the top and bottom chords of the front vertical truss forming the two vertices in the vertical plane and the back chord forming the third vertex. Refer to Figure 4.2.1-3 for a typical tri-chord structure.

Figure 4.2.1-3: Typical Tri-Chord Sign Bridge.

Two-Chord Overhead Sign Support

The two-chord full-span sign support consists of a single vertical parallel-chord plane truss that is supported at each end by a single post. The details of the truss are similar to those of the four-chord design, except typically the truss section will be a Pratt Truss Pattern instead of a Warren Truss pattern. The diagonals on a Pratt Truss do not alternate directions, instead the diagonals angle down toward the center of the span. Refer to Figure 4.2.1-4 for a typical two-chord sign structure.

Two-chord full-span sign supports are generally inspected with a bucket lift in conjunction with a lane closure and traffic control. If the spans cannot be climbed safely; each lane under the structure will need to be closed off incrementally for the structure to be inspected. It is the inspector's decision as to whether the structure can be climbed safely.

Figure 4.2.1-4: Typical Two-Chord Sign Support Structure.

Full Span Monotube Overhead Sign Support

Monotube sign supports consist of a single horizontal chord supported by a single tapered steel post at each end. The monotube structures are typically made with tapered members instead of pipe members. The chord is comprised of two tapered members, with the large diameter ends connected at the middle of the span. If a long span is required, a pipe member is placed between the tapered members. Refer to Figure 4.2.1-5 for an overall view of a monotube structure.

Figure 4.2.1-5: Typical Monotube Sign Support Structure.

Cantilever Sign Bridge

The standard four-chord cantilever sign bridge structure consists of a three-dimensional space frame rigidly supported at one end by a single steel post, or a reinforced concrete column. The cantilever space frame is always fabricated as one unit. The member configuration of the space frame is similar to that of a four-chord sign structure. Refer to Figure 4.2.1-6 for a typical four-chord cantilever structure.

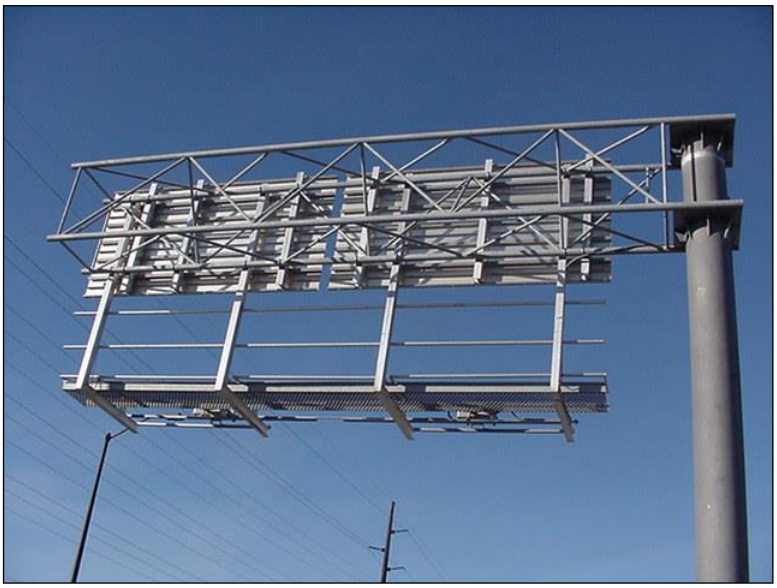

Figure 4.2.1-6: Typical Four-Chord Cantilever Sign Bridge.

Butterfly Sign Bridge

A Single Pole or "Butterfly" sign bridge structure consists of a single vertical steel pipe or reinforced concrete column for the support with horizontal members centered about the column supporting signing. These structures may have two dimensional or three dimensional horizontal members or frames supporting signage on both sides of the column. A Butterfly Sign Bridge with a concrete column will have a short steel pipe section which will be bolted to the top of the column for connection to the sign panels. Alternate configurations may be identified in the field, such as secondary truss members placed between the horizontal tubes, or four-chord box trusses cantilevered off each side of the vertical post. Refer to Figure 4.2.1-7 for a typical single pole butterfly sign bridge.

Figure 4.2.1-7: Typical Single-Pole Sign Structure with a Single Sign Panel.

Structure-Mounted Sign Bridge

This category consists of sign bridges that are mounted directly to the highway infrastructure. The most common style is the bridge-mounted sign structure; however, there are also examples of sign structures that are mounted to retaining walls. Refer to Figure 4.2.1-8 for a structure-mounted sign bridge.

Figure 4.2.1-8: Typical Structure-Mounted Sign Structure, Mounted to a Retaining Wall.

4.2.1.2 Overhead Sign/Signal Supports

Mast arm structures typically fall under the same layout of a vertical post connecting a horizontal arm out over a roadway. The height, length and size of the vertical and horizontal members is dependent on the size of the roadway and/or the number of signal heads and/or size of type II signs being supported overhead. The following section describes the typical layout the inspector may expect to find in the field.

Mast Arm Sign Supports

Mast arm sign supports are comprised of a single support post and a single cantilever arm. Both members are typically tapered at 0.14 inch-per-foot. Mast arm structures are made of steel, usually galvanized, and are typically used to carry small regulatory signs and street name signs. The post base plate is anchored to the foundation typically with four to eight anchor bolts embedded in a concrete foundation. The base plates rest on leveling nuts, creating a space between the base plate and the foundation. Some structures may be mounted to a trapezoidal breakaway transformer base, which is anchored to the foundation.

At the supported end of the mast arm, a vertical plate is shop welded to the end of the tube. A matching vertical plate is welded to the side of the support post. The plates are aligned and field bolted together. The mast arm is connected to the support post typically with four to eight bolts. The bolts secure the arm base plate to the post connection plate. The arm lengths range from 15 feet to over 50 feet, although they typically range from 25 to 45 feet. In some instances, an arm may have an extension at the end of it to get a longer length. These extensions are spliced with an overlap slip joint. Refer to Figure 4.2.1-9 for a typical mast arm sign support structure and Figure 4.2.1-10 for a typical mast arm to post connection.

Figure 4.2.1-9: Typical Mast Arm Sign Support.

Figure 4.2.1-10: Typical Mast Arm Support Post Connection.

Trombone Arm

Trombone arms are comprised of two members clamped to the single cantilever support post. The trombone arm members are typically farther apart at the support post to provide stability. Trombone arms are made of either aluminum of galvanized steel and are typically used to carry traffic signal heads, small regulatory signs, and street name signs. The post base plate is anchored to the foundation with four anchor bolts. The base plates are either attached to a breakaway transformer base or to the foundation directly.

The trombone arm is connected to the support post by a clamp at the top and bottom chord locations. The clamps are bolted to the support post. The clamps can be over-stressed if the bolts are over-tightened. The clamps should be inspected for over/under tightening and possible fatigue cracking. Refer to Figure 4.2.1-11 for a typical trombone arm.

Figure 4.2.1-11: Typical Trombone Arm Sign Support Structure.

Traffic Signal Mast Arms Supports

Traffic signal mast arm supports are comprised of a single support post and a single cantilever arm. Mast arm supports are made of steel, usually galvanized, and are primarily used to carry traffic signals, small regulatory signs and street name signs. WisDOT currently uses four variations of these structures:

- Type 9 (15’ – 30’ arm without street luminaires)
- Type 10 (15’ – 30’ with street luminaires)
- Type 12 (35’ – 55’ arm without street luminaires)
- Type 13 (35’ – 55’ arm with street luminaires).

The post base plate is anchored to the foundation with six anchor bolts embedded in a concrete foundation. The base plates rest on leveling nuts, creating a space between the base plate and the foundation.

The mast arm is connected to the support post with six bolts (Type 9 and 10) or eight bolts (Type 12 and 13). The bolts secure the arm base plate to the post connection plate. In some instances, an arm may have an extension at the end of it to get a longer length. These extensions are spliced with an overlap slip joint. Refer to Figure 4.2.1-12 and Figure 4.2.1-13 for a typical traffic signal mast arm support.

Figure 4.2.1-12: Typical Type 12 Traffic Signal Mast Arm Structure.

Figure 4.2.1-13: Typical Type 13 Traffic Signal Mast Arm Structure.

4.2.1.3 Nomenclature

The inspector shall know the proper nomenclature of the different components of each of the different types on structures. This will be necessary to properly address defects as they are found. Refer to Figure 4.2.1-14 through Figure 4.2.1-17 for the typical nomenclature for each of the main types of structures.

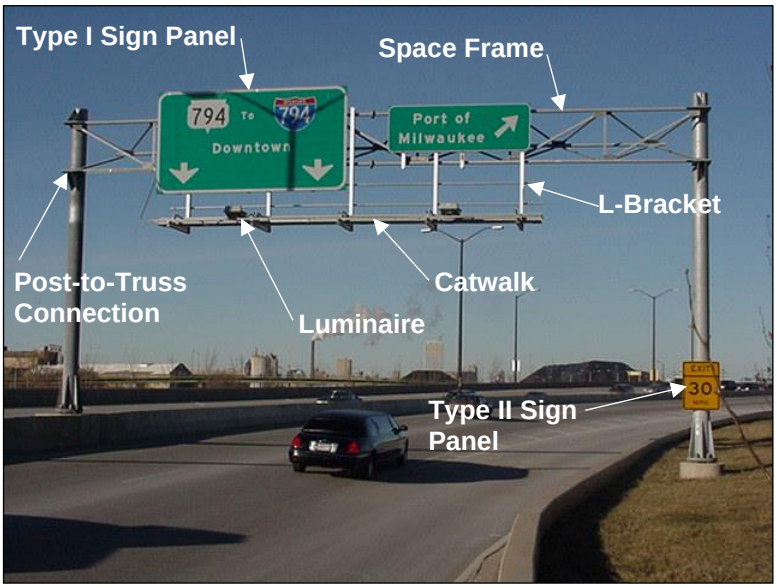

Figure 4.2.1-14: Full Span Sign Structure Nomenclature

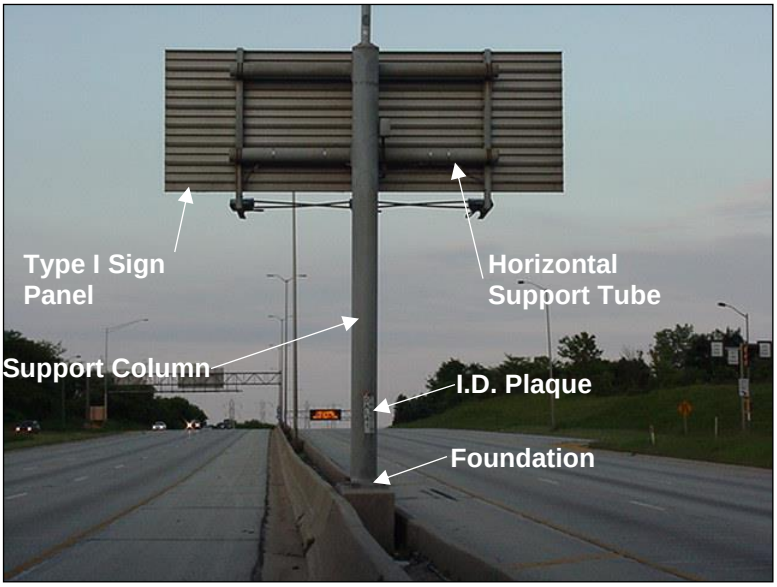

Figure 4.2.1-15: Single Pole (Butterfly) Sign Structure Nomenclature.

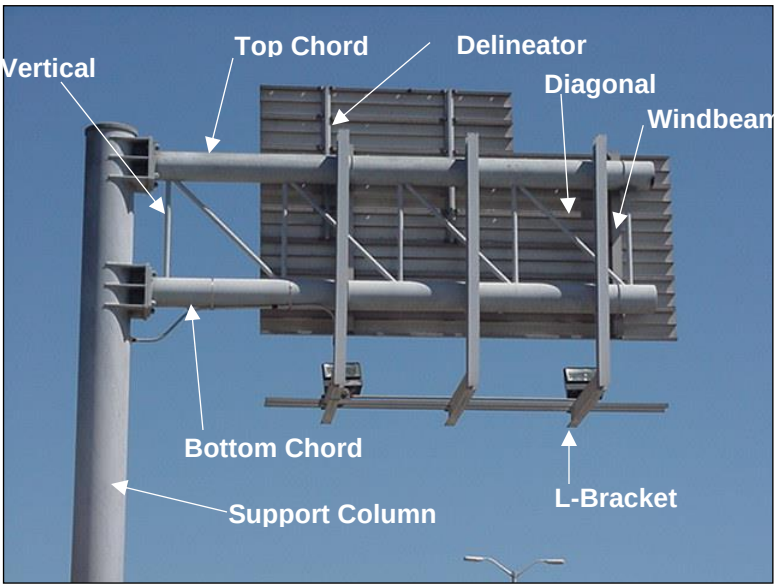

Figure 4.2.1-16: Cantilever Sign Structure Nomenclature.

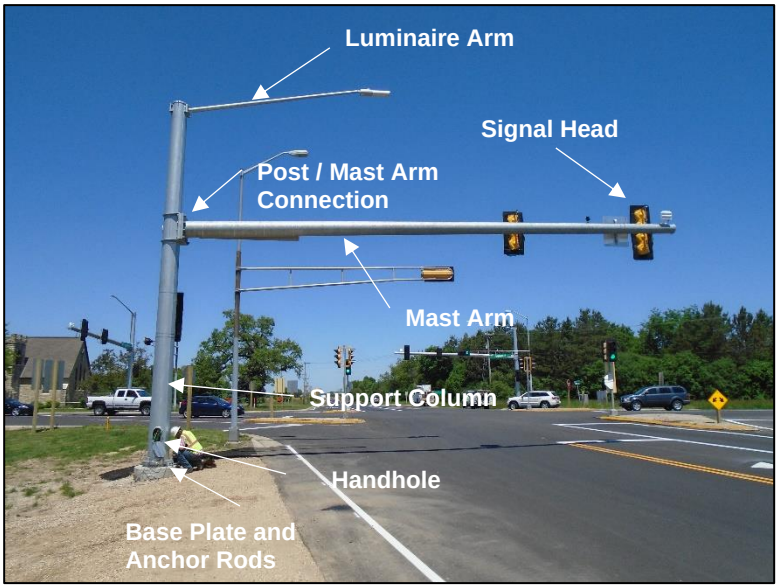

Figure 4.2.1-17: Signal Mast Arm Nomenclature

Sign Member Numbering System

Refer to Figure 4.2.1-18 through Figure 4.2.1-20 for the correct numbering system to be used when referencing the members in a frame. In this system, the side of the frame with the signs mounted on it will always be referred to as the front side. If the structure spans both directions of traffic, and signs are located on both sides, the signs for northbound or eastbound traffic are considered to be on the front. The four chords will be referenced as LF, lower front; UF, upper front; UB, upper back; and LB, lower back. The panel points will be numbered from left to right, (while looking at the front of the frame) starting at zero. To describe any member on the space

frame, the reference points at its two ends are used. For example, the second vertical on the front of the frame would be called member UF1-LF1.

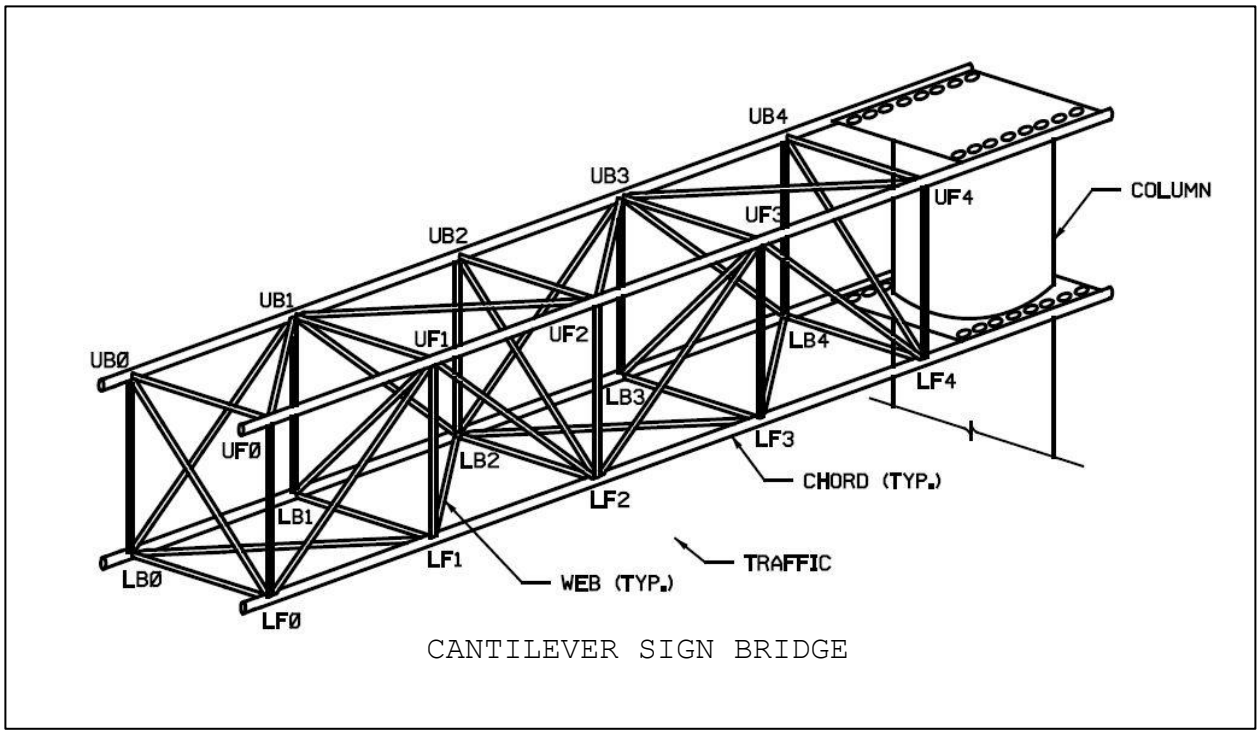

Figure 4.2.1-18: Typical Inspection Numbering System for Cantilever Sign Bridge.

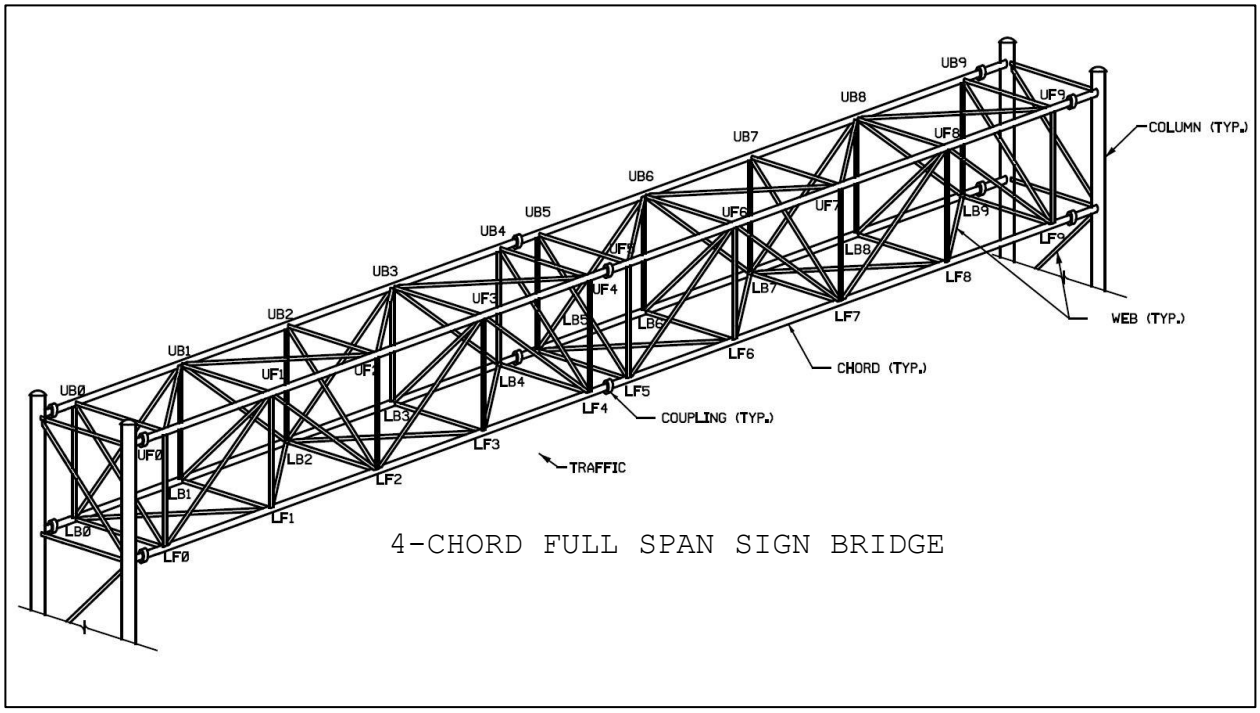

Figure 4.2.1-19: Typical Inspection Numbering System for 4-Chord Full Span Sign Bridge.

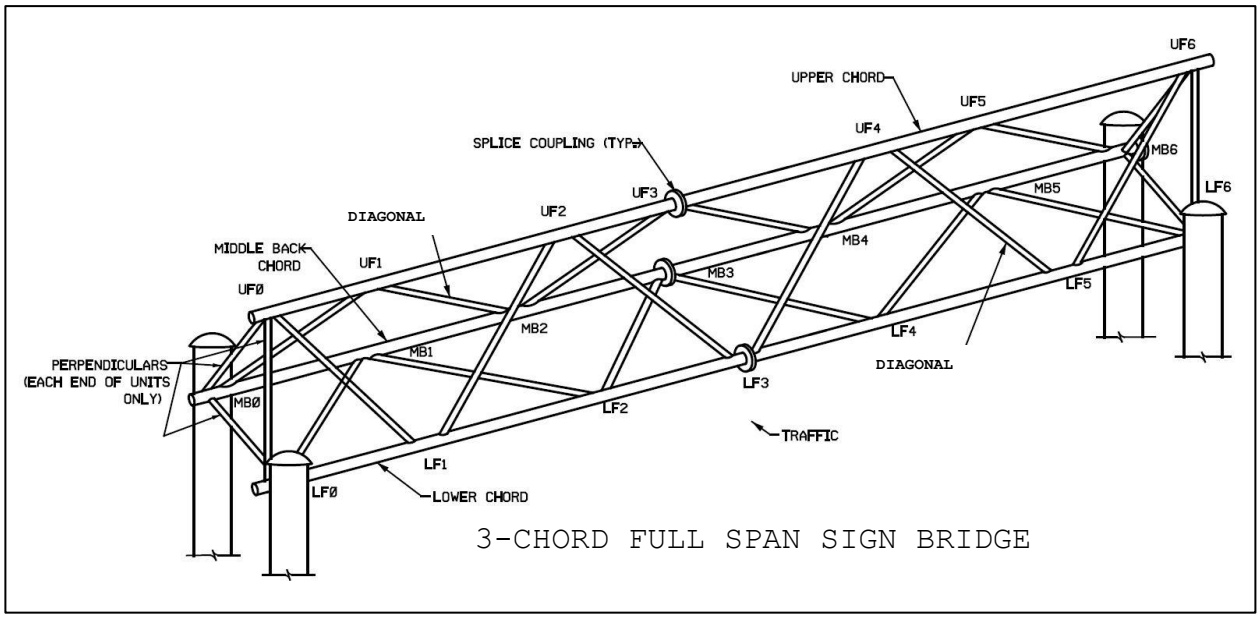

Figure 4.2.1-20: Typical Inspection Numbering System for 3-Chord Full Span Sign Bridge.

4.2.2 Overhead Structure Inspection

A successful, quality inspection incorporates proper safety procedures, a well-planned sequence, and sound inspection methods. Other Parts in this Manual establish the basics for conducting inspections. Part 1, Administration, reviews the inspector qualifications and duties.

Due to the large number of overhead structures that typically need to be inspected, it is important to have well-planned and well-managed inspection procedures to eliminate errors and inconsistencies. Some of the factors that will affect the sequence of the inspection are the type and size of the systems, traffic density, and availability of special equipment, preferences of the inspection supervisor, and requirements for traffic control.

The inspection equipment to complete inspections of these structures shall vary depending on structure type, structure geometry, height, traffic and clear zone adjacent to the structure. Inspectors have an array of options for inspecting overhead sign structures including climbing, bucket truck and miscellaneous tools for hands-on. General inspection procedure recommendations are provided at the end of this chapter.

Climbing

The support frames of the overhead sign supports should be climbed with a body harness and two lanyards per OSHA standards. Attach one lanyard to the lower horizontal. Stand on that member and attach the second lanyard to the next higher horizontal. Release the first lanyard, and climb onto the upper horizontal and repeat until the top is reached. Refer to Figure 4.2.2-1 for an inspector climbing the end support frame. The support frames and space frames are climbed with one lanyard attached to the structure at all times. The lanyards should be attached to the top of the frame whenever possible to shorten the fall distance. If it must be attached to a vertical or diagonal, the lanyard should be wrapped around the member and under itself at least once, forming a hitch knot, to keep the attachment from slipping down the member. If the lanyard is attached to a lower member of the frame, and a fall occurs, the inspector may fall into the path of a passing vehicle, and will be exposed to a potentially lethal fall factor. It is the inspector's responsibility to have the proper lanyards for climbing a sign structure, and use the proper connection points on the sign structure.

Figure 4.2.2-1: Inspector Climbing an End Support Frame.

Bucket Truck

Depending on the support tower, a bucket truck may be required to elevate an inspector into the truss or structure. Overhead sign bridges commonly have concrete tower pedestals and require the use of a bucket truck to gain access. Furthermore, an inspector may use a bucket truck in lieu of climbing a steel tower. However, unless a full roadway closure is available, use of a bucket truck to perform an entire truss inspection is rare and time consuming when compared to climbing. Other structures, such as butterfly and cantilever structures may be accessed and inspected entirely from within a bucket.

Sign support structures are typically very difficult to efficiently and safely climb. Consequently, bucket trucks are typically used for sign support inspections. Inspectors shall have harnesses on with a minimum of one lanyard secured to the bucket during an inspection. The inspector shall also have all pertinent inspection tools available within the bucket during inspection to safely and efficiently perform the inspection.

Inspectors shall have harnesses on with a minimum of one lanyard secured to the bucket during an inspection. When transferring from the bucket to the truss, the inspector shall secure his/her second lanyard to the truss prior to removing the lanyard from the bucket.

Traffic Control

Traffic control is paramount whenever using a bucket truck. It is necessary for the protection of the inspector and to inform and notify the travelling public of work ahead and overhead. This is especially true on highly trafficked roadways. The inspection project manager and inspector shall be well versed in entering lane closures within the Lane Closure System (LCS) whenever inspection of a sign on a state highway is required. The inspection project manager should contact the Regional Ancillary Inspection Program Manager to coordinate appropriate lane closures. Whenever possible, the inspector should make use of the shoulder and clear zone away from any lanes. On off-system roadways where space for a bucket truck is limited, it

may be possible to take a lane closure under the structure. The inspector should have numerous signs in place to caution approaching traffic of work ahead or possible lane shifts.

Miscellaneous Tools

All members and connections should be inspected by close visual examination. The inspection mirror is a useful tool for viewing opposite sides, undersides, or other hard-to-reach areas. During the inspection, reference points should be marked on the structure with chalk, keel, or permanent marker. These reference points are useful for describing the location of damaged members. A stick rule and/or measuring tape is a useful tool when taking photographs for scale or measuring deficiencies. Laser distance finders are also beneficial for a climbing inspector to determine the minimum vertical clearance on a structure. A camera should be on hand at all times to photograph any condition state 3 or 4 defects on the structure.

When climbing a structure, it is important for the inspector to secure these tools to his/her person so as to not have them fall onto traffic below. Retractable devices are easily attached to an inspector's harness or inspection bag and permit ease of use while preventing falls.

4.2.2.1 Visual Inspection

Overhead Sign Structures

Visual inspection of vertical frame supports should detect most deficiencies such as corrosion; buckled, bent, ruptured, cracked or missing members; cracked, incomplete or excessively-ground welds; and gusset plate cracks.

When paint loss or surface corrosion is observed, the adjacent area should be sounded with a hammer to check for internal corrosion. In addition, the lower five feet of the support should be sounded. These areas are more susceptible to internal corrosion of the members, leading to either partial or complete section loss. This condition is most often found around the base of the supports, especially if a grout pad is present. Internal corrosion also causes weep holes in the grout pad to be blocked with rust scale, thereby trapping moisture and intensifying the problem. The weep holes should be checked for blockage, and cleared out whenever possible.

On painted structures, isolated areas of cracked or splitting paint can be an indication of an overstressed section. These areas should be examined for other signs of distress. If an area is suspected of internal corrosion, the remaining thickness of the member needs to be determined. An ultrasonic thickness gage (D-meter) should be used in and around the suspected areas to determine the extent (if any) of section loss.

The connections of the spans to the support frames should be visually inspected, and the bolts hand checked for tightness. Typical problems which may be encountered include missing post-to-truss connection bolts; loose nuts on the post-to-truss connection bolts; improper use of lock washers; bolts that are not long enough to fully engage all of the threads of the nuts; and oversized holes (cut in field) for the post-to-truss connection bolts. All deficiencies should be recorded. Any burned or otherwise rough holes should be closely inspected for cracks.

Vertical post supports should be inspected similar to the support frames. All nuts and bolts should be checked for tightness. Loose bolts can cause some of the connection plates to bend or shift.

Space frames are visually inspected for corrosion; buckled, bent, ruptured, cracked, or missing members; incomplete, excessively-ground, or cracked welds; porous splice flange castings; cracked splice flange plates; and span camber. Refer to Figure 4.2.2-2 through Figure 4.2.2-6 for typical deterioration of steel and aluminum members. The flange splice nuts and bolts should be hand checked for tightness using a wrench or inspection hammer. Corroded areas may need to be checked for internal corrosion by a D-meter as described for the vertical frame supports.

Figure 4.2.2-2: Steel Corrosion in a Vertical Member.

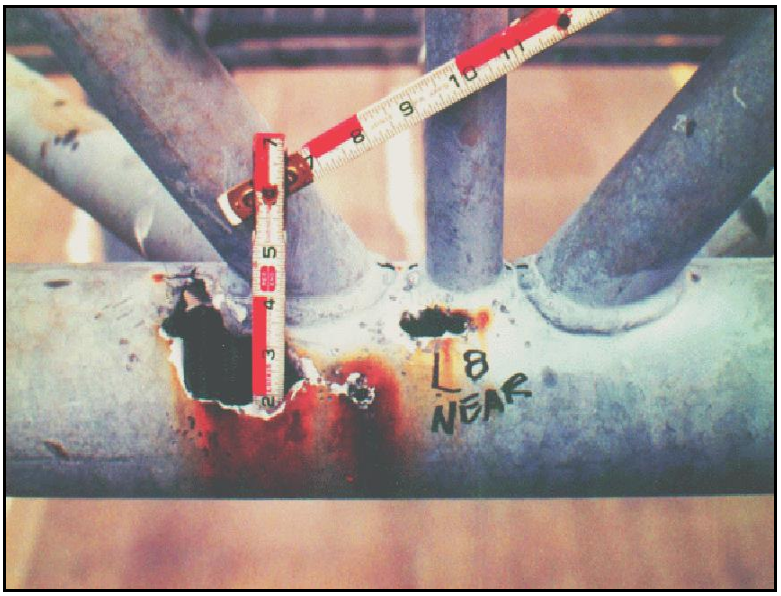

Figure 4.2.2-3: Corrosion in a Horizontal Steel Chord Member.

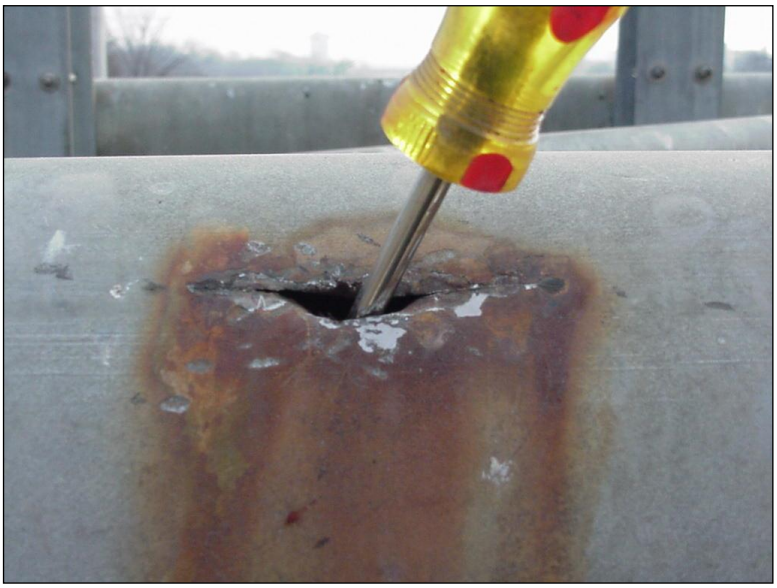

Figure 4.2.2-4: Split Associated with Corrosion in a Horizontal Chord Member.

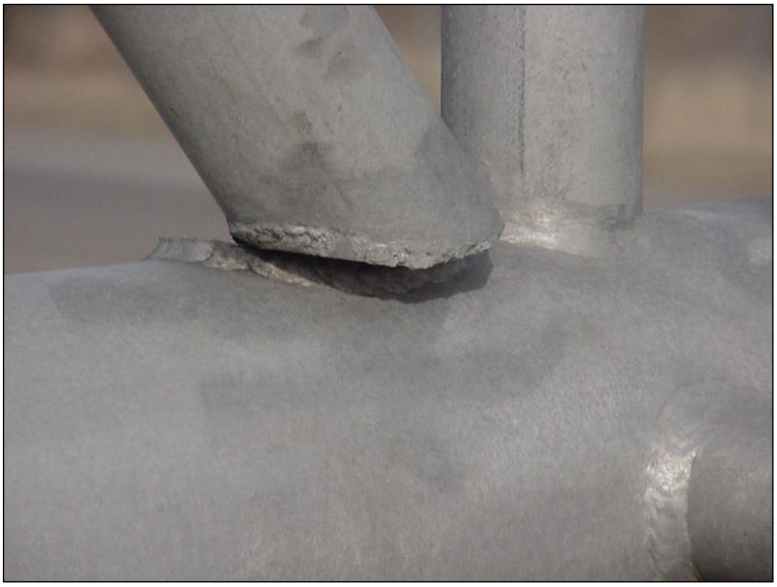

Figure 4.2.2-5: Broken Weld at an Aluminum Diagonal Member.

Figure 4.2.2-6: Crack through an Aluminum Gusset Plate.

On the full span structures, the space frames should be checked to insure there is a positive camber present in the frame. The space frames on the full span structures are either fabricated with this camber built in, or shims are bolted between the top flange splice plates to create a camber. If the space frame is sagging, the inspector should check for proper assembly/ shim placement. If assembled improperly, the frame may have distorted and may have serious structural problems. During the initial inspection of a sign structure, the inspector should briefly review the shop drawings and compare versus the structure in the field to ensure proper installation of the truss. Refer to Figure 4.2.2-7 and Figure 4.2.2-8 for examples of sign structures with improperly installed trusses.

Figure 4.2.2-7: Truss installed upside down. Diagonals of Cantilever Should Angle Down Towards Sign Panel.

Figure 4.2.2-8: Right Truss Installed Upside Down. Truss Diagonals Should Run Down Towards Center From Upper Chord.

The vertical clearance shall be measured at the lowest point over each lane and the lowest reading recorded on the inspection form. This is usually from the catwalk L-bracket or the bottom of the sign panel. If the lowest point cannot be reached safely by hand, the measurement can be taken from a higher point. The distance from the higher point to the lowest point is measured and subtracted from the first measurement. If available, a laser distance meter is an excellent tool for obtaining these clearance measurements; however care must be taken when purchasing these as cheaper models lose accuracy in cold weather. The accuracy of the meter should be checked periodically with the survey rod, or other measuring device. Also, if a laser distance meter is used, a minimum of three measurements should be taken at each location with the minimum measurement being recorded as the vertical clearance. Alternatively, the clearance may be measured from the ground with an inverted survey rod held on the lowest point on the structure. Several positions should be checked to determine the minimum clearance, and suitable traffic control must be in place.

Overhead Sign Supports and Signal Mast Arm Structures

Regardless the support structure type, special attention should be given to the welded connection of the mast arm to the connection plate. This weld is subject to a high level of stress due to the weight of the arm and a high number of cycles from wind vibrations, truck induced loads, etc., which contributes to the development of fatigue cracks. The connections of mast arms to posts should be visually inspected, and the connection bolts hand checked for tightness.

Typically the mast arm will be fabricated with an upward angle in at the connection plate. This creates an increase in elevation as the arm extends outward over traffic. The inspector shall inspect post to mast arm connection faying surface to ensure the plates are flush. The use of washers to provide camber is not allowed. If assembled improperly, the arm may deflect downward with vertical clearance issues with the traffic below. Refer to Figure 4.2.2-10 for an example of an improperly installed column to arm connection.

Figure 4.2.2-9: Typical Column to Traffic Signal Mast Arm Connection.

Figure 4.2.2-10: Column to Sign Mast Arm Connection with Washers Between Faying Surfaces.

Typical problems observed in post-to-mast arm connections encountered in the field include faying surfaces not flush or gapped due to washers installed between plates; loose nuts on the connection bolts; improper use of lock washers; bolts that are not long enough to fully engage all of the threads of the nuts. Improper installation of DTIs includes placing a washer between the DTI and the bolt head / nut; and observing the feeler gauge inserts all the way to the bolt shank between the DTI dimples (non-refusal). Refer to Figure 4.2.2-11 for non-refusal of a feeler gauge at a DTI bolted connection. All deficiencies should be recorded. Any burned or otherwise rough holes should be closely inspected for cracks.

Figure 4.2.2-11: Non-Refusal of Feeler Gauge at DTI Installation.

The vertical clearance shall be measured at the lowest point over each lane and the lowest reading recorded on the inspection form. This is usually from the bottom of a signal or sign closest to the post. If available, a laser distance meter is an excellent tool for obtaining these clearance measurements; however care must be taken when purchasing these as cheaper models lose accuracy in cold weather. The accuracy of the meter should be checked periodically with the survey rod, or other measuring device. Also, if a laser distance meter is used, a minimum of three measurements should be taken at each location with the minimum measurement being recorded as the vertical clearance. Alternatively, the clearance may be measured from the ground with an inverted survey rod held on the lowest point on the structure. Several positions should be checked to determine the minimum clearance, and suitable traffic control must be in place.

Element Level Inspection

On the inspection report form, each overhead sign structure and support element is recorded in units of Each, save for truss chords and mast arms. It is important to note that sign structure and support elements do not possess any material defects. Rather, each element condition state definition provides all the defect language within it. Therefore, the inspector is tasked with noting all defects observed during the inspection under the element notes and determining the appropriate element condition state. For an Each unit of measure, the coded Condition State applies to the entire element. In this manner, an Each unit of measure essentially acts as an overall rating for that element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

4.2.3 Overhead Sign Elements

All overhead sign structures and supports are broken down into basic components and recorded within the inspection report. For continuity throughout the state, the structures shall be evaluated under designated elements. All sign elements are recorded in units of each with the exception of the truss chords and mast arms which are recorded in lineal feet.

Each element shall be evaluated in its entirety and assessed under the appropriate element condition state. Generalized material defects and common deficiencies typically observed in these elements are included within condition state definitions as noted within this chapter.

For truss chords and mast arms, each linear foot includes the three dimensional projection through all of the of the truss members within the cross section. This cross section is inspected and evaluated for any distress with deficiencies captured and quantified under the appropriate element condition state. The element condition states quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

4.2.3.1 Base/Foundation (Element 8701)

As discussed in various other sections of this Manual, the most common signs of concrete deterioration on the foundations include: cracking, scaling, spalling, and delamination.

Impact damage, spalling, scaling, and most cracking are typically visible to the naked eye. Some cracking or delamination may be below the surface; therefore, the concrete can be typically struck with a standard inspection hammer to locate these deficiencies. The delaminated or internally cracked areas will reveal a hollow sound when struck with a standard inspection hammer. When these delaminated sections are encountered, the loose outer sections should be broken off to determine the extent of the deterioration. The inspector should also determine if there is any erosion or undermining of the footing that could lead to an instability problem.

All deficiencies shall be recorded by describing the type of deficiency (spall, crack, etc.), location, size (length, width, and depth), orientation, and severity of the defect. Exposed steel reinforcement should be noted, as well as the severity of corrosion. If corrosion has caused section loss, then the percentage of section loss should be determined and noted. Refer to Figure 4.2.3-1 through Figure 4.2.3-6 for views of common foundation deterioration.

Figure 4.2.3-1: Map Cracking with Efflorescence on a Four Chord Sign Structure (Condition State 3).

Figure 4.2.3-2: Map Cracking with Delamination and Spall on a Butterfly Foundation (Condition State 3).

Figure 4.2.3-3: Mast Arm Foundation Initially Installed Below Grade (Condition State 2)

Figure 4.2.3-4: Cracked, Delaminated and Spalled Concrete Foundation (Condition State 3)

Figure 4.2.3-5: Foundation Spall Exposing Anchor Rod (Condition State 3).

Figure 4.2.3-6: Anchor Rods Installed Offset from Center Foundation (Condition State 2 if Verified by Engineer within Report).

The Wisconsin Department of Transportation (WisDOT) has phased out the use of grout pads on the foundations of traffic operations support systems, due to past problems. The current practice is to leave the void created by the leveling nuts between the concrete foundation and the base plate empty, and to wrap the exterior edge of the base plate with metal screening to prevent rodents from nesting in the void when electrical conduit or wires are present. The open void allows moisture to drain down the support tubes and evaporate, rather than saturating the grout pad, which in turn would contribute to the corrosion of the base plate and anchor bolts.

However, some older structures may still have grout pads in place. Problems found with the foundation grout typically include: cracks, delamination, moisture, voids, and loose grout. Sources of grout problems include poor grout mix; poor consolidation; freeze-thaw cycles; poor installation; and the action of deicing salts. Grout pads should be visually inspected for cracking, scaling, delamination, efflorescence, the presence of moisture, and loss of grout. The grout pad should also be sounded with a standard inspection hammer to detect subsurface problems similar to the base/foundation. If the grout pad is deteriorated, the recommended course of action is to have the grout pad removed in its entirety and a metal rodent screen installed around the base plate if electrical conduit or wires are present. Prior to removing substantial portions of the existing grout, the inspector should verify that leveling nuts are in place and tight against the bottom of the base plate. Refer to Figure 4.2.3-7 for an example of typical deterioration of a grout pad on a sign structure.

Signal mast arm structures should be constructed with the top of the concrete foundation at or just above the surrounding grade. Soil covering the foundation acts to promote the deterioration of the protective coatings on the sign structure members. It may be necessary for the inspector to remove fill material from a top the foundation.

This element includes reinforced concrete foundations used for High Mast Lighting Structures, Overhead Sign Structures and Overhead Sign Supports. Four chord, two-chord, cantilever and butterfly structures will commonly be supported by vertical concrete columns. There columns are seen as extensions of the foundation and shall be included under this element. If a grout pad is present, it is also evaluated in this element. If Anchor Rod(s) were placed eccentrically, but the configuration has been approved by structural engineers, note that eccentricity in the notes with measurements for off-set.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element shows no deterioration.

- Condition State 2: Minor cracks and spalls may be present in the foundation, but only minimal reinforcing steel is exposed. When efflorescence is present, it is minor and there is no evidence of rust staining. Grout pad (if present) is in good condition. Minor erosion around foundation may be present, but does not affect structural capacity.

- Condition State 3: Many spalls are present. Corrosion of reinforcement and/or loss of concrete section is evident though not sufficient to warrant structural analysis. Grout pad (if present) has moderate cracking, spalls, or delaminations. Erosion may be present that reduces the foundation embedment significantly but does not pose a threat to the stability of the structure.

- Condition State 4: The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

Figure 4.2.3-7: Deteriorated Grout Pad (Condition State 3).

4.2.3.2 Steel Anchor Rods (Element 8702)

Anchor rods are typically secured to the sign structure via leveling nuts, anchor nuts and jam nuts. Jam nuts should not be confused with lock nuts which have a ring of nylon around the interior to “lock” the nut to the threads of a bolt.

Problems with anchorages include: bent or corroded anchor bolts; missing washers; loose or missing anchor rod jam nuts/ leveling nuts; and fatigue cracks. Sources of anchorage problems include misaligned anchor rods during construction (which is typically a result of the failure to use a template); poor or damaged galvanized coating; and incorrect or poor tightening methods.

Anchor rods, nuts, and washers should be visually inspected for corrosion; loose or missing lock nuts, leveling nuts, and washers; bent rods; and nuts not fully engaged (threaded). The tops of anchor rods should be tapped with a standard inspection hammer. This may reveal dull or hollow sounds, which could indicate a loose anchor rod or a possible fatigue crack in the rod. If a crack is suspected, the anchor rod should be ultrasonically tested to verify the presence and location of a crack. All cantilever structure anchor rods shall be ultrasonically tested with each routine inspection. Due to the excessive galloping of the structure from natural and induced wind loading, fatigue and overstressing is more prevalent in anchor rods for these structures.

The nuts and washers should be tapped with a standard inspection hammer to check for looseness. If a nut is not clamping the base plate to the foundation (or to the leveling nut), the anchor bolt is ineffective. This results in increased stress for the other anchor rods. Base plates installed flush to the foundation should have anchor rod nuts snug tight (snug tight defined as the full effort of an ironworker on an anchor nut using a 12 inch spud wrench). Base plates to leveling nuts shall be installed following the required torque and tensioning requirements. The washers should also be tapped as a nut could be tight on the anchor rod, but not on the base plate and the nut may not move when tapped with a hammer. A loose washer will reveal an under-tightened nut. Excessive rod length between the base plate and

foundation, “stand-off” distance, may also create increased stresses within the anchor rods. The length from the bottom of the leveling nut to the top of the foundation should not measure more than the diameter of the anchor rod. The inspector shall note any anchor rods with excessive standoff and may recommend load rating to determine whether the rods are stressed beyond capacity.

All initial sign structure inspections with base plate to leveling nut foundations shall have torque verification testing performed on one anchor rod. The inspector shall note within the inspection report which anchor rod was tested and results of the verification. All sign mast arm structures and cantilever sign structures shall have the anchor rod located closest to the centerline of the superstructure and below the mast arm or chords torque verified. Refer to Figure 4.2.3-8 for the required anchor rod torque verification testing location on mast arm and cantilever structures. The importance of verifying this anchor rod is due to the loading of the foundation by the superstructure. This area of base plate and anchor rod will typically be in compression once the superstructure is installed. If the anchor rods were tensioned prior to the superstructure installation, the loading on the rods below the cantilevered superstructure may take the rod out of tension thus creating an improperly installed structure susceptible to elevated stresses within the other anchor rods.

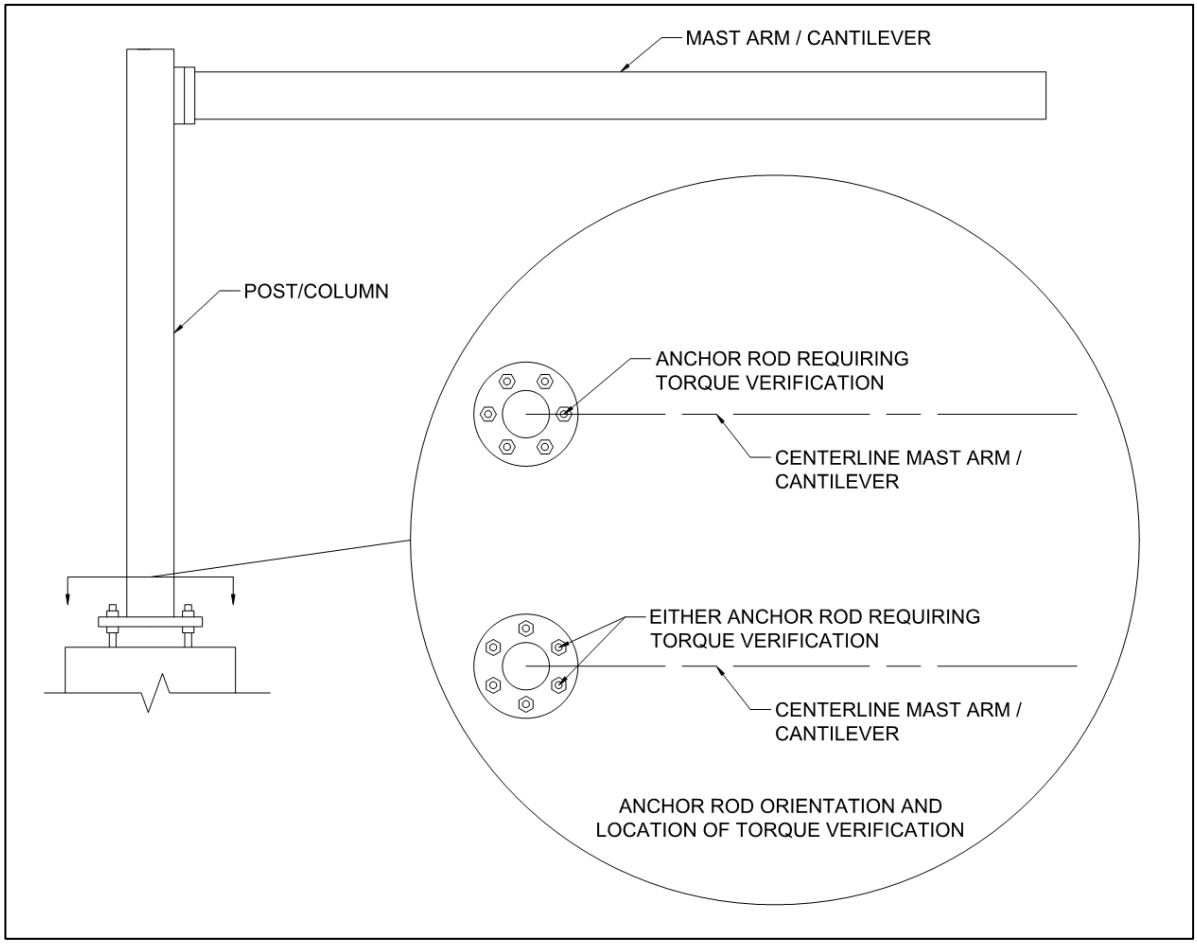

Figure 4.2.3-8: Varying Anchor Rod Orientations and Required Location of Torque Verification for Mast Arm / Cantilevered Structures.

Torque verification testing for each subsequent routine inspection after is at the inspector's discretion. Torque verification testing requires the use of a calibrated torque wrench. The inspector may also find it advantageous to use a torque multiplier to reduce the amount of torque the inspector has to apply to the wrench during verification (the inspector must be aware of the appropriate torque conversions when using the multiplier). Refer to Figure 4.2.3-9 for view of an inspector using a torque multiplier for a torque verification test.

Figure 4.2.3-9: Inspector Using Torque Multiplier to Verify Anchor Rod Tension.

On older structures, the inspector may find various details, such as the anchor rods having lock nuts or lock washers. The first top nut shall be fully engaged (threaded), and the second top nut shall be engaged so the anchor rod has the threads are exposed through the second (lock) nut. The second top nut (lock nut) is not always a normal structural nut. Frequently, a jam nut (low profile nut) is used as the lock nut. A jam nut should only be used as a lock nut in conjunction with a normal structural nut. Refer to Figure 4.2.3-10 through Figure 4.2.3-15 for typical problems of anchor bolts found by inspectors. If a lock washer is encountered on an anchor rod, it should be noted as such in the report. Since the anchor rod assembly is a tensioned connection, lock washers are not permitted to be used as they can creep out of the connection.

This element defines anchor rods, anchor nuts, leveling nuts, and washers connecting the column support to the foundation. Quantity is per each rod.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element shows no deterioration.

- Condition State 2: Minor corrosion of the element is present, but the rods appear to be properly tensioned and the nuts are fully engaged. Anchor rods may be slightly out of plumb (1:40 or less) without need for remediation, or out of plumb between 1:20 and 1:40 with proper beveled washers, and PE Stamped calculations on file to certify the configuration is ok. Anchor rod standoff exceeds current standards, but is within acceptable limits based on calculations.

- Condition State 3: Moderate corrosion of the elements may be present, but not enough to warrant structural analysis. Lock washers may be present. Anchor rod standoff exceeds current standards, and there is no analysis on file and rods appear to be performing adequately.

- Condition State 4: Severe corrosion is present. Anchor rods are not properly tensioned, nuts may be missing or not fully engaged. Anchor rods are greater than 1:40 out of plumb w/o remediation (Greater than 1:20 requires immediate replacement). Anchor rod standoff exceeds current standards, and rod(s) show signs of bending, movement, buckling, or elongation. The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

Figure 4.2.3-10: Anchor Bolt Installed Out-of-Plumb Greater than 1:40 (Condition State 3).

Figure 4.2.3-11: Severe Corrosion of the Leveling Nuts (Condition State 4).

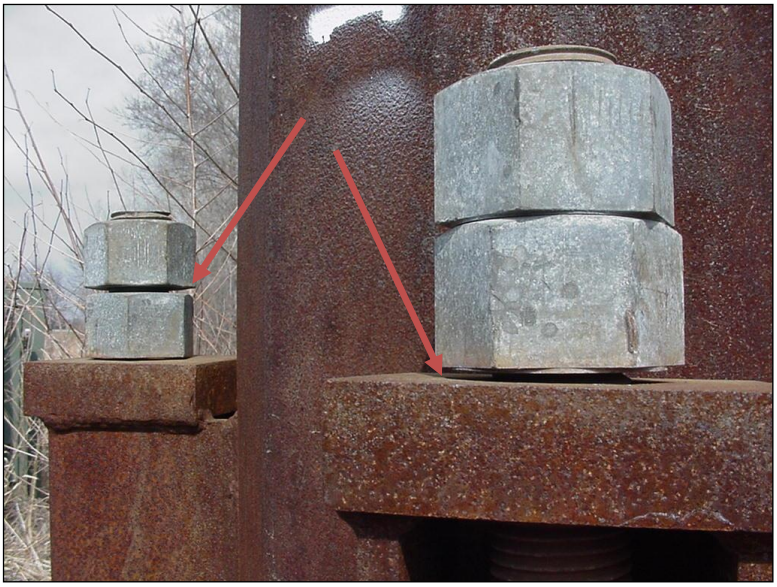

Figure 4.2.3-12: Anchor Nuts and Jam Nuts not Fully Tight to the Base Plate, or to Each Other (Condition State 4)

Figure 4.2.3-13: Leveling Nuts Not Fully In Contact with the Base Plate (Condition State 4).

Figure 4.2.3-14: Excessive Anchor Rod Stand-Off from Foundation (Condition State 3).

Figure 4.2.3-15: Anchor Bolt Out-of-Plumb Due to Impact (Condition State 3).

4.2.3.3 Base Plate(s) (Aluminum or Steel) (Element 8703)

The vertical posts for sign structures will typically be welded to a horizontal base plate. The base plate is fastened to the foundation via the anchor rods. It is common for base plates to have stiffeners welded between the anchor rod locations to add rigidity to the welded socket connection and reduce the impact of fatigue on the connection.

Aluminum columns for sign supports are rare if not non-existent in Wisconsin. However, they still exist for sign structures. For aluminum columns, the column does not connect directly to the base plate. Base casting clamps are used to clamp the bottom of the support frame. The base casting clamps are bolted to the concrete foundation utilizing anchor bolts embedded in concrete. The vertical support frame is clamped between two base-casting clamp halves and secured by horizontal collar bolts. Base casting clamps can be over tightened, thus causing over stressing, and possible failure. Refer to Figure 4.2.3-16 for a base casting clamp that has failed due to over stressing.

Figure 4.2.3-16: Failed Aluminum Base Casting Clamp Initiated by Over-tightening the Stainless Steel Horizontal Column Bolts.

The base plate shall be visually inspected for any deformity due to installation or unlevel foundation. The welds of the post to plate connection and stiffener welds (vertical to post and horizontal to base plate) shall be inspected. If castings are present, cracking and bolt tightness shall be inspected. If necessary, ultrasonic or magnetic particle testing may be required to determine the extent of weld cracking if visually present during routine inspections.

This element defines the base plates, flanges, casting clamps, gusset plates, seam welds, and welds at the connection of the column support to the foundation. Quantity is one each per base plate.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element shows no deterioration.

- Condition State 2: Minor corrosion of the element is present. No cracking of the element is observed. Casting clamp, if used, has no more than one horizontal bolt loose.

- Condition State 3: Moderate corrosion of the element is present. Cracks may be present on the base plate to column support connection weld, but have been arrested or are no longer active and do not affect the capacity of the plate. The base plate may be distorted (dished). Casting clamp, if used, has no more than two horizontal bolts loose.

- Condition State 4: Severe corrosion is present. Cracks may be present on the base plate to column support connection weld that have not been arrested. Section loss is significant and may affect the ultimate strength or serviceability of the element. Three or more horizontal casting clamp bolts are loose or missing, or cracks exist in the casting clamp assembly. The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.2.3.4 Column Support(s) (Aluminum or Steel) (Element 8704)

For two-column supports, the frame consists of two steel or aluminum pipe columns connected by diagonal bracing pipe or angle members. Some two-column support frames also have horizontal pipe or angle members between each diagonal branch member. The column base design differs for steel and aluminum. Aluminum casting plates shall be evaluated under 4.2.3.3 Base Plate(s) (Aluminum or Steel) (Element 8703).

Tri-chord full-span sign structures may be supported at each end by either a two-column support frame or by a single column. Two-chord and monotube sign structures will be supported at each end by a single column. All sign supports are comprised of a single post. Typical design includes a slight angle from vertical between the column and base plate during fabrication. Once the mast arm and signs/signals are installed, the intent is the loading will deflect the column to plumb. The inspector shall note whether the column is out of plumb (taking into account the column is most likely tapered).

This element includes the vertical posts, diagonal members, and circumferential welds for the column support(s) on the structure. The quantity is one each per vertical column. Depending on the structure type and configuration, there may be one or two columns per support.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element shows no deterioration.

- Condition State 2: Minor corrosion is present. Standing water may be observed inside the post.

- Condition State 3: Moderate damage or corrosion is present, but does not warrant structural review. Cracks may be present on the pole, but have been arrested or is no longer active.

- Condition State 4: Heavy damage or corrosion of elements with section loss. Cracks may be present on the pole that have not been arrested. Elements may be misaligned or have severe impact damage that warrants structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

Figure 4.2.3-17: Square Support Column Cantilever (Condition State 1).

Figure 4.2.3-18: Base of Post Distorted Out of Plane Due to Impact (Condition State 3).

4.2.3.5 Truss Chord(s) / Mast Arm(s) (Aluminum or Steel) (Element 8705)

For a four-chord sign structure, each space frame is constructed of two vertical parallel-chord plane trusses. Each truss consists of a top and bottom chord connected by vertical and diagonal members. The diagonals are arranged in the pattern of a Warren Truss where the diagonals alternate between being directed down toward the center of the span and being directed up toward the center of the span.

To form the space frame, the two trusses are set parallel to each other and joined together by horizontal transverse members and horizontal diagonal members. These members are

welded/bolted in place between the two top chords and between the two bottom chords, thus forming the rectangular cross-section of the space frame. Interior diagonal members are also framed between the two vertical trusses. Each interior diagonal connects a top panel point of one truss with the corresponding bottom panel point of the other truss. The interior diagonals alternate in direction with each panel. Refer to Figure 4.2.3-19 for an example of a four chord truss.

Figure 4.2.3-19: Four Chord Truss.

Butterfly structures will typically be comprised of one or two pairs of horizontal steel tubes without trussing, or two to four chords with trussing. The total length for the inspection shall be the measurement from end to post connection for tubes bolted directly to post, refer to figure below, or end to end of tubes / truss when continuous and connected to post. The exact makeup of the structure is dependent on whether the structure has one or two sign panels as well as the age of the structure. Refer to Figure 4.2.3-20 for an example of a butterfly structure with two pipes bolted to a single post with no trussing. Refer to Figure 4.2.3-21 and Figure 4.2.3-22 for an example of a single and dual mast arm support structure, respectively.

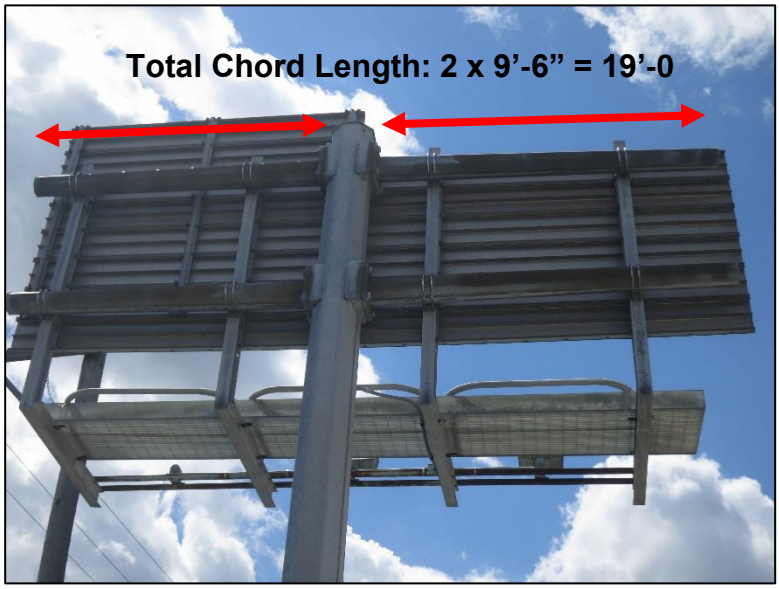

Figure 4.2.3-20: Butterfly Chords Without Trussing.

Figure 4.2.3-21: Signal Mast Arm Length.

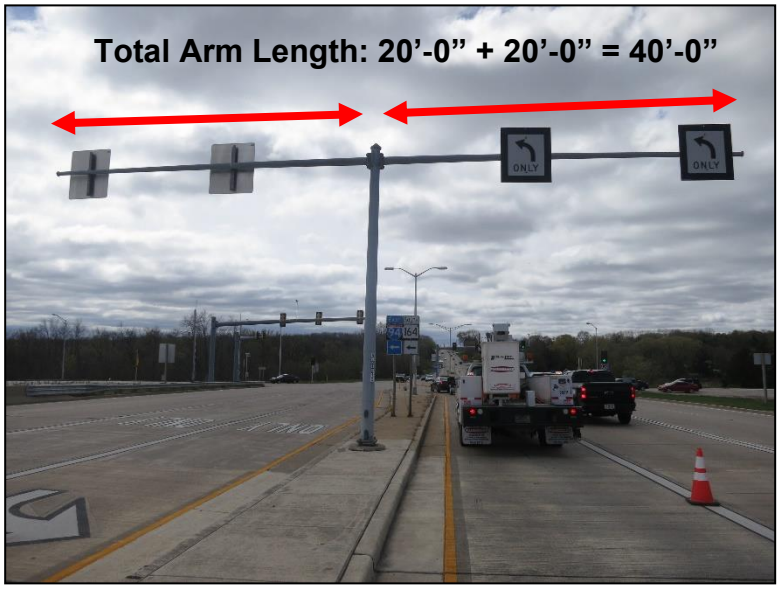

Figure 4.2.3-22: Dual Sign Mast Arm Length.

Cantilever structures are typically two-chord or four-chord truss systems.

Sign support mast arms are typically constructed of steel plate rolled into a tapered pipe section with a wider base located at the post to arm connection. A seam weld runs down the center of the arm. On longer arms, a slip joint splice may be observed to extend the arm a greater distance. This splice joint shall be evaluated under the element 4.2.3.7 Connection(s) - Splices for Columns, Chords, Arms (Element 8707).

Mast arms are typically fabricated with a slight angle of incline from the post to arm connection. This ensures that the tip of the arm is higher out over traffic reducing the vertical clearance impacts. The inspector shall sight down the mast arm to determine whether the arm deflects lower than the post to arm connection or if an angle of ascent is not present at the post to arm connection. It should be noted that shimming or altering the leveling nuts of the column base is not permitted to provide the appropriate camber in the mast arm. Refer to Figure 4.2.3-23 for an example of a mast arm with a fabricated angle of ascent.

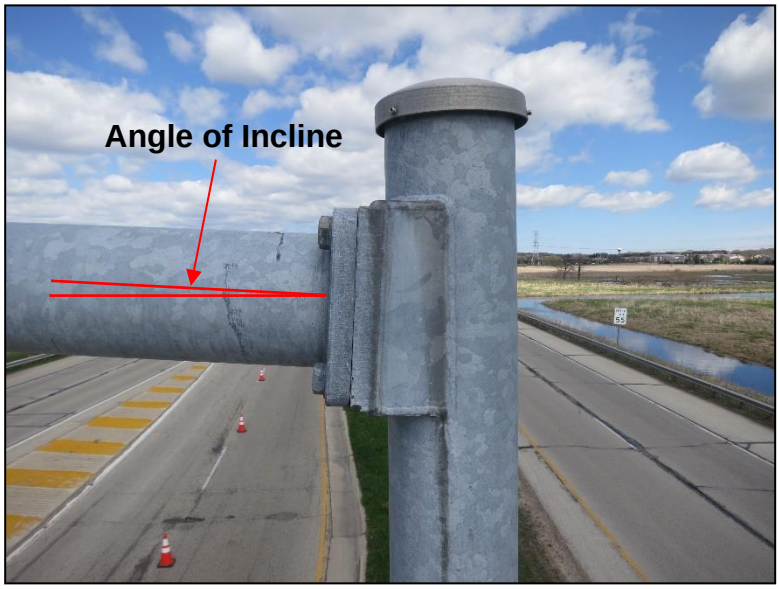

Figure 4.2.3-23: Angle of Incline in Mast Arm from Post to Arm Connection.

This element defines the chords of a truss system, or mast-arms. It also includes all diagonal members and struts integral to the truss. Total Quantity is the length of the truss or mast arm span, regardless of number of chords. (i.e. if Truss is 40' long, quantity is 40 LF regardless if 2, 3, or 4 chord truss; if mast arm is 40' long, quantity is 40 LF; dual arm structure where each arm is 20', quantity is 40 LF).

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element shows no deterioration.
- Condition State 2: Minor corrosion is present, but with no discernable section loss. Superficial damage to the element may exist.
- Condition State 3: Moderate damage or corrosion is present, but does not warrant structural review. Cracks may be present, but have been arrested or are no longer active.
- Condition State 4: Heavy damage or corrosion of elements with section loss. Cracks may be present that are active. Elements may be misaligned or have severe impact damage that warrants structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.2.3.6 Connection(s) - Column to Truss Chord/Arm (Element 8706)

Four chord structure chord to truss connections are typically comprised of saddles welded to each of the two-column frames which support the ends of the chords of the four-chord full-span. Horizontal holes are drilled into the ends of each of the chords. The four-chord space frame is placed in the saddles and bolted to the saddles. The bolts on the older structures were not stainless steel, and typically exhibit heavy corrosion. Heavily corroded end connection bolts should be noted in the inspection report. Refer to Figure 4.2.3-24 for an example of a saddle bracket connection for a four chord truss to post connection.

Figure 4.2.3-24: Four Chord Truss to Post Saddle Bracket Connection.

Another style of four-chord cantilever sign support found throughout the State utilizes flange plates to connect the space frame to a square support column. Square stubs are welded to the square support column, which support the space frame. The welds between the support column and the space frame have been prone to cracking. Refer to Figure 4.2.3-25 for a view of the square stubs welded to the square support column.

Figure 4.2.3-25: Square Stubs Welded to Support Column.

If a single post supports the tri-chord space frame truss, the typical connection will be bolted to the top and bottom chord members to the post by plates welded to the chord members. If a two-column support frame supports the tri-chord space frame, the bottom chord is U-bolted to a "T-beam" welded between the two support columns. The back chord member is U-bolted to the back support column. In the past, the T-beam welds have exhibited cracks along the bottom edge of the weld. Many tri-chord structures have been retrofitted with T-section repairs to correct the cracked welds. Close attention should be paid to these areas to ensure welds are not cracking. Refer to Figure 4.2.3-26 for a tri-chord truss to single post connection. Refer to Figure 4.2.3-27 for retrofitted T-section repairs.

Figure 4.2.3-26: Tri-Chord Truss to Post Connection for Single Post Structure.

Figure 4.2.3-27: Tri-Chord Bolted T-Section Repair.

For cantilevered structures, the supported end of the space frame, a horizontal plate is welded to the inside end of each of the four cantilever chords. Large plates and plate stiffeners are welded to the support column. Matching holes are fabricated in the support post plates and the horizontal plates on the chord members. Refer to Figure 4.2.3-28 for a typical space frame to support column on a cantilever structure.

Figure 4.2.3-28: Typical Space Frame to Post Connection on a Cantilever Structure.

Two-chord and monotube structure horizontal members are secured to a single post at each end by a bolted clamp connection. The clamping assembly is bolted to the post and an

alignment pin is used to secure the horizontal tube to the clamp. Refer to Figure 4.2.3-29 for a monotube to support post connection.

Figure 4.2.3-29: Typical Monotube Chord to Support Column Connection.

Butterfly structures will typically have the horizontal tubes U-bolted to saddle brackets welded to the pipe column. Refer to Figure 4.2.3-30 for a view of a typical butterfly saddle connection.

Figure 4.2.3-30: Corroded U-Bolt Connection Securing Horizontal Sign Support Member to the Post of a Butterfly Sign Structure (Condition State 2).

Column to arm connections on sign support mast arms are typically comprised of welded and bolted plate connections. A vertical plate, plane parallel to the traffic lane, is welded to vertical or horizontal plates extending off the vertical post. A socket weld connects a vertical base plate to the mast arm. Corresponding bolt holes are located in both the vertical post connection plate and mast arm plate and subsequently bolted together with high strength bolts. Depending on the span and sign or signal load located on the mast arm, the number of bolts

securing the mast arm to post connection will range from 4 to 8 bolts. Refer to Figure 4.2.3-31 and Figure 4.2.3-32 for typical column to mast arm connections.

On older structures, the mast arm may be connected to the column via a clamping casting. This type of connection is not common for signal mast arm connections, however the premise of the connection is similar but without the welded plates to the column. The inspector should carefully check for any loose, corroded or missing connection bolts. The socket weld of the plate to mast arm connection should be inspected. Refer to Figure 4.2.3-33 for a post to arm clamped connection. Refer to Figure 4.2.3-34 for an example of missing bolts on a column to mast arm connection.

As these arms are cantilevered from the connection, the connection experiences high stresses in all dimensions. Therefore, inspection of the welds on the column, socket weld on the arm and bolts connecting the plates is critical. Cracked or poor welding and missing bolts will highly impact the structural capacity of the mast arm. The bolts on the older structures were not stainless steel, and typically exhibit heavy corrosion. Heavily corroded end connection bolts should be noted in the inspection report. The faying surface between the post and mast arm plates should be flush. Washers are not permitted between the plates. Refer to Figure 4.2.3-35 for cracks along a mast arm socket weld.

Figure 4.2.3-31: Column to Arm Connection with Horizontal Column Plates Welded to Vertical Plate. Note Washers Improperly Included Between Plates.

Figure 4.2.3-32: Column to Arm Connection with 8 Bolts. Note Horizontal and Vertical Plates Welded to Vertical Plate.

Figure 4.2.3-33: Column to Mast Arm Clamped Connection.

Figure 4.2.3-34: Missing Lower Connection Bolts of a 4 Bolt Connection (Condition State 4).

Figure 4.2.3-35: Transverse Weld Cracks Along Circumferential Socket Weld of Mast Arm to Vertical Plate Connection (Condition State 3)

Newer structures may be designed with the column to mast arm/cantilever connection being fastened with structural bolts and direct tension indicators (DTIs). An inspector can determine if a DTI was used by observing circumferential indentations on the edge of the component. Refer to Figure 4.2.3-36 for an example of a DTI. When DTIs are present the inspector shall note whether the WisDOT form DT2322 was submitted by the contractor and within HSI. Verification of proper DTI installation requires a 0.005 inch thick feeler gauge. Proper installation is verified by the number of refusals of the feeler gauge in the DTI gaps. It is important to note that a seemingly snug nut on a DTI connection does not necessarily

represent a properly installed connection. The inspector shall refer to DT2322 for determining proper DTI installation.

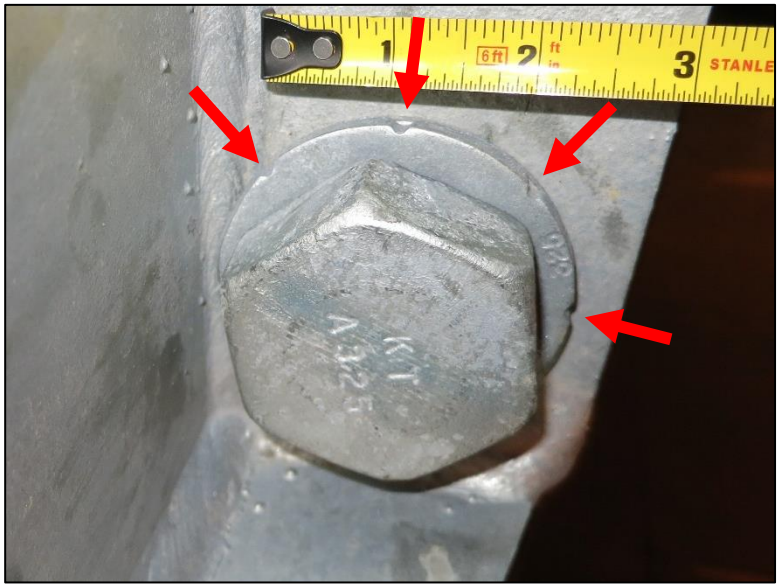

Figure 4.2.3-36: Indentations Along Edge Indicate the Component is a DTI.

This element defines the flange, gusset plates and hardware/fasteners connecting the span arms or chords to the column supports. Quantity is one each per connection location.

Refer to the following for condition state descriptions:

- Condition State 1: The element shows no deterioration.
- Condition State 2: Minor corrosion is present, but with no discernable section loss. Superficial damage to the element may exist, as may minor misalignments. The connection is solid and is performing the intended function with no loss in capacity. Faying surface contact (0.01 in) is >75%. DTI installation verification yields appropriate refusals per structural bolt connection.
- Condition State 3: Moderate damage, corrosion or misalignment is present, but does not warrant structural review. Faying surface contact (0.01 in) is between 50~75%. Connection is performing the intended function with no loss in capacity. DTI installation verification yields improper number of gap refusals per bolt on 5-20% DTIs of each connection.
- Condition State 4: Heavy damage or corrosion of elements with section loss. Cracks may be present that are active. Faying surface contact (0.01 in) is less than 50%. DTI installation verification yields improper number of gap refusals per bolt on >20% DTIs of each connection. Elements may be misaligned or have severe impact damage that warrants structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.2.3.7 Connection(s) - Splices for Columns, Chords, Arms (Element 8707)

Longer sign structures require several sections to be joined to span between supports. This joining of separate sections, which can be up to 40 feet long each, is accomplished through chord splices. Splice flanges, approximately twice the diameter of the chord members, are welded to the chords at the end of each of the frame sections. The sections are field bolted together at these splice flanges. To provide camber in the structure, filler plates are often used in the upper chord connections. The inspector should verify a camber or level chord system across the span. Splice flanges are the typical splice connection for all Wisconsin full span overhead sign structures. Refer to Figure 4.2.3-37 for an example of a splice in a four chord overhead sign structure.

Column splices may be present on taller two post supports of four-chord or tri-chord structures. The splice flange shall be inspected in the same manner as a chord splice with the inspector verifying bolt snugness, weld condition and flushness of the flanges.

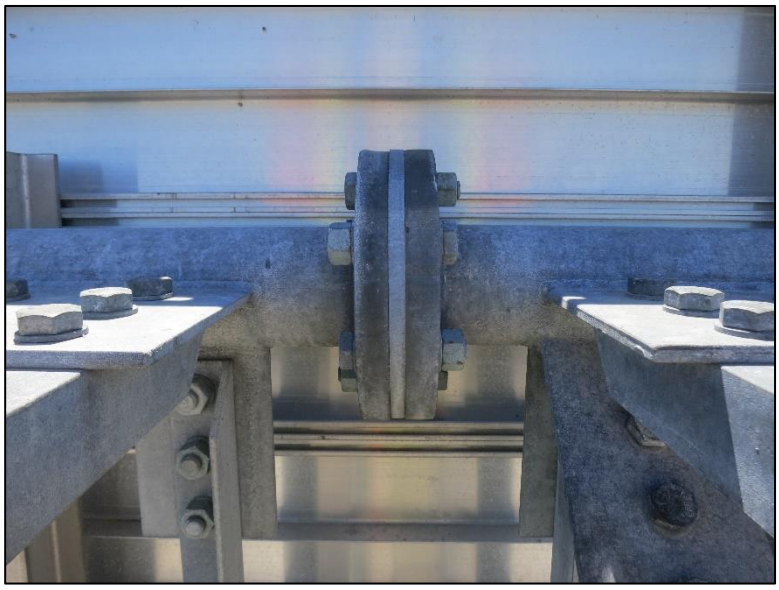

Figure 4.2.3-37: Four-Chord Truss Chord Splice with Filler Plate.

Mast arms over multiple lane roadways may require additional length. Due to fabrication length restrictions prohibiting the pipe to be constructed in one solid piece, an extension is provided that slides over the end of the mast arm. To prevent the extension from sliding off the end, a through bolt is installed through the overlapping joint. The inspector should verify slip joint bolt nut snugness and inspect the exposed seam weld of the extension pipe. Refer to Figure 4.2.3-38 and Figure 4.2.3-39 for a view of a slip joint and slip joint through bolt, respectively.

Figure 4.2.3-38: Slip Joint Extending Length of Mast Arm.

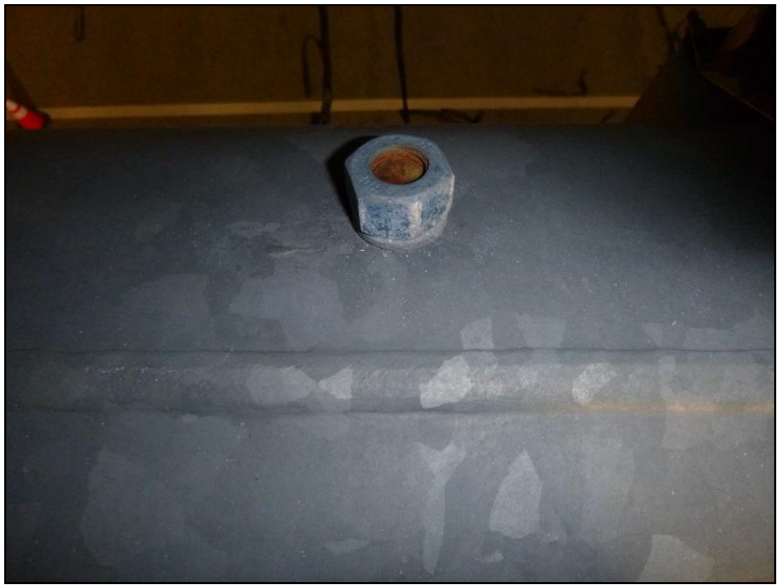

Figure 4.2.3-39: Slip Joint Through Bolt Nut Not Fully Engaged.

This element defines the splice(s) used to connect members together. Quantity is one each per connection location. The inspector shall pay close attention to proper bolt tightening, the use of filler plates between splice flanges (if present) and the circumferential welds between the chords and flange. The inspector shall note any looseness of extensions in slip joint connections.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect.

The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element shows no deterioration.

- Condition State 2: Minor corrosion is present, but with no discernable section loss. Superficial damage to the element may exist, as may minor misalignments. The connection is solid and is performing the intended function with no loss in capacity. Less than 5% of the connection Bolts/fasteners are loose or missing. Faying surface contact (0.01 in) is >75%. DTI installation verification yields appropriate refusals per structural bolt connection.

- Condition State 3: Moderate damage, corrosion or misalignment is present, but does not warrant structural review. Faying surface contact (0.01 in) is between 50~75%. 5~20% of the connection bolts/fasteners may be loose or missing, though the connection is performing the intended function. DTI installation verification yields improper number of gap refusals per bolt on 5-20% DTIs of each connection.

- Condition State 4: Heavy damage or corrosion of elements with section loss. Cracks may be present that are active. Faying surface contact (0.01 in) is less than 50%. Greater than 20% of the connection bolts/fasteners are loose or missing. DTI installation verification yields improper number of gap refusals per bolt on >20% DTIs of each connection. Elements may be misaligned or have severe impact damage that warrants structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.2.3.8 Overhead Sign Panels (Element 8708)

Reflective sign panels are either connected to catwalk L-brackets, or to sign panel hanger wind beams (vertical sign supports) with sign panel clips/connectors. Refer to Figure 4.2.3-40 and Figure 4.2.3-41 for a typical reflective sign panels. There are two categories of sign panels: Type I and Type II. Type I signs are comprised of extruded aluminum panels and are typically connected to the superstructure by vertical windbeams and sign panel connectors (discussed under element 4.2.3.10 Connection(s) - Overhead Sign Panel / Signal Heads (Element 8710)). Type II signs are smaller in size, comprised of plywood or flat sheet aluminum and typically connected to overhead sign supports by steel straps or brackets.

Sign panels should be visually inspected for peeling or delamination of the sign letters or sign backgrounds; loose sign edging; missing signs; and sign vandalism. When signs with defects are encountered, the deficient sign panel should be photographed or the message sketched with the location(s) of the defect(s) noted. Refer to Figure 4.2.3-42 through Figure 4.2.3-44 for typical sign defects.

Figure 4.2.3-40: Typical Type I Reflective Sign Panel.

Figure 4.2.3-41: Typical Type II Reflective Sign Panel.

Figure 4.2.3-42: Damaged Sign Panel with Reflective Coating Beginning to Tear and Peel (Condition State 2)

Figure 4.2.3-43: Damaged Sign Panel with Reflective Coating Missing and Fading (Condition State 3).

Figure 4.2.3-44: Sign Surface Cracked and Fading (Condition State 3)

Dynamic Message Signs (DMS) are located throughout major routes where heavy traffic and congestion are present. These signs are controlled from the State Traffic Operations Center (STOC). The messages displayed are typically drive time and accident reports. These messages are useful to the traveling public to warn them about delays along their path of travel. DMS's can be mounted to four-chord full span, or cantilever structures. The inspector shall verify the condition of the displayed message and note any missing burnt out bulbs or discrepancies in the display. The message should be clearly legible to all on-coming traffic. Refer to Figure 4.2.3-45 and Figure 4.2.3-46 for a typical DMS sign.

Figure 4.2.3-45: Typical Dynamic Message Sign.

Figure 4.2.3-46: Typical Dynamic Message Sign, Rear View.

This defines the overhead sign panel(s) of a sign structure. The rating shall include the legibility as well as condition of the panel(s). Quantity is one each per sign. The inspector shall indicate dimension(s) of the signs in notes for this element. All sign types installed along the horizontal structure members shall be evaluated under this element.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element(s) are present and show no deterioration. The sign is legible and up to current standards.
- Condition State 2: Minor loss of element legibility due to dulled paint or reflectorization. Minor deterioration may be present.
- Condition State 3: Moderate corrosion or damage may exist but not significantly affecting element legibility, nor stability.
- Condition State 4: Signs are illegible, or there is significant deterioration or damage to the sign panel that needs attention immediately.

4.2.3.9 Signal Head(s) (Element 8709)

Signal heads connected to the mast arm shall be inspected by the inspector to verify the signals are operating and functioning as intended. Typically the bottoms of signal heads on mast arms will be the points of lowest vertical clearance. Consequently, signal heads should

be inspected for any indications of vehicular impact. The connection brackets attaching the signal heads to the mast arms shall be evaluated under the following element 4.2.3.10 Connection(s) - Overhead Sign Panel / Signal Heads (Element 8710).

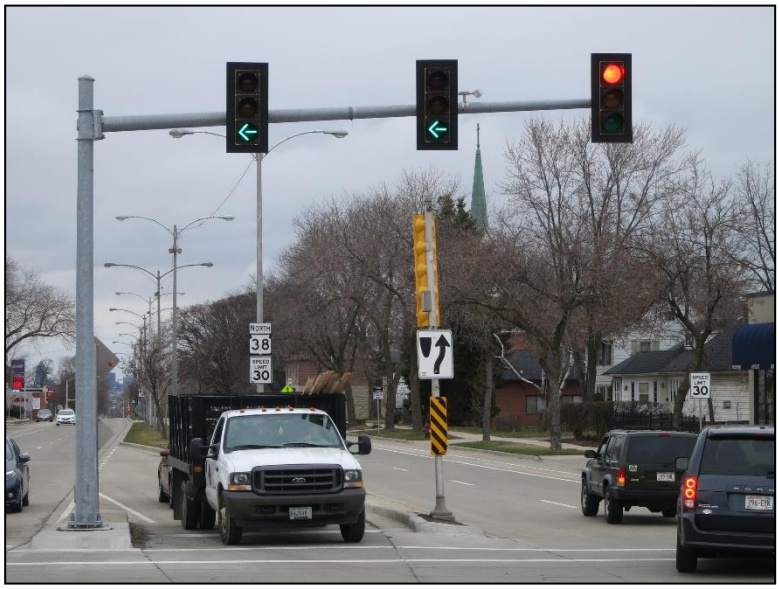

Figure 4.2.3-47: Signal Mast Arm with Three Properly Functioning Signal Heads.

This defines the signal heads on a structure. Quantity is one each per signal head.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element(s) are present and show no deterioration.
- Condition State 2: Minor deterioration and/or damage is present, but signal is still performing the intended function with no apparent loss in visibility.
- Condition State 3: Moderate deterioration or damage may be present. Signal head orientation is not in correct and should be adjusted. All bulbs appear to be functioning as intended.
- Condition State 4: Signal bulbs may be burned out, or orientation does not allow for signal to be used by traffic. There is sufficient damage to warrant analysis ASAP by Traffic Engineers.

4.2.3.10 Connection(s) - Overhead Sign Panel / Signal Heads (Element 8710)

Reflective sign panels are secured to windbeams or vertical hangers by sign panel connectors, commonly referred to as “SPCs” in the field. All sign panel connectors are to be inspected for deteriorated, missing or loose sign panel connectors. Note the number of deficient sign panel connectors and the total number of sign panel connectors under the element notes. The inspector should check by hand all SPCs for snugness. When loose connectors are encountered, the inspector should make all attempts to tighten the connector with a tied off wrench.

Dissimilar metal corrosion has been found on some sign panel connectors and bolts. Many aluminum sign panel connectors have had spilt nuts and broken studs. Each connector should be checked to ensure the stud is not broken (by trying to move the sign panel connector by hand). Current standards will have stainless steel studs and nuts. Furthermore, current design standards (state plate A4-6) state that when Type I signs are mounted over live traffic, sign panel connectors shall be installed on each side of the vertical windbeam on each extruded aluminum rib. Refer to Figure 4.2.3-48 for missing sign panel connectors on a Type I sign.

For butterfly structures, vertical sign mounting I-beams are U-bolted to the horizontal tubes to support the sign panel.

VMS’s are typically mounted to the space frame with U-bolts. Due to the weight of the unit, numerous U-bolts are used, and all are essential for proper mounting. Care should be taken to ensure that all U-bolts are installed with the nuts fully engaged and tight.

On older structures, catwalks may be present. The catwalks are secured to L-brackets which are fastened to the space frame of the sign structure typically with U-bolts.

Type II signs and signal heads are connected to mast arms with a variety of connections. Commonly, aluminum castings are strapped to the arm via stainless steel bands. Stainless steel U-bolts are found on smaller structures with thinner mast arms. When loose, fractured, or corroded connectors are encountered, the inspector should make notes within the report and attempt to tighten the connector if possible, also noting results within the report.

Monotube sign structures commonly have signal heads connected to the horizontal member via clamping connections. As monotubes tend to be more flexible when wind loaded, these connections may experience more intense cyclical loading and are commonly found to have fractured or failed connection components. Care should be taken to look closely at these overhead connections and as failures may result in the signal head falling onto traffic below. Refer to Figure 4.2.3-49 and Figure 4.2.3-50 through Figure 4.2.3-52 for examples of signal heads with deficient or failed connections.

This element defines the L-brackets, vertical hangers, horizontal braces, U-bolts, sign panel connectors, sign connection clamps, banding, and other structural members that mount the sign panels or signal heads to the structure. Quantity is one per vertical connection (e.g. Windbeam, Hanger, L-bracket, etc.)

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect.

The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element(s) are present and show no deterioration.
- Condition State 2: Minor deterioration and/or minimal loose connections are present.
- Condition State 3: Moderate deterioration or damage may be present. Multiple connection components may not be fully functioning. Multiple loose/missing sign clips that could significantly affect the strength and/or serviceability.
- Condition State 4: Connection components may be cracked, sheared or missing. There is sufficient concern to warrant structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

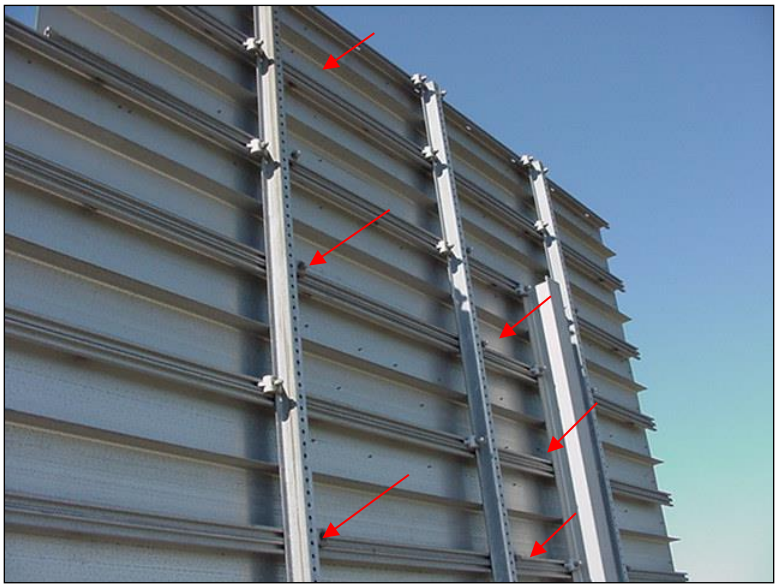

Figure 4.2.3-48: Missing Sign Panel Connectors (Condition State 4)

Figure 4.2.3-49: Monotube Sign Structure with Signal Heads Attached to Horizontal and Vertical Members.

Figure 4.2.3-50: Failed Bracket to Vertical Post Connection on Monotube Structure (Condition State 4)

Figure 4.2.3-51: Cracked Aluminum Signal Bracket (Condition State 4)

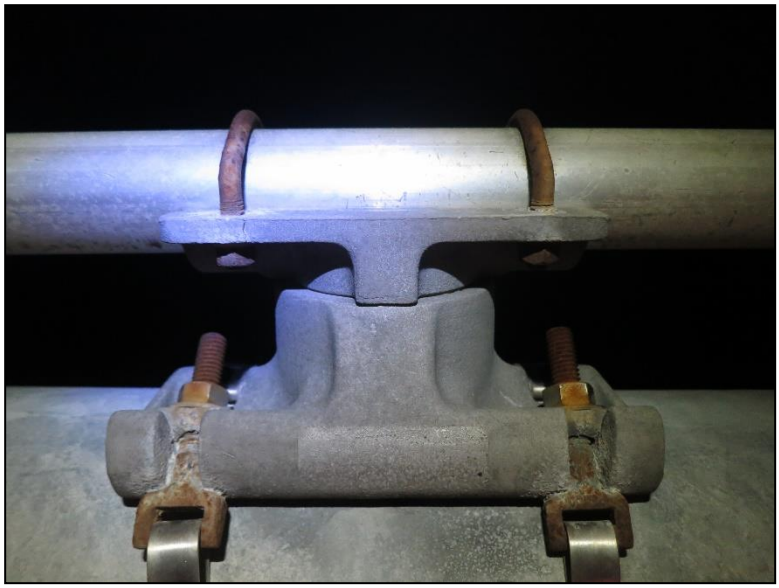

Figure 4.2.3-52: Moderately Corroded Bracket Bolts and U-Bolts (Condition State 2)

4.2.3.11 Luminarie Arm, Head, and Connections (Element 8711)

Luminaire arms, heads and connections are typically found on Mast Arm Structures

Inspection of the luminaire arm, head and connections should include a cursory inspection of the hand holes, if present, the material condition of the components and checking the snugness of all connections. Refer to Figure 4.2.3-53 for a luminaire base connection.

Figure 4.2.3-53: Signal Mast Arm with Luminaire Post Anchored to Top of Post. All of Luminaire is Inspected Under Element 8711.

This defines the connection, arm, and luminaire head on some signal and sign structures. Quantity is one each per assembly.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element.

- Condition State 1: The element(s) are present and show no deterioration.
- Condition State 2: Minor deterioration and/or minor loose connections are present.
- Condition State 3: Moderate damage or corrosion is present, but does not warrant structural review. Cracks may be present, but have been arrested or are no longer active.
- Condition State 4: Connection components may be cracked, sheared or missing. There is sufficient concern to warrant structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.2.4 Assessments

To provide additional refinement to the sign structure inventory, WisDOT also requires the inclusion and evaluation of secondary members or those components that if failing, missing or damaged would not necessarily compromise the structural integrity of the structure. All

assessments observed on a sign structure shall be included, inspected and assigned an assessment state within the inspection report. All assessments are in units of each. Therefore the inspector shall evaluate each assessment as a whole taking into account the size of the particular assessment when determining the appropriate assessment state.

4.2.4.1 Aesthetic Treatment (Assessment 9010)

This assessment defines the conditions of the aesthetic coatings or treatments on a sign structure. Examples of aesthetic treatments include concrete staining, form liners and rustications. The inspector must be aware that this assessment is for the evaluation of the aesthetics on the structure but not the structural members; similar to the protective coating element.

Inspection of aesthetic treatments should include noting the type(s) of aesthetic treatments on a sign structure and locations of distress under the assessment notes. It is the inspector's task to examine each aesthetic treatment assessment and reasonably assign the most appropriate condition state to the whole assessment. Refer to Figure 4.2.4-1 for a concrete pedestal with multiple aesthetics.

This assessment defines the staining, form lining or other aesthetic treatments that may be found on a sign structure that is not part of the protective coating element for the superstructure elements. Quantity is one each per sign structure.

Figure 4.2.4-1: Concrete Pedestal Supporting Four-Chord Cantilever with Form Lined and Stained Concrete.

Refer to the following assessment states:

- Good: System is in good condition, with no notable issues.
- Fair: Aesthetic system is in fair condition, with some fading or discoloration. Minor issues.

- Poor: Aesthetic system is in poor condition, with fading or discoloration.
- Severe: Aesthetic system is in severe condition and is not functioning as intended.

4.2.4.2 Rodent Screen (Assessment 9200)

If electrical conduit or wires are present, the base plate must be wrapped with a rodent screen. The rodent screen should have no gaps between the base plate/screen interface, and the screen/concrete foundation interface. If the structure has electrical conduit or wires, and the base is missing the metal rodent screening, it should be noted in the inspection report. Refer to Figure 4.2.4-2 for an example of properly installed rodent screen.

Figure 4.2.4-2: Rodent Screen Around Base Plate of Sign Structure.

This assessment defines the presence and condition of the screen used to keep rodents from accessing the underside of the baseplate area.

Refer to the following assessment states:

- Good: Rodent screen is present and is in good condition.
- Fair: Rodent screen is present - it may have some damage or deterioration but it is performing the intended function.
- Poor: Rodent screen is present - it may have some damage or deterioration and is not performing the intended function.
- Severe: Rodent screen is absent on structures were it is required, or incorrectly installed where it does not function properly.

4.2.4.3 Miscellaneous Mounted Attachments (Assessment 9202)

This assessment defines the presence and condition of cameras, walk signals, miscellaneous signs, electrical boxes, traffic control counters, dampeners, etc. mounted to the structure, as well as connections for these items. Type II signs mounted to the vertical posts of the structure shall be evaluated under this assessment.

Inspection of VMS structures should include a cursory inspection of the electrical conduit throughout the structure. The conduit will be placed inside the space frame to house the electrical wires. All conduit should be securely fastened to the space frame members.

Current design practice is phasing out sign lighting due to the high reflectivity of the sign panels. However, several of the structures are equipped with luminaires to illuminate the sign panel(s). The inspection of the electrical components consists of a visual inspection of the luminaire fixture, junction boxes, and the conduits. The inspector should note any impact damage; non-working luminaires; broken or missing luminaire fixtures; broken or missing fixture covers; and rusted shut or broken cover latches. The electrical circuits for the lighting should be checked for exposed wiring; open electrical boxes; loose electrical boxes; loose, broken, or missing sections of conduit; and missing cover plates. The uni-strut should also be inspected for corrosion, broken connections to the L-brackets and overall strength to support the luminaires. Refer to Figure 4.2.4-3 for an example of a luminaire and Figure 4.2.4-4 for an example of a failed luminaire connection.

Refer to the following assessment states:

- Good: Components appear to be fully functioning.
- Fair: The element(s) exist and are performing the intended function. Minor corrosion or damage exists.
- Poor: Moderate damage or misalignment exists. The elements may be loose and need tightening.
- Severe: Element is missing, or is significantly damaged and not performing the intended function.

Figure 4.2.4-3: Typical Sign Structure Luminaire.

Figure 4.2.4-4: Failed Connections of a Luminaire to U-Bracket on a Butterfly Sign Structure (Condition State 4)

4.2.4.4 Handhole Covers and Caps (Assessment 9205)

All handhole covers and fasteners and column caps shall be inspected. Loose or corroded fasteners shall be noted under the assessment notes. Refer to Figure 4.2.4-5 and Figure 4.2.4-6 for an example of a handhole cover and column cap, respectively.

Figure 4.2.4-5: Handhole Cover Unsecured at Base of Signal Mast Arm Column (Condition State 3).

Figure 4.2.4-6: Column Cap for a Two-Chord Sign Structure.

This assessment defines handhole covers and caps for columns, chords, and mast-arms.

Refer to the following assessment states:

- Good: The element(s) are present and show no deterioration.
- Fair: The element(s) exist and are performing the intended function. Minor corrosion or damage exists.

- Poor: Moderate corrosion or damage exists. The element may be loose and need tightening.

- Severe: Element is missing, or is significantly damaged and not performing the intended function.

4.2.4.5 Catwalk/Handrail/Safety Chains (Assessment 9206)

Catwalks, handrails and safety chains are found only on larger older sign structures. This assessment will not be used on sign support structures.

Current design practice is phasing out the use of catwalks on structures; however, older structures still have catwalks in place. Catwalks should be visually inspected for collision impact damage; tripping hazards due to unequal grating elevations; missing or misaligned grating sections, and corroded connection bolts. Catwalks can usually be accessed from the supporting space frames; however, sometimes the sign extends beyond the end of the catwalk, making access difficult or impossible without the use of a bucket lift. Refer to Figure 4.2.4-7 and Figure 4.2.4-8 for views of a catwalk.

Handrails should be visually inspected for collision impact damage; gaps; missing hinge bolts and broken or misaligned locking pins. Handrails should be operated to ensure stability and proper operation. All handrails should always be equipped with safety end chains. Safety chains should be visually inspected for latches that are rusted shut; inaccessible safety chains (i.e. chains that are attached behind the signs); latches that cannot be opened; and chains that are corroded or missing.

This assessment defines the walkway gratings, handrails, safety chains, and connections on the structure.

Refer to the following assessment states:

- Good: The element shows no deterioration.

- Fair: Minor damage and/or deterioration may be observed. Handrails and locking pins may be misaligned. Safety chains may be deteriorated.

- Poor: Moderage deterioration and/or damage. Sections of gratings or handrails may be unstable, damaged or missing. Safety chains are missing, or inoperable. Handrails and locking pins may be inoperable.

- Severe: Significant damage or deterioration is visible that threatens separation of component from the structure. There is sufficient concern to warrant structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

Figure 4.2.4-7: Catwalk with Handrails in Collapsed Position.

Figure 4.2.4-8: Catwalk with Handrails in Upright Position.

4.2.4.6 Crash Protection (Guardrail, Barrier, etc.) (Assessment 9207)

Overhead sign structures are located throughout the state adjacent to heavily trafficked roadways. It is common for these structures' foundations and columns to be protected from errant vehicle impacts. The inspector shall visually inspect the condition and function of all crash protection adjacent to and protecting the structure. Sign structures located on bridge superstructures shall not include this assessment as the barrier is inspected during routine bridge inspections as its own element. Refer to Figure 4.2.4-9 for an example of a distressed crash protection system at the foundation of a sign structure.

Figure 4.2.4-9: Deteriorated Barrier Wall with Approach Guardrail Protecting the Foundation of a Butterfly Sign Structure (CS 2)

This assessment defines the state of devices used to protect the structure. Evaluate for a distance of no more than 50FT from the structure, or as deemed appropriate by the engineer.

Refer to the following assessment states:

- Good: The element shows no deterioration.
- Fair: Minor damage or deterioration is noted. Element is still performing its intended function.
- Poor: Minor damage or deterioration is noted. Element is still performing its intended function.
- Severe: Significant damage or deterioration is visible. There is sufficient concern to warrant structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.2.4.7 Structure ID Plaque (Assessment 9208)

Each structure should be marked with an Identification Number plaque. The plaque consists of an alphanumeric series, which identifies the type of structure, Region, and structure number. New structures and all lighting structures/appurtenances are also required to have an electrical circuit plaque. Refer to Part 1, Chapter 5 for an explanation of the numbering.

Figure 4.2.4-10: Structure ID Plaque on Column of Sign Mast Arm Structure.

This assessment defines the plaque used to identify the structure.

Refer to the following assessment states:

- Good: The element is present and show no deterioration.
- Fair: The element is present, but has minor deterioration.
- Poor: The element is present, though may not be installed in the correct location. The element may have moderate deterioration.
- Severe: The element is missing, illegible, or incorrect.

4.2.4.8 Protective Coating(s) (Galvanization, Paint, etc.) (Assessment 9209)

In general, overhead sign structures will be comprised of galvanized steel although painted coatings may also be observed in the field. Painted concrete columns supporting four-chord, cantilever and butterfly structures are not applicable under this assessment. Only protective coatings on metal components shall be coded and evaluated.

It is important for the inspector to evaluate the effectiveness of the coating. A loss of coating is, in effect, an indication of failure as it is no longer present and performing as intended. Therefore, it is the inspector's responsibility to evaluate the entire structure and apply the most appropriate element state to the condition of all protective coatings present. Refer to Figure 4.2.4-11 and Figure 4.2.4-12 for a view of galvanized steel protective coating. Refer to Figure 4.2.4-13 and Figure 4.2.4-14 for protective coatings on sign supports.

Figure 4.2.4-11: Galvanized Coating on a Steel Column of an Overhead Sign Structure. (Condition State 1).

Figure 4.2.4-12: Cracked, Chipped and Peeling Galvanized Protective Coating on a Four-Chord Upper Diagonal Member (Condition State 3)

Figure 4.2.4-13: Scraped Galvanized Coating at Base of Column Due to Impact. Bare Steel Exposed in Several Locations (Condition State 3).

Figure 4.2.4-14: Painted Coating on a Steel Signal Mast Arm Structure (Condition State 1).

This assessment defines the overall condition of the protective system(s) of the metal components of the structure.

Refer to the following assessment states:

- Good: The coating(s) are fully effective.
- Fair: The coating(s) are substantially effective, though small areas of peeling, dulling, bubbling or cracking of the coating may be present.

- Poor: The coating(s) have limited effectiveness and need touch-up work.
- Severe: The coating system has failed, and provides no protection to the underlying metal for a majority of the structure.

4.2.5 Recommended Inspection Procedures

The following general sequence can be used as a guideline for Traffic Operations Support System inspections:

1. Arrive at site and implement traffic control.
2. Identify structure number and record pertinent information.
3. Perform Inspection:
    - Base/Foundation; Grout pad; Anchor bolts, nuts and washers.
    - Base plate socket weld
    - Support pole or end frame.
    - Column to mast arm connection.
    - Repeat above two steps for other support, if applicable.
    - Span frame or member.
    - Sign(s)/Signal head(s) and connection members.
    - Catwalks, railings, and safety chains, if applicable.
    - Lighting, if applicable.
    - Obtain minimum vertical clearance measurement.
    - Obtain edge-of-lane to support distance and any other needed measurements.
    - Perform NDT on anchor bolts, if applicable.
    - Torque verification testing of anchor rod if not previously completed and noted within inspection report.
4. Review photographs and inspection notes to ensure completeness and correctness.
5. Remove traffic control and leave site.

For a full span sign structure, inspect both foundations prior to climbing. This procedure ensures that one is not forgotten after the structure is inspected. Climb and inspect the structure from left to right, this way the panel point numbers increase as the inspection

proceeds. The inspector can either inspect the entire span, come back to inspect the catwalks and signs; or inspect half of the span, inspect the catwalks, etc., and then complete the other half of the span.

For a cantilever structure or a tri-chord span with a single post support, a bucket lift shall be used by the inspector to inspect the post, then to lift the inspector up to the truss section. Once there, the inspector can exit the bucket to inspect the space frame span by climbing or remain in the bucket in the case of two-chord and monotube spans. The connections of the span to the support can be inspected by climbing or with the bucket lift. The support post for this type of structure should be checked for plumb alignment. Any leaning should be recorded for monitoring of this misalignment at the next inspection.

Monotube structures are inspected with a bucket lift with a lane closure and traffic control. The foundations are inspected similar to other full-span sign support structures.

A bucket lift shall be used by the inspector to inspect the post, then to lift the inspector up to the post to mast arm connection and each sign/signal head. The support post for this type of structure should be checked for plumb alignment. Any leaning should be recorded for monitoring at the next inspection.

Structure-mounted frames will normally be inspected with a bucket lift and traffic control with possible lane closures. It may be possible to climb down onto the frame behind the sign from the bridge upon which it is mounted. However, this is usually only possible if the bridge is at a skew, creating a larger space between the sign and the bridge.

There are so few members to inspect for these supports that a sequence is not necessary. The inspection should include all frame components; L-brackets, lighting and accessories, bracket connections to the bridge, anchor or connecting bolts, and all signs and attachment hardware.

All members and connections of structure-mounted frames should be inspected as described above for similar components. In addition, the anchorage to the concrete must be checked for corrosion, bolt pullout, and cracked concrete.

For structure-mounted frames, the L-brackets support the sign catwalk and lights (if present). The L-brackets are connected to their supports with U-bolts. These U-bolts should be checked for tightness and full engagement of the nuts.

4.2.6 Initial Inspections

Due to the large number of sign structures and supports being installed throughout the State, a thorough initial inspection shall be performed prior to the contractor leaving the site. WisDOT has mandated these inspections due to past poor construction practices on installation procedures. These third-party inspections assist in verifying the correctness of the construction installation. Furthermore, a contractor punchlist is assembled and delivered to the Project Manager. This forces the contractor to correct all deficient items prior to project closeout. Currently, initial inspections should take place before traffic is under the structure to ensure safety to the travelling public. If this is not feasible, the inspection should take place within two weeks of erection. Major construction projects will install structures during certain stages of a project and may not have the correct signage for the final roadway pattern. WisDOT requires

that these structures be inspected immediately after installation to ensure safety to the travelling public. Once the signs are replaced with the correct roadway pattern, the new sign connections must be inspected to ensure safety to the travelling public.

Many of the procedures for initial inspection are identical to the inspection procedures outlined in Section 4.2.5; however, special attention must be paid to the plan set, shop drawings, and design details. State details and specifications need to be checked for adequacy and correctness. In addition to the typical structural inspection, some of the details that the inspector should be cognizant of are as follows:

- Footing Elevation: Properly placed above the grade.
- Drainage: Water properly flows away from the foundation
  Water does not pond on the foundation/surrounding areas.
- Anchor Rods: Proper placement within foundation and installed plumb.
  Correctly tensioned.
- Stand-Off: Base plate is at proper elevation above foundation.
- Post Alignment: Erected plumb.
- Mast Arm Orientation: Installed with camber up.
- Truss Orientation: Installed per plan or shop drawing.
- Sign Connections: Proper material, amount, and spacing. Structural bolts are properly tensioned (using DTI's)
- ID Plaque Installed properly on structure, with correct ID noted.
- Rodent Screen For VMS and other structures with electrical connections, this must be installed.
- Vertical Clearance: Meets State minimum and maximum for the structure type.

Prior to inspection, certain WisDOT DT forms must be obtained by the inspection crew. These are as follows:

- DT2321: High Strength Steel Anchor Rod Installation Tensioning Record.
- DT2322: Ancillary Structures Pre-Installation Verification of High Strength Bolts

Other forms that should be completed by the contractor and available, but not required, for the inspector's information prior to the initial inspection:

- DT2113: Rotational Capacity Test/Record.
- DT2114: Pre-Installation Verification Test of High Strength Bolts.

These documents should be reviewed prior to the initial installation and attached to HSIS as a PDF attachment to the final report.

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4.3 GENERAL

4.3.1 Introduction

This chapter describes the high mast light towers used as roadway lighting. High mast lighting structures are generally large diameter post structures with typical heights of 100 to 150 feet supporting a movable ring with up to eight luminaires.

A comprehensive system of inventory and uniform inspection program has been implemented for high mast lighting structures. WisDOT currently inspects the high mast light towers at the recommended intervals outlined in Chapter 1 of Part 4. It is recommended that high mast light towers' mechanical and electrical systems be inspected by maintenance personnel during luminaire replacement. In addition to maintenance-related inspections, it is required that routine and detailed inspections be completed by qualified structure inspection Team Leaders in accordance with Figure 4.3.1-1. More frequent inspections may be warranted if the high mast light tower defects are known and it encroaches on the roadway, roadside, overhead power lines, publicly used areas, or private property. During the routine and maintenance-related inspections, all existing systems should be inventoried by element and assessment. During the installation of new high mast lighting structures, the resident engineer on all State roadway projects should complete an inventory of all installed items. Refer to Part 4, Appendix B, for the applicable inspection forms.

Inspection Type	High Mast Light Tower
Maintenance Related Inspection	1. Interval Based on Maintenance Interval 2. Full Luminaire Ring and Winch Inspection 3. Electrical and Mechanical Inspection
Routine Inspection	1. 48 Month Maximum Interval 2. Inventory All Structures 3. Foundation Inspection 4. NDT Anchor Rods 5. Visually Inspect Tower and Luminaire Ring for Defects using High Power Telescope
In-Depth Inspection	1. When Structural Capacity may be Compromised due to Defect 2. NDT of Post/Base Weld 3. Close Visual Inspection using Climbing Camera Device, Mounted Camera on Ring, Inspector Access with Specialized Equipment, or Temporary Lowering of Tower with Crane

Figure 4.3.1-1: Types and Levels of Effort Associated with the Inspection.

4.3.2 High Mast Lighting Inspection

The inspector must be familiar with all the available elements within the Highway Structures Information System (HSI) and appropriately capture them within the inspection report for each high mast lighting structure. Evaluating elements provides a more accurate picture of the structure being inspected and alerts the Department of potential future issues including traffic hazards or functionality of the high mast light. The major elements for inspection are the Foundation, Anchor Bolts, Steel Base Plate, and Steel Pole and Splices. In addition to inventorying all applicable elements and evaluating each element with the appropriate defects, the inspector must also report all appropriate assessments on the high mast light. Refer to Section 4.3.4 High Mast Lighting Assessments. Examples of assessments are the structural, electrical, and luminaire ring mechanical components, signs, and drainage.

Element Level Inspection

On the inspection report form, each high mast lighting element is recorded in units of Each. It is important to note that high mast lighting elements do not possess any material defects. Rather, each element condition state definition provides all the defect language within it. Therefore, the inspector is tasked with noting all defects observed during the inspection under the element notes and determining the appropriate element condition state. For an Each unit of measure, the coded Condition State applies to the entire element. In this manner, an Each unit of measure essentially acts as an overall rating for that element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

4.3.3 High Mast Lighting Elements

High mast light towers are tall, vertical poles that are rigidly anchored to their foundations. The light tower heights can range from 60 feet up to 175 feet, but generally fall between 100 feet and 150 feet. Refer to Figure 4.3.3-1 for an overall view of a high mast light tower. High mast light tower sections can be manufactured to a maximum length of about 55 feet. Splices between sections may either be butt-welded with a back-up bar, or overlap slip joints. Refer to Figure 4.3.3-2 for a close-up view of an overlap slip joint. The towers are typically tapered at a slope of 0.14 inch-per-foot, and are secured to a concrete foundation by anchor bolts with leveling nuts, top nuts, and secondary jam nuts.

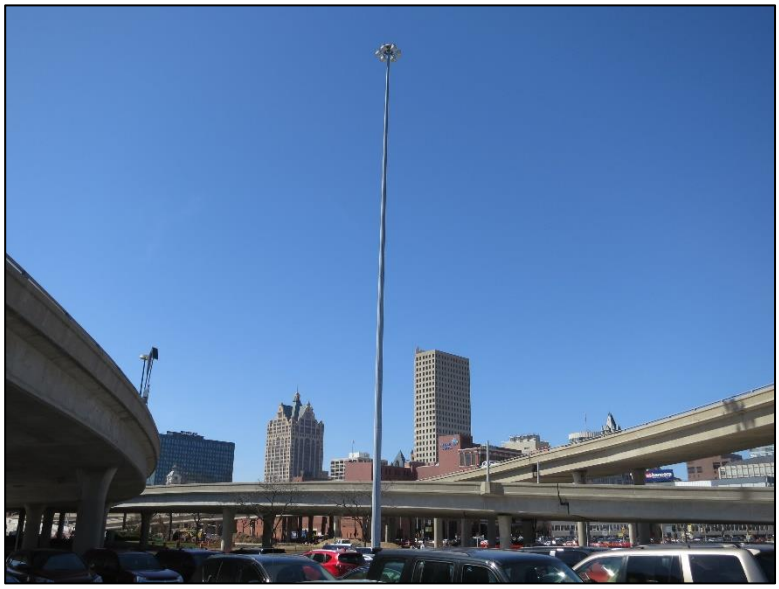

Figure 4.3.3-1: Overall View of a Typical High Mast Light Tower.

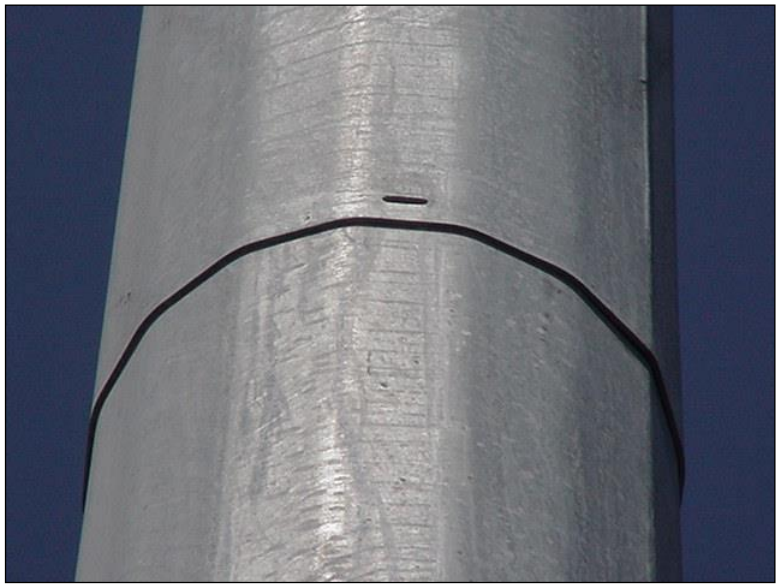

Figure 4.3.3-2: View of a High Mast Light Tower Overlap Slip Joint.

High mast light towers are typically fabricated with galvanized steel or weathering steel. All weathering steel support systems in Wisconsin are currently in the process of being phased out and replaced. Any high mast light towers fabricated of weathering steel should be subjected to more frequent inspections including the removal of handhole plates to examine internal surface corrosion.

Since high mast light towers are slender structures, they possess a low frequency of vibration. These structures are typically very sensitive to the dynamic nature of wind forces, resulting in

vortex shedding related vibrations. High mast light towers are susceptible to vortex shedding due to large bending moments at their base. The bending moments within the structure develop tensile and compressive stresses within the extreme material fibers. The effects of solar heating from the sun can cause additional bending in a tower with the sunlit side exhibiting tensile stress cracks. Since a high mast light tower is a single component structure, it does not possess load path redundancy, and should be considered a nonredundant steel tension member.

High mast light towers support a ring with several luminaires mounted to it which illuminate a large area. The luminaires are typically controlled by use of a photocell mounted on a nearby control panel. Each tower has either an internal or external electric winch that can be used to raise or lower the luminaire ring for routine maintenance.

4.3.3.1 Base / Foundation (Element 8701)

This element includes reinforced concrete foundations used for High Mast Lighting Structures, Overhead Signs, and Signal Monotubes. If a grout pad is present, it is also evaluated in this element. If Anchor Rod(s) were placed eccentrically, record the eccentricity in the notes with measurements for offset. The inspector should verify the eccentricity was reviewed and approved by the owner and that this information is stated within the element notes.

Reinforced concrete bases and foundations are made by placing ready-mix concrete into slip forms steel reinforcing throughout the foundation. The soil adhesion, skin friction, and bearing pressure resist loading from the structure.

Figure 4.3.3-3: View of a Delaminated/Spalled Concrete Foundation – Condition State 3.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect.

The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element, as well as protective systems.

- Condition State 1: The element shows no deterioration.
- Condition State 2: Minor cracks and spalls may be present in the foundation, but only minimal reinforcing steel is exposed. When efflorescence is present, it's minor and no evidence of rust staining. Grout pad (if present) is in good condition. Minor erosion around foundation may be present, but does not affect structural capacity.
- Condition State 3: Many Spalls are present. Corrosion of reinforcement and/or loss of concrete section is evident though not sufficient to warrant structural analysis. Grout Pad (if present) has moderate cracking, spalls, or delaminations. Erosion may be present that reduces the foundation embedment significantly but does not pose a threat to the stability of the structure.
- Condition State 4: The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

Figure 4.3.3-4: Exposed Foundation Due to Adjacent Construction Work. Condition State 3.

4.3.3.2 Steel Anchor Rods (Element 8702)

This assessment element defines anchor rods, anchor nuts, leveling nuts, and washers connecting the column support to the foundation. Steel anchor rods are embedded within the foundation, and transfer loads from the pole to the foundation. Leveling nuts are placed on the anchor rods and establish a level base for the pole base plate. The base plate is then secured to the anchor rod through tension in the rod supplied by the washer and anchor nut. A jam nut is often found on top of the top nut to prevent top nut from loosening.

Figure 4.3.3-5: View of Loosened Anchor Rods. Note No Jam Nuts Over Anchor Nuts. Condition State 4 (2 Each).

Element Condition States

Below defines the Condition States specific to each assessment element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element, as well as protective systems.

- Condition State 1: The element shows no deterioration

- Condition State 2: Minor corrosion of the element is present, but the bolts appear to be properly tensioned and the nuts are fully engaged. Anchor rods may be slightly out of plumb (1:40 or less) without need for remediation, or out of plumb between 1:20 and 1:40 with proper beveled washers, and PE Stamped calculations on file to certify the configuration is ok. Anchor rod standoff exceeds current standards, but is within acceptable limits based on calculations.

- Condition State 3: Moderate corrosion of the elements may be present, but not enough to warrant structural analysis. Lock washers may be present. Anchor rod standoff exceeds current standards, and there is no analysis on file and rods appear to be performing adequately.

- Condition State 4: Severe Corrosion is present. Anchor rods are not properly tensioned, nuts may be missing or not fully engaged. Anchor rods are greater than 1:40 out of plumb w/o remediation (Greater than 1:20 requires immediate replacement). Anchor rod standoff exceeds current standards, and rod(s) show signs of bending, movement, buckling, or elongation. The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review

has been completed and it has been found that the defects impact strength or serviceability.

4.3.3.3 Base Plate(s) (Aluminum or Steel) (Element 8703)

This assessment element defines the base plates, flanges, gusset plates, seam welds, and welds at the connection of the column support to the foundation. Base plates are typically welded to the pole, and secured to the anchor rods using high strength nuts and washers. Leveling nuts are used below the base plate to align the structure, and the anchor nuts are utilized to secure the plate to the leveling nuts. The base plate transfers loads from the pole to the anchor rods.

Figure 4.3.3-6: View of Weathering Steel Base Plate.

Element Condition States

Below defines the Condition States specific to each element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element, as well as protective systems.

- Condition State 1: The element shows no deterioration.
- Condition State 2: Minor corrosion of the element is present. No cracking of the element is observed. Casting clamp, if used, has no more than one horizontal bolt loose.
- Condition State 3: Moderate corrosion of the element is present. Cracks may be present on the base plate to column support connection weld, but have been arrested or is no longer active and do not affect the capacity of the plate. The base plate may

be distorted (dished). Casting clamp, if used, has no more than two horizontal bolts loose.

- Condition State 4: Severe corrosion is present. Cracks may be present on the base plate to column support connection weld that have not been arrested. Section loss is significant and may affect the ultimate strength or serviceability of the element. Three or more horizontal casting clamp bolts are loose or missing, or cracks exist in the casting clamp assembly. The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.3.3.4 Column Support(s) (Aluminum or Steel) (Element 8704)

This element includes the vertical pole, handhole covers, slip joints and circumferential welds for the column support on the structure. Poles are typically welded to the base plate. Poles greater than 50 feet in length are typically spliced together as separate pieces. Splices can be welded, but are most often slip joints, with an upper section fitting over the top of the section below it. Slip joints depend on gravity to prevent the sections from separating, and are often bolted to prevent rotation and secure the connection. The splice transfers loading between pole sections, and the base plate weld transfers loads to the base plate.

The inspector should pay close attention to the steel and weldments surrounding hand holes, stiffener plates and other connections near the base of the structure. These components are fatigue prone details and may develop cracking due to the cyclic wind loading. The upper pole at a lap splice should also be carefully inspected as the excessive movement at the splice may create failure in the weld of the pole.

Element Condition States

Below defines the Condition States specific to each assessment element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element, as well as protective systems.

- Condition State 1: The element shows no deterioration.

- Condition State 2: Minor corrosion is present. Standing water may be observed inside the post.

- Condition State 3: Moderate damage or corrosion is present, but does not warrant structural review. Cracks may be present on the pole, but have been arrested or is no longer active.

- Condition State 4: Heavy damage or corrosion of elements with section loss. Cracks may be present on the pole that have not been arrested. Elements may be misaligned or have severe impact damage that warrants structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

Figure 4.3.3-7: View of Minor Scraping Exhibited at the Bottom of the Vertical Pole. (Condition State 2).

4.3.3.5 Connection(s) - Splices for Columns, Chords, and Mast Arms (Element 8707)

This element defines the splice(s) used to connect members together. This includes the slip joint connections used in many pole applications. The inspector should include the first foot of the vertical pole above the base of the slip joint under this element. Beyond this extent, the pole should be evaluated under Element 8704. This element should not be used to code any defects with the weldments or stiffener plates at the pole/base plate connection. Defects found on these elements should be reported under the appropriate pole/base plate element. Element 8707 shall be used to evaluate the condition of the vertical pole splice connections.

The inspector should pay close attention to rust staining around the base of the slip joint or splice or the buildup of pack rust between the members. This may be evidence of additional distress at the joint. In-depth means of inspection may be warranted in some instances.

Element Condition States

Below defines the Condition States specific to each assessment element. The defects are expounded on and critical areas are discussed to aid the inspector in determining the severity of a defect. The Condition States are comprised of general descriptions and uniquely colored to follow the severity the description represents. Condition states consider material and structural defects of the element, as well as protective systems.

- Condition State 1: The element shows no deterioration

- Condition State 2: Minor corrosion is present, but with no discernable section loss. Superficial damage to the element may exist, as may minor misalignments. The connection is solid and is performing the intended function with no loss in capacity. Less than 5% of the connection Bolts/fasteners are loose or missing. Faying surface contact (0.01 in) is >75%.

- Condition State 3: Moderate damage, corrosion or misalignment is present, but does not warrant structural review. Faying surface contact (0.01 in) is between 50~75%. 5~20% of the connection bolts/fasteners may be loose or missing, though the connection is performing the intended function.

- Condition State 4: Heavy damage or corrosion of elements with section loss. Cracks may be present that are active. Faying surface contact (0.01 in) is less than 50%. Greater than 20% of the connection bolts/fasteners are loose or missing. Elements may be misaligned or have severe impact damage that warrants structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.3.4 High Mast Lighting Assessments

Assessments allow for the evaluation of secondary components not necessarily impacting the structural integrity of the structure but those components that are necessary for the proper function of the structure. The inspector must be familiar with all the available assessments within the HSI system that are permitted for each structure type and appropriately capture them within the inspection report for each high mast lighting structure.

Evaluating assessments provides a more accurate picture of the structure being inspected and alerts the Department of potential future issues including traffic hazards or functionality of the lighting structure. The following section describes all the available assessments for high mast lighting that may be reported within HSI.

Assessments are all measured in units of Each, regardless of size. Therefore, it is the inspector's duty to gauge the overall deterioration of the assessment and record the appropriate assessment state.

4.3.4.1 Rodent Screen (9200)

This assessment defines the presence and condition of the screen used to keep rodents from accessing the underside of the baseplate area.

Figure 4.3.4-1: View of Rodent Screen between Base Plate and Foundation.

Refer to the following assessment states:

- Good: Rodent screen is present and is in good condition.
- Fair: Rodent screen is present - it may have some damage or deterioration but it is performing the intended function.
- Poor: Rodent screen is present - it may have some damage or deterioration and is not performing the intended function.
- Severe: Rodent screen is absent on structures were it is required, or incorrectly installed where it does not function properly.

4.3.4.2 Electrical, Luminaire Device and/or Cameras (9201)

This assessment defines the visual condition of luminaries and/or cameras in the system on the structure, as well as the visual condition of the electrical boxes. A separate electrical inspection covers the electrical systems in detail.

Typically the ring will not be operational during a routine inspection by the inspector. Therefore, the inspector must bring the appropriate tools to best evaluate the overall condition of the device. Binoculars or scope are often used for inspection purposes.

Figure 4.3.4-2: View of Luminaire Ring.

Refer to the following assessment states:

- Good: Components appear to be fully functioning.
- Fair: Minor damage or misalignment is noted. Light cover latches are broken.
- Poor: Broken or missing lenses. Bulbs are burnt out or missing. Camera (if present) does not appear to be functioning.
- Severe: Significant damage or deterioration is visible that threatens separation of component from the pole structure.

4.3.4.3 Winch and Cables (9204)

This assessment defines the visual condition of the winch and cable system used to hoist and suspend the luminaire in HML structures. This includes the support brackets, housing anchorage, and cable attachments. A separate electrical inspection covers the motor and mechanical system used to raise and lower the luminaire.

Typically the inspector will not be able to verify the winch system is functioning. However, the inspector should attempt to inspect all attached cable and fasteners for any deterioration or distress. Commonly rodents will build nests on the winch system. This may be an indication the rodent screen is missing or not properly fastened to the base plate.

Figure 4.3.4-3: View of an Internal Winch System. Note Nesting Material Located Between Spools.

Refer to the following assessment states:

- Good: Elements appear to be fully functioning.
- Fair: Bolts may be loose or missing. Elements may be out of alignment.
- Poor: Housing may be bent or cracked. Cable anchorages may be loose, damaged or missing. Cables may be kinked, tangled, fraying, or damaged.
- Severe: The element is damaged or deteriorated to the point that failure is eminent.

4.3.4.4 Handhole Covers and Caps (9205)

This assessment defines handhole covers and caps for columns, chords, and mast-arms.

The inspector shall remove the fasteners and verify the cover or cap is functioning and securely fastened upon completion of the inspection of the interior.

Figure 4.3.4-4: View of a Handhole Cover.

Refer to the following assessment states:

- Good: The element(s) are present and show no deterioration.
- Fair: The element(s) exist and are performing the intended function. Minor corrosion or damage exists.
- Poor: Moderate corrosion or damage exists. The element may be loose and need tightening.
- Severe: Element is missing, or is significantly damaged and not performing the intended function.

4.3.4.5 Crash Protection Systems (Guardrail, attenuator, barrier) (9207)

This assessment defines the state of devices used to protect the structure. Evaluate for a distance of no more than 50' from the structure, or as deemed appropriate by the engineer.

Figure 4.3.4-5: View of Barrier Wall Acting as Crash Protection System for Structure.

Refer to the following assessment states:

- Good: The element shows no deterioration.
- Fair: Minor damage or deterioration is noted. Element is still performing its intended function.
- Poor: Minor damage or deterioration is noted. Element is still performing its intended function.
- Severe: Significant damage or deterioration is visible. There is sufficient concern to warrant structural analysis to ascertain the impact on strength or serviceability, or a review has been completed and it has been found that the defects impact strength or serviceability.

4.3.4.6 Structure ID Plaque (9208)

This assessment defines the plaque used to identify the structure.

The ID plaque should be installed on the vertical pole at the base and clearly visible.

Figure 4.3.4-6: View of a Structure ID Plaque.

Refer to the following assessment states:

- Good: The element is present and show no deterioration.
- Fair: The element is present, but has minor deterioration.
- Poor: The element is present, though may not be installed in the correct location. The element may have moderate deterioration.
- Severe: The element is missing, illegible, or incorrect.

4.3.4.7 Protective Coating(s) (Galvanization, Paint, etc.) (9209)

This assessment defines the overall condition of the protective system(s) of the metal component of the structure.

Corrosion or other deterioration of the base material should not be considered under this assessment. Assessment 9209 shall only include the effectiveness of the coating itself. Areas of missing coatings are considered fully ineffective or failed. However, it is the inspector's duty to apply an overall evaluation to the assessment not simply rate those few localized areas of failed coating, if present.

Refer to the following assessment states:

- Good: The coating(s) are fully effective.
- Fair: The coating(s) are substantially effective, though small areas of peeling, dulling, bubbling or cracking of the coating may be present.
- Poor: The coating(s) have limited effectiveness and need touch-up work.

- Severe: The coating system has failed, and provides no protection to the underlying metal for a majority of the structure.

4.3.5 Recommended Inspection Procedures

High mast lights are difficult to inspect because they are too high for lift trucks, and most are inaccessible to vehicles because of ditches or railings. The following checklist of items comprises the inspection process for High Mast Lighting.

1. Arrive at site and set-up traffic control (if required).

2. Identify Structure Number (Note location within Structure Specific Notes)

3. Measure and document the distance(s) to the nearest paved roadway, public facility, private property or overhead power lines.

4. Perform Inspection. Provide photographs and/or sketches for Condition State 3 and 4 defects.

    - Verify anchor rod diameter and length, then scan for defects.

    - Examine anchor rods for tightness and embedment using a standard inspection hammer. Inspect visually for corrosion, section loss, and plumbness.

    - Examine anchor rods for any eccentricity. Verify element notes indicate eccentricity was noted and approved by Owner or Owner's representative. Note any noticeable eccentricity measurements within the element notes.

    - Note height of base plate above top of foundation.

    - Examine rodent screen visually for holes, connection, and corrosion.

    - Examine the foundation visually and by sounding with a standard inspection hammer.

    - Inspect ground line for any material washing out around foundation.

    - Examine inside of pole for corrosion, section loss, cracking, water accumulation, and/or pack rust by removing handhole cover. Utilize a light inside cover. Note and remove any nesting debris.

    - Inspect the welds around hatches and handholes and all transverse welds near the base of the high mast pole for potential cracking.

    - Perform ultrasonic testing of anchor rods to note any breaks and verify lengths. Perform ultrasonic testing of base plate/post connection during in-depth inspections. Magnetic particle testing may be used as determined by the inspector.

- Examine the pole using visual methods. The most common approach would be to use a 25X scope, triangulated from 3 sides, for deterioration of the material, such as cracking, corrosion, and/or pack rust, noting the width, length, depth, and/or orientation of the deterioration. Pay specific attention to splice connections. (Light ring cameras and drones have also been used for pole inspections, and are an acceptable alternative. See Figure 4.3.5-1).

- Examine light ring (likely by using a 25X scope) for loose hardware, cracked lenses, damaged housing, and levelness of ring.

- Note previous inspection interval and recommend inspection interval.

5. Review of inspection notes to ensure completeness and correctness.

6. Remove any traffic control.

If severe deterioration is noted or suspected during the maintenance related or routine inspections, a need for an in-depth inspection may be warranted. A detailed inspection will be scheduled on an as-needed basis.

Figure 4.3.5-1: View of a Magnetic Climbing Camera Unit Used During Close Visual Inspection.

Figure 4.3.5-2: View of a Typical Luminaire Ring, Lowered for Maintenance.

Figure 4.3.5-3: View of the Hatch Opening and Internal Winch System with a Portable Motor.

Figure 4.3.5-4: View of a Misaligned Luminaire Ring.

Refer to Figure 4.3.5-5 for a close-up view of the hatch doors on an old and a new high mast light tower. There are differences in the door design on the older structures as shown on the left compared to the new structures as shown on the right. The new door is secured with bolts, a lock in easily accessible areas such as Park and Rides, and no longer has a latch. Note that the electrical circuit plaque has not yet been installed on the new structure.

Figure 4.3.5-5: View of the Hatch Doors for Old and New High Mast Light Towers.

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4.4 RETAINING WALLS

4.4.1 Introduction

A retaining wall structure is designed to maintain an abrupt difference in ground surface elevations. These structures support near vertical slopes of soil masses by preventing the soil from sliding and deep shear failure. These structures also provide a means of protecting an earth embankment from erosion, which is common on steep slopes. For instance, a slope can be completely eliminated or constructed at a naturally stable angle by raising the slope's toe elevation with a wall. A steeper slope can be constructed by utilizing soil reinforcement methods or a series of tiered shorter landscape walls.

Roadway proximity to soil cuts and fills necessitate the use of a retaining structure. Typical applications of these structures consist of retaining walls as part of a roadway system, dockwalls along a body of water, and extended wingwalls from a bridge abutment. This chapter contains a comprehensive discussion of the different types of retaining wall structures, types of failure, and element level inspection procedures.

Most newly constructed walls will have an "R" number assigned to it. Refer to Chapter 14 of the Wisconsin Bridge Manual (WBM) for numbering criteria. The ID plaque is generally located at the end of the wall; however, older walls may not necessarily have been assigned numbers and/or may have ID plaques missing. WisDOT inventories all "R" numbered structures within HSI with inspection intervals not to exceed 6 calendar years. This interval may shorten depending on several risk based factors including the overall rating, specific wall type, wall location, defect type, quantity of defect and/or manager preference. Refer to the Structures Inspection Manual (SIM) Part 4 Chapter 1 for more information on interval requirements and recommendations.

Non-integral wingwalls without "R" numbers are evaluated as retaining walls on the bridge inspection report. Therefore, retaining wall elements and appropriate material defects will be used for these wingwalls. Non-integral wingwalls are separated from the abutment footing with a full height joint. In some instances, typically older structures, the bridge plans may need to be reviewed to determine whether a wingwall is integral or not. A wingwall essentially acts as a free standing structure retaining fill. Non-Integral wingwalls with "R" numbers will not be evaluated as part of the bridge inspection but will be inspected as a separate inspection and interval under the inspection program. During bridge inspections, however, it is the inspector's due diligence to perform a cursory inspection of an adjacent "R" numbered retaining wall to ensure that any notable deterioration of the wall does not threaten the integrity of the bridge structure, or endanger the travelling public. Any areas of concern can be captured as maintenance items or through documented communication with the inspection program manager.

4.4.1.1 Wall Types

Retaining walls can be divided into many categories. For inspection purposes, we'll briefly define the common wall types found in Wisconsin in this chapter and refer to Chapter 14 of the WBM for a more in-depth discussion.

Walls can be constructed either bottom-up or top-down. Examples of bottom-up constructed walls include CIP Cantilever, MSE, and modular block walls. If a wall is constructed downward,

from the top of the wall to the bottom, it is a considered a top-down type of wall. Examples include soldier pile and sheet pile walls.

Wall Category	Wall Sub-Category	Typical Construction	Wall Type	Element #
Gravity	Mass Gravity	Bottom Up (Fill)	CIP Concrete Gravity	8600
	Semi-Gravity		CIP Concrete Cantilever	8600
	Reinforced Earth		MSE - Precast Panels	8603
			MSE - Modular Blocks	8603
			MSE - Wire Face	8603
	Modular Gravity		Modular Block	8602
			Gabion	8601
	In-Situ Reinforced	Top Down (Cut)	Soil Nailing	8606
Non - Gravity	Cantilever	Both	Sheet Pile	8605
			Soldier Pile	8604
			Secant/Tangent	8607
Non - Gravity	Anchored	Top Down (Cut)	Sheet Pile	8605
			Soldier Pile	8604
			Secant/Tangent	8607

Table 4.4 1 Wall Categories and Elements

Another category indicates the primary methodology that the wall uses to retain the fill material. Gravity walls are considered externally stabilized as these walls use self-weight to resist lateral pressures. There are numerous sub-categories for this type of wall including mass gravity, semi-gravity, modular gravity, mechanically stabilized reinforced earth (MSE), and in-situ reinforced earth (soil nailing) (see Figure 4.4.1-1). Non-gravity wall are classified into cantilever and anchored wall categories (see Figure 4.4.1-2). These walls are considered externally stabilized and are generally used in cut situations. Typical sub-categories for these walls include sheet pile, soldier pile, and tangent/ secant pile types, either with or without anchors. See WBM Chapter 14 for more information.

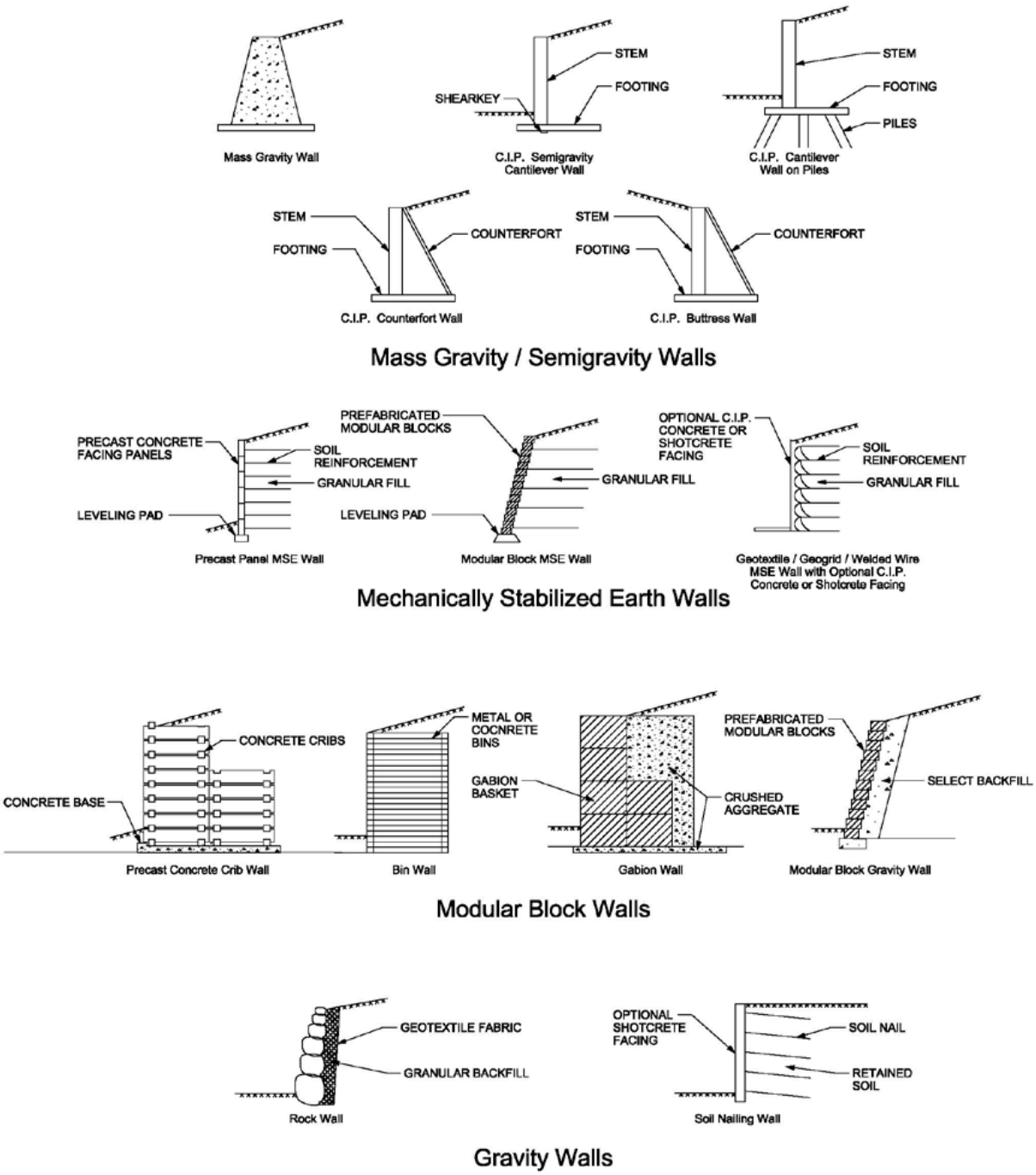
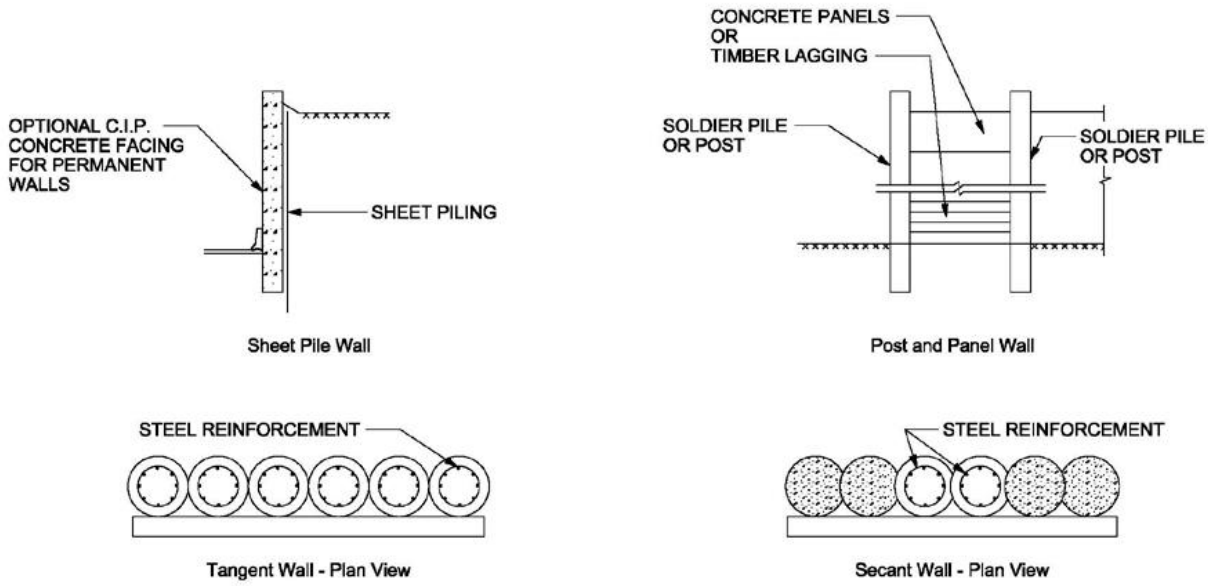


Figure 4.4.1-1: Typical Gravity Walls



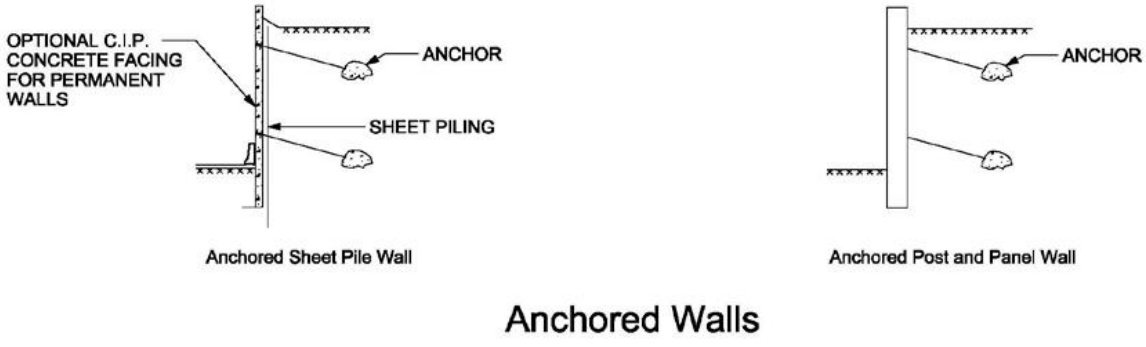


Figure 4.4.1-2: Typical Non-Gravity Walls

4.4.1.2 Wall Classifications

Retaining walls can be divided into three categories of applications: Fill-Section; Cut-Section; and Dockwall. The inspector should identify, if possible, which category of retention structure is present in order to better understand the significance of distress or localized failures. Each wall report within HSI designates whether a wall application is Fill, Cut or Dockwall. This information is located within the inspection report in HSI. Each wall application is elaborated as follows:

1. **Fill-Section Application:** The structure retains a roadway facility at a higher elevation than the adjacent earth. Specific areas of concern for inspection include heavier deterioration at the top and face of the wall from roadway salts washing over the wall, and geotechnical failure due to the added surcharge from traffic.

Figure 4.4.1-1: MSE “Fill” Wall

2. **Cut-Section Application:** The structure retains the adjacent earth at a higher elevation than the roadway facility. Specific areas of concern for inspection include heavier deterioration at the base of the wall from roadway salts washing against the wall, and vehicular impacts.

Figure 4.4.1-2: CIP “Cut” Wall

3. **Dockwall Application:** The structure retains the adjacent earth at a higher level than the waterway channel bottom. The deterioration mechanisms of a dockwall are the same as for a land-based earth retention structure; however, since a portion of the wall exists beneath the water the environment is more severe. This makes the wall more susceptible to scour and accelerated material deterioration. Also, dockwalls are typically associated with portions of the waterway where watercraft traffic and docking occurs, making the dockwall more susceptible to impact and ice flow damage. Underwater inspections are typically required to fully assess the condition of dockwalls. Refer to Part 1, Chapter 3, for a discussion of underwater inspection procedures.

Figure 4.4.1-3: Steel Sheetpile Dockwall

4.4.2 In-Service Retaining Wall Failures

Primary causes of in-service retaining wall failures include poor drainage, corrosion, facing deterioration, inadequate connection details, and latent construction defects. For MSE walls in particular, infiltration due to poor control of external drainage is the most common cause of failure.

Failure of wall construction material is frequently observed in older earth retention structures due to deterioration. Newer walls may exhibit structural material failure due to structural overstresses or poor material properties. Inadequate drainage behind the wall or an unexpected surcharge load can often cause material overstress.

Impact damage may also fail the material, and is typically a result of a collision between a moving object and the earth retention structure. The moving object is typically an errant vehicle or vessel, but may also be smaller objects hurled into the wall due to wind or explosion.

Impact damage is typically confined to an area directly adjacent to the point of contact and is characterized by distortion or crushing of the construction material at the point of contact with cracking, splitting, or splintering radiating from this point. Refer to Figure for a view of an impacted dockwall.

Figure 4.4.2-1: Impact Damage to a Steel Sheet Pile Dockwall.

While impact damage is typically expected above ground, utility operations and other similar activities may damage buried elements of an earth retention system. For instance, installation of utility poles behind a mechanically stabilized earth (MSE) wall often severs anchor strapping leading to wall movement. Therefore, it is important to note new construction on or adjacent to earth retention structures and record any distress, deterioration or movement in subsequent inspections.

During retaining wall inspections, geotechnical or structural defects found shall be quantified in their entirety and placed under the appropriate structural defect. While these material

defects may eventually compound or expand, structural defects are more global in nature with effects that may require more immediate attention. The following sections inform inspectors to the various material defects for each construction material used in earth retention structures.

4.4.2.1 Geotechnical Failure Modes

During the design phase, engineers check the wall for anticipated failure mechanisms relating to external stability, internal stability, movement and overall stability. The following figures give some examples of common geotechnical failure types (See WBM Chapter 14 for more information). Defects for wall movements used in the Element based inspection will be discussed in Section 4.4.3.4 and 4.4.3.6.

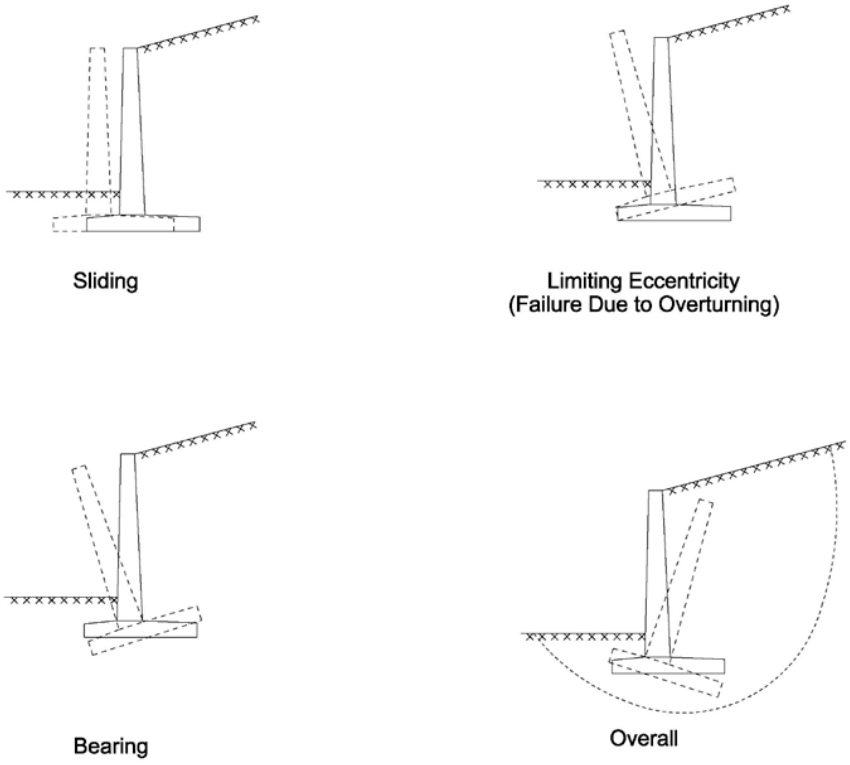


Figure 4.4.2-2: External Stability Failure of a CIP Semi-Gravity Wall

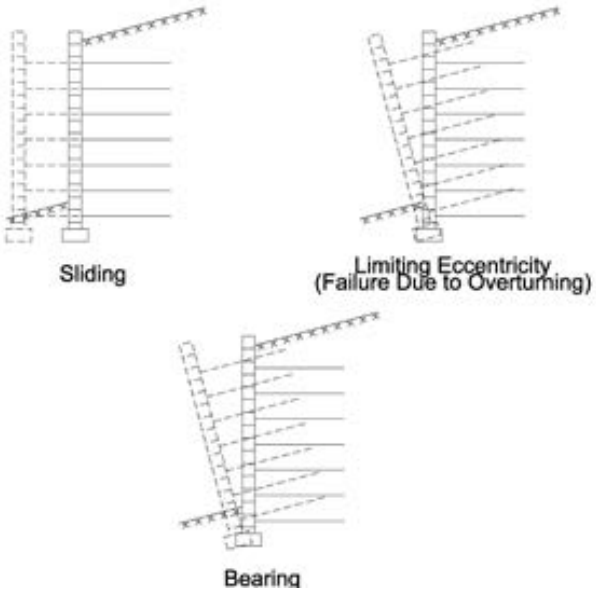


Figure 4.4.2-3: External Stability Failure of MSE Walls

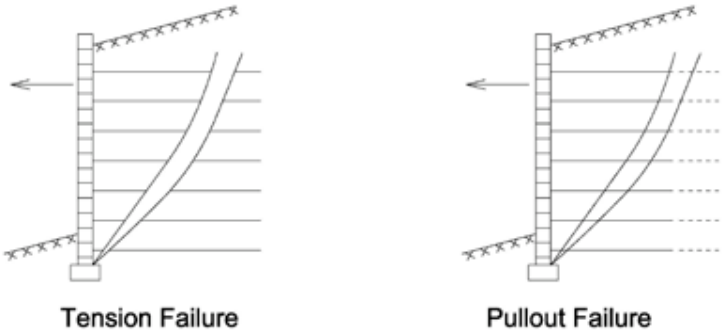


Figure 4.4.2-4: Internal Stability Failure of MSE Walls

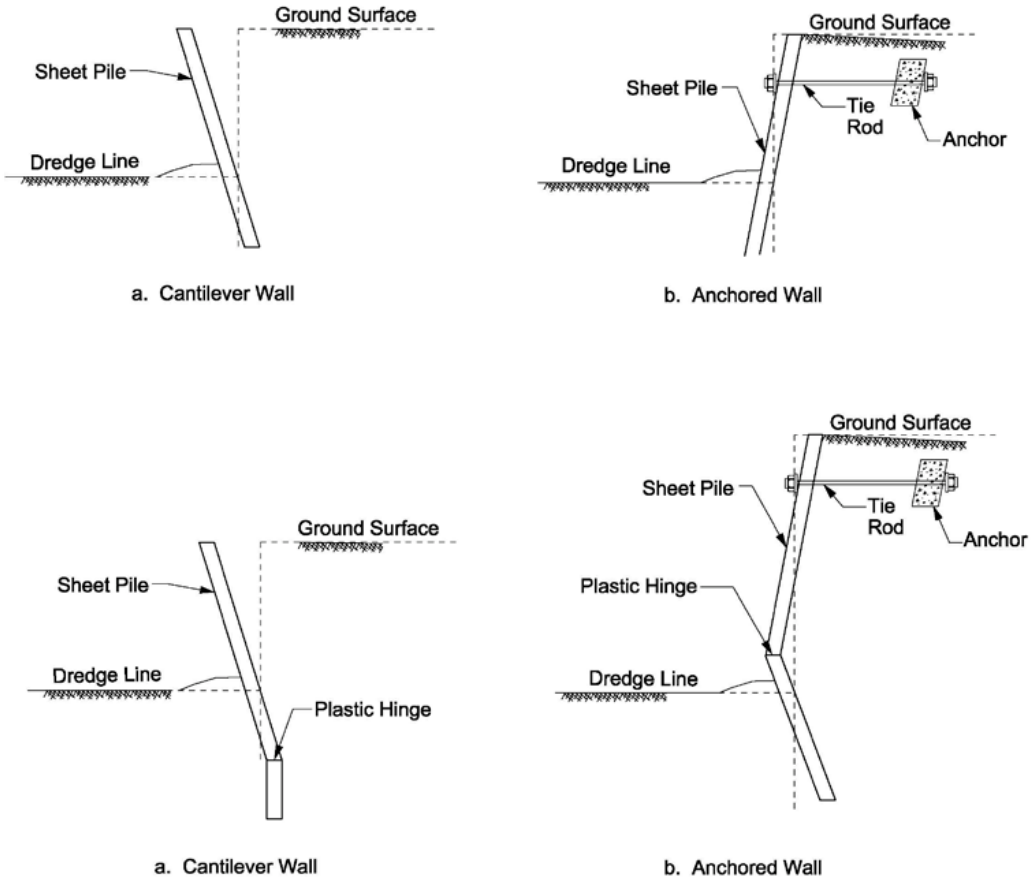


Figure 4.4.2-5: Flexural Failure of Non-Gravity Walls

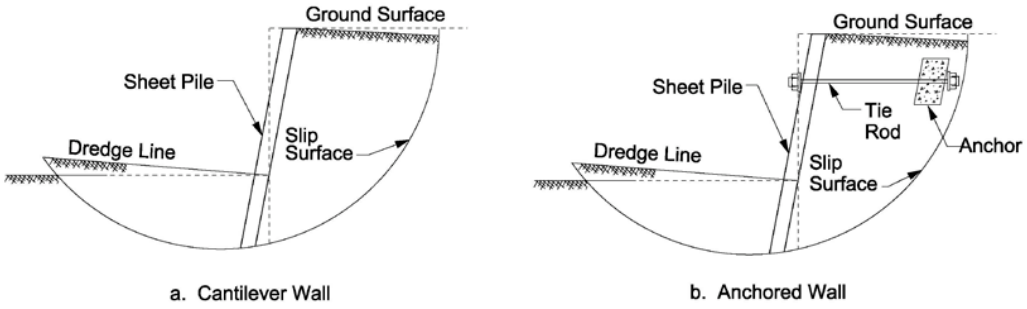


Figure 4.4.2-6: Deep Seated Failure of Non-Gravity Walls

Vertical movement can occur in the form of uniform settlement or differential settlement. Uniform settlement will have little effect on the structural stability of the wall; however, overtopping of the wall may occur if the settlement is significant. Differential settlement, on the other hand, can create serious problems in the wall. Differential settlement may cause the

opening of joints or cause wall cracking or transverse tipping. Refer to Figure 4.4.2-7, Figure 4.4.2-8 and Figure 4.4.2-9 for a diagram and view of differential settlement under a retaining wall.

The most common causes of vertical movement consist of soil bearing failure; soil consolidation; erosion; and foundation material deterioration.

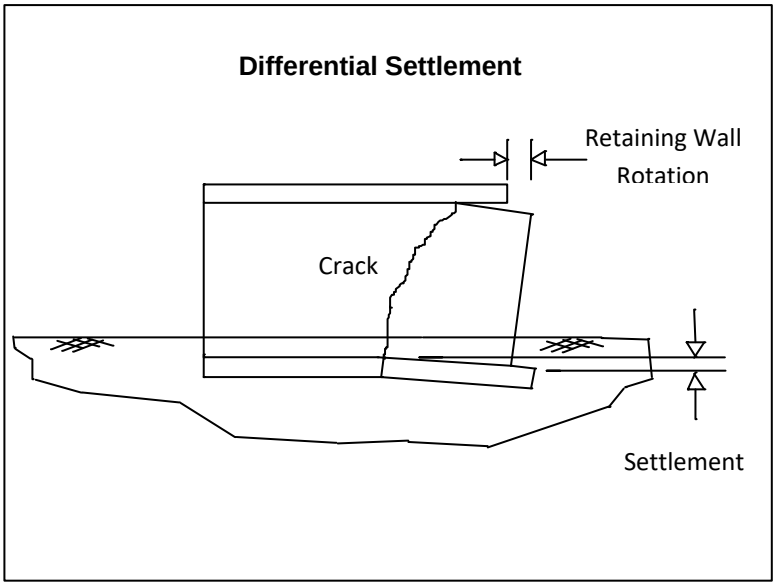


Figure 4.4.2-7: Differential Settlement Under a Retaining Wall.

Figure 4.4.2-8: View of Differential Settlement between Precast Concrete Panels. Note Misalignment in Rustication.

Figure 4.4.2-9: Settlement as Seen from Bridge Rail at Top of Wall.
Note, Settlement was Reflected in Adjacent HMA Pavement.

Earth retention structures are also susceptible to lateral movements or sliding. Lateral movement may occur when the lateral soil pressures exceed the resisting soil frictional and shearing forces, wall anchorages that hold the wall in place or the self weight of different wall components.

The most common causes of lateral movement are slope failures (deep shear failures); seepage; changes in soil characteristics (e.g., frost action and ice); and consolidation of the original soil. Deep shear failures occur along a cylindrical surface when there is a weak layer of soil under the wall at a depth of approximately 1.5 times the width of the base of the wall. Refer to Figure 4.4.2-10 for a diagram depicting lateral movement of a wall due to deep shear failure. Refer to Figure 4.4.2-11 for a view of a mechanically stabilized earth (MSE) wall with extensive lateral movement.

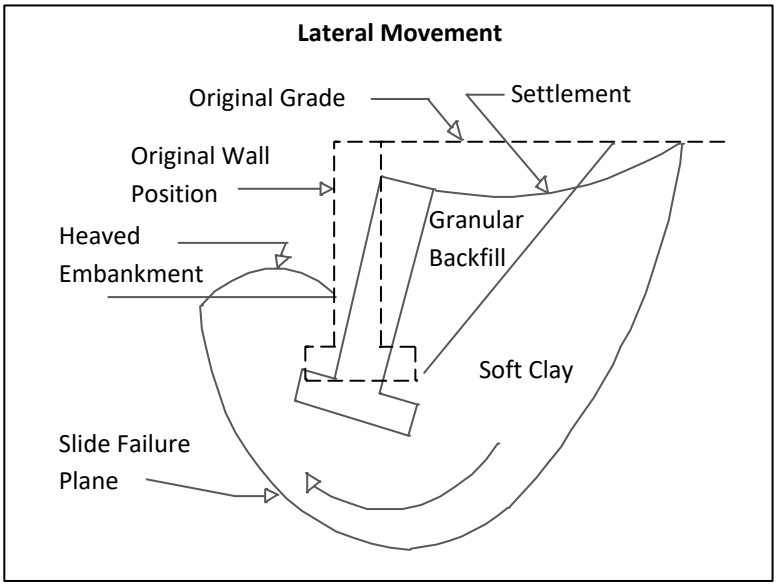


Figure 4.4.2-10: Lateral Movement and Rotation of a Retaining Wall Due to Slope Failure.

Figure 4.4.2-11: Lateral Movement of an Individual Panel of a MSE Wall.

Rotational movement, or overturning, is generally the result of asymmetrical settlements or lateral movements; however, it may result from increased soil pressure behind the wall. Refer to Figure 4.4.2-12 for a diagram of rotational wall movement.

The most common causes of rotational movement are saturation of backfill due to clogged drains; embankment erosion along the front of the wall; and improper design.

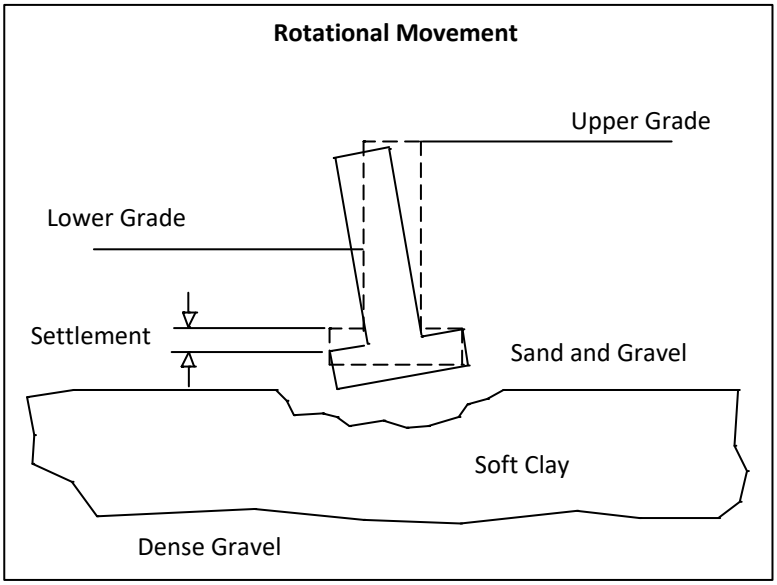


Figure 4.4.2-12: Rotational Movement of a Retaining Wall.

4.4.2.2 Material Deterioration

This section will give a brief description of common material flaws that will be observed during retaining wall inspections. Specific defect condition states will be addressed with specific language for each wall type in Section 4.4.4.

Stone Masonry and Concrete Masonry Units

The following is a discussion of different types of material defects that may be found when inspecting Masonry units. Unreinforced concrete masonry units are commonly used in the construction of segmented block retaining walls.

- **Construction Defects:** Construction flaws are often characterized by damage to the blocks that appear to be due to poor construction technique or errors. Examples might include evidence of cracked units, open joints in locations other than a bend, or evidence of improper design or construction, such as improper block alignment. Refer to Figure 4.4.2-13 for a view of construction defects as shown by the open joint.

Figure 4.4.2-13: Poor Construction Methods Resulting in Block Separation.

- **Corner Breaks:** One or more of the block corners are cracked or broken off. The plane of fracture is approximately 45 degrees from vertical, and the size of the fracture exceeds 2 inches along all three major axes. Smaller breaks should be considered to be fraying or edge spalls. Refer to Figure 4.4.2-14 for a view of a corner break as well as cracking in the top face.

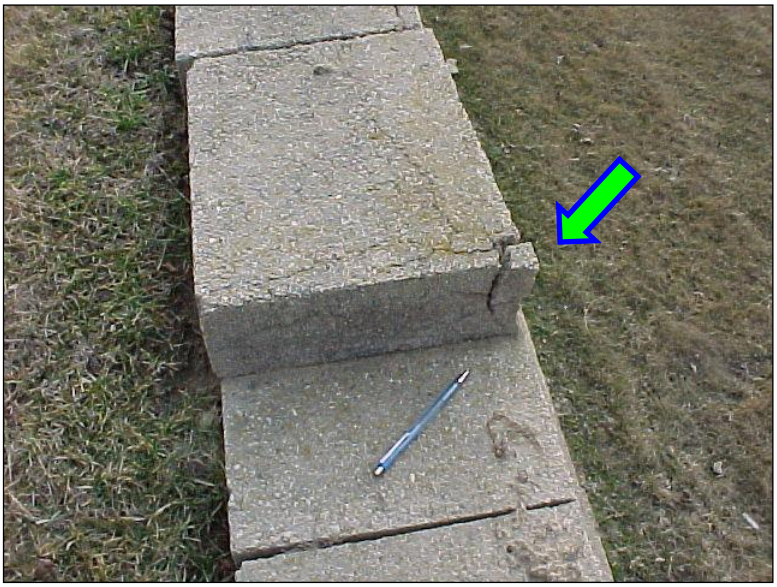


Figure 4.4.2-14: Corner Break with Cracking on the Top Face.

- **Cracked Block:** Randomly cracked block units. The direct cause of the crack development is uncertain. Examples might include diagonal or straight crack propagation across the capstone, or vertical crack propagation along the face of the

stone. Refer to Figure 4.4.2-15 for a view of a transverse crack through the front face of a block.

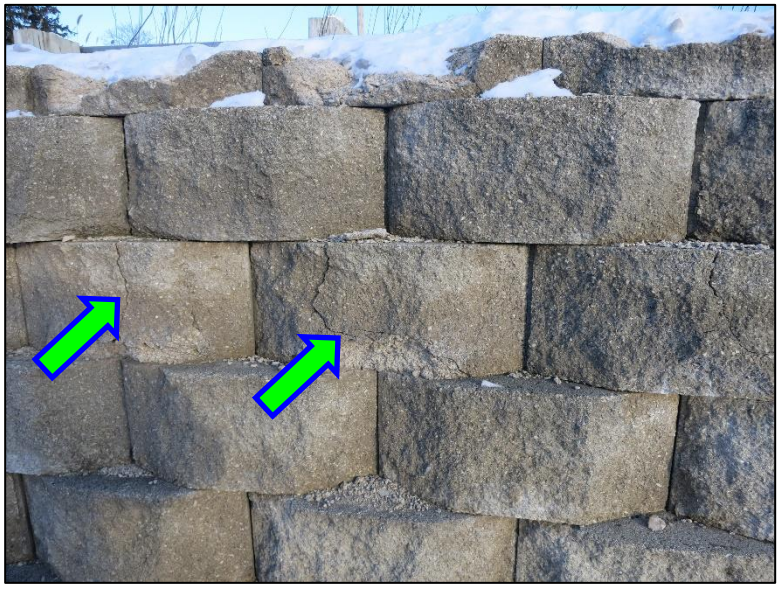


Figure 4.4.2-15: Vertical Cracking through the Front Face of a Block Unit.

- **Efflorescence:** Efflorescence, informally referred to as leaching, is a white deposit on the concrete surface caused by the crystallization of soluble salts (calcium chloride, calcium hydroxide) contained within the cement paste. Water traveling through the concrete dissolves these salts and usually deposits them along cracks where the water exits. Efflorescence indicates that water and dissolved chemicals are able to pass through and contaminate the concrete. This is primarily an aesthetic problem, but can serve as evidence of increased block porosity and weakness in extreme cases. Refer to Figure 4.4.2-16 for a view of efflorescence.

Figure 4.4.2-16: White Precipitate Formed due to Efflorescence.

- **Embedded Vegetative Growth:** This is the presence of plant foliage growing from between the block units or in wash-through deposits. The penetration of plant roots into the blocks may cause units to crack either through root growth into pores and small flaws or by extensive plant growth between block units. Neither exposed fine tree and plant roots that have grown through the wall from behind, nor plants present as architectural enhancements are considered Embedded Vegetative Growth.

- **Abrasion:** Abrasion is the loss of surface material due to the action or water or wind-blown abrasives. This distress may be easily confused with surface scaling and freeze-thaw damage because the latter two are generally more severe in areas of water flow and saturation. Abrasion is typically evidenced by relatively uniform loss of surface mortar along the paths of water flow. Surface scaling and freeze-thaw damage is generally greater in depth.

- **Fraying/Spalling (Block Edges):** Fraying or spalling is the presence of minor chipping along block edges and corners. Occasionally fraying or spalling will be the result of improper handling and placement during construction. Other causes include thermal expansion, or the placement of blocks on uneven surfaces. The presence of a large number of spalls in the same place on each block may indicate a problem in manufacturing. Refer to Figure 4.4.2-17 for a view of spalling and fraying along the block edges.

Figure 4.4.2-17: Spalling and Fraying on the Top Edge of the Block.

- **Freeze-Thaw Damage:** This is the progressive internal deterioration of saturated block material in the presence of freezing and thawing temperatures. The expansion of water during freezing periods can produce significant internal damage. Resulting defects will appear as areas of crumbling or general deterioration. This type of damage is most often found in areas that are frequently saturated and exposed to freezing conditions. Note if this type of damage is occurring on a vertical or horizontal surface. Refer to Figure 4.4.2-18 for a view of typical freeze-thaw damage.

Figure 4.4.2-18: Freeze-Thaw Damage at a Corner.

- **Manufacturing Flaws:** These flaws are evidence by systematic flaws in the block units that are due to a design or manufacturing problem.

- **Popouts:** Aggregate particles near the surface of the block units that have expanded and caused concrete to flake or chip-off.

- **Positioning-Guide Damage:** Some block designs include a small vertical concrete "lip" near the back edge of the block. This lip provides guidance in the positioning of each block to ensure a uniform rate of setback. They are susceptible to manufacturing flaws, or damage due to poor handling.

- **Scaling:** This is a special type of freeze-thaw damage. It is generally characterized by significant exterior damage and crumbling, more so than typical freeze-thaw damage.

- **Staining:** This is the discoloration of block units caused by exposure to elements such as surface runoff containing dark clays or organic material, deicing chemicals, mold growth, moss, and other sources. Refer to Figure 4.4.2-19 for a view of staining.

Figure 4.4.2-19: Typical Staining Evidenced by the Brown Discoloration of the Blocks.

Figure 4.4.2-20: Excessive Gap Forming At Base of Stone Block.

- **Wash Through:** This is the erosion of retained material through the wall. Check for evidence of deposits of retained material on flat surfaces of the exposed side of the wall. Minor deposits are generally cosmetic problems, but can promote vegetative growth. Larger deposits could indicate severe erosion of the retained material.

Concrete

The following is a discussion of different types of material defects that may be found when inspecting Concrete Earth Retention Structures. This includes cast-in-place and precast

concrete wall components. Refer to the SIM Part 2, Chapter 1 for further discussion on concrete deterioration.

- **Cracking:** Random cracks in the structure. The direct cause of the crack development is uncertain and may be attributed to movement, shrinkage or temperature. Examples might include diagonal, straight, or horizontal crack propagation. Refer to Figure 4.4.2-21 for a view of typical vertical cracking found in precast concrete panels.

Figure 4.4.2-21: Vertical Crack Through Precast Concrete Panel.

- **Scaling**: This is a special type of freeze-thaw damage. It is generally characterized by significant exterior damage and crumbling, more so than typical freeze-thaw damage.

- **Spalling/Exposed Reinforcing Steel:** Spalling is the presence of minor chipping along concrete edges and corners. Occasionally spalling will be the result of improper handling and placement during construction but most cases it is caused by the increase in section of reinforcing steel due to corrosion, causing concrete to delaminate from the reinforcing steel and spall. Other causes include thermal expansion, or the placement of concrete panels on uneven surfaces. The presence of a large number of spalls in the same place on each panel may indicate a problem in manufacturing. Consistent spalling over time can result in exposed reinforcing steel. Refer to Figure 4.4.2-22 for a view of spalled concrete with exposed reinforcing steel.

- **Abrasion:** Abrasion is the loss of surface material due to the action or water or wind-blown abrasives. This distress may be easily confused with surface scaling and freeze-thaw damage because the latter two are generally more severe in areas of water flow and saturation. Abrasion is typically evidenced by relatively uniform loss of surface mortar along the paths of water flow. Surface scaling and freeze-thaw damage is generally greater in depth.

Figure 4.4.2-22: Concrete Spalling with Exposed Reinforcing Steel.

Timber

The following is a discussion of different types of material defects that may be found when inspecting Timber Retention Structures. Refer to the SIM Part 2, Chapter 1 for further discussion on timber deterioration.

- **Decay:** This is the breaking down of a material as a result of bacteria or fungi attack. Signs include discolored wood with a soft rotted texture. Look also for fruiting bodies and depressed areas of the wood surface.

- **Insect Infestation:** Often, insects such as termites use timber as food and shelter. These and other insects can be detrimental to the integrity of a timber wall and can cause significant internal damage. Signs can include piles of sawdust, small holes in the surface, insects themselves, or hollows sounds when tapping timber with an inspection hammer.

- **Vermin Damage:** Damage by small animals and birds using the timber for shelter can also be significant.

- **Fire Damage:** This item is generally self-explanatory. If the structure appears black, a piece of it has disappeared, or a large amount of ashes are present at the site, it is likely the structure was damaged by fire.

Metal

The following is a discussion of different types of material defects that may be found when inspecting Metal Retention Structures. Refer to the SIM Part 2, Chapter 1 for further discussion on metal deterioration.

- **Corrosion:** This type of deterioration is the slow, steady material deterioration due to chemical reactions between the material and outside elements. Refer to Figure 4.4.2-23 for a view of steel corrosion.

- **Cracking:** Cracks can occur due to fatigue and or brittle fracture. Cracks are arrested by drilling mouse holes at the tips, or sometimes with bolted splices.

Figure 4.4.2-23: Severely Corroded Steel Sheeting Cantilever Wall.

Other Materials

Several other materials have recently been introduced for use in the construction of retaining walls. These include plastic lumber, vinyl, and geotextile fabrics and grids.

Plastic composites come in a variety of forms including plastic lumber, which is typically formed from recycled high-density polyethylene (HDPE) plastic, vinyl sheet piling, and integrated hybrid composites of plastic and steel. Refer to Figure 4.4.2-24 and Figure 4.4.2-25 for views of plastic, vinyl, and geotextile grid cell retention structures.

Plastics typically exhibit the following types of deterioration:

- Ultraviolet deterioration
- Material Incompatibility
- Corrosion Damage
- Overstress Damage

Plastics will typically exhibit discoloration when undergoing ultraviolet deterioration. The material may also begin to fray when under constant sunlight.

Connections for securing, supporting or bracing other material components should be inspected for corrosion or other similar material deficiencies. For instance the tie rod or nuts anchoring a waler to the outer face of a vinyl sheet piling wall should be inspected to ensure they are properly tightened with no signs of corrosion.

Figure 4.4.2-24: Plastic Lumber Retaining Wall.

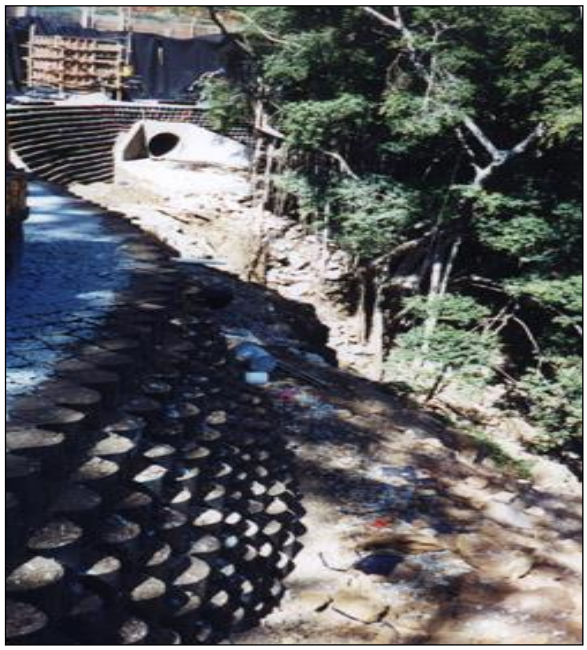


Figure 4.4.2-25: Geotextile Grid Cell Retaining Wall.

4.4.3 Retaining Wall Inspections

A wall inspection consists of observations and/or measurements to determine the condition of wall elements and to identify changes in the condition from previous inspections to ascertain potential maintenance needs and more importantly, assess the ability of the structure to safely serve its intended function.

Walls will be inspected using the applicable WisDOT Elements and Assessments discussed later in this Chapter.

An inspector will not typically be able to inspect tie-backs or other types of anchorages that are embedded behind a wall for stabilization. Only the visible features of the wall including the front face (facade), top, and sides of a wall will typically be inspected during a normal routine inspection. It is the inspector's duty to discern from distress through the observable components if other unseen issues are at play.

4.4.3.1 Inspection Types

Routine

All retaining walls will require a routine visual inspection to be conducted under the inspection interval guidelines set forth in Part 4 Chapter 1 of this manual. In addition, if portions of the wall are submerged where the routine inspector cannot adequately assess the condition of the structure, then an underwater dive inspection will also be required on the same interval as the routine.

In-Depth

In-Depth inspections can also be utilized on retaining wall structures if needed to assess one or more structural elements not readily detectible using routine inspection procedures. These are hands-on, close-up inspections. Each element under investigation should be within arm's reach of the inspector. Non-Destructive Evaluation equipment and/or other material tests may need to be performed.

The inspection may include a request for a structural evaluation to assess the residual capacity of the damaged or deteriorated members, depending on the extent of damage or deterioration.

For large MSE structures (more than 20' in exposed height) where movements are suspected, three dimensional (Lidar) surveys may be requested at 10 year intervals to ascertain movement by the WisDOT survey crew.

This type of inspection is usually on an as needed basis, and scheduling of such an inspection is at the discretion of the ancillary regional program manager.

Damage or Special

Unscheduled inspections are to be performed after a significant event, such as a vehicular impact, extreme weather that could compromise the structure, or indications of significant wall movements.

4.4.3.2 Recommended Routine Inspection Procedures

The following is a list of common inspection procedures when conducting a routine inspection on a retaining wall. For procedures when utilizing NDE/NDT technologies, consult Part 5 of this manual.

1. Arrive at site and set-up traffic control (if required).

2. Identify & Verify Structure Number (Note location within Structure Specific Notes)

3. Perform Inspection.

Due to the similar function of retaining wall types, the following list should be used when inspecting retaining walls to verify proper material function and wall stability.

- Check wall for signs of settlement, rotation, or bulging.

- Inspect the vertical alignment of the wall with a plumb-bob. (Note: Most walls are constructed with a battered or sloped face, therefore this must be taken into account and noted).

- Examine the opening of the construction joints between sections of the wall.

- Inspect joints near ground line for any fill material washing out from between panels or joint.

- Inspect panel joints for differential movement or rotation. Sight down panel face to note individual rotation or tipping out of plane.

- Inspect for erosion of the embankment material in front of the wall.

- Inspect for heaving of the embankment material in front of the wall.

- Inspect for settlement of the fill material behind the wall.

- Examine the wall for deterioration of the material, such as cracking, spalling, and/or corrosion, noting the width, length, depth, and/or orientation of the deterioration. Provide photographs for Condition State 3 (Poor) and 4 defects.

- Some wall types (post and panel, for example) may require the inspector to randomly select a few posts and dig down 3~6” below groundline to see if piling is deteriorating at the soil level.

- Lagging or cribbing should be checked for excessive deflections. Excessive deflections may allow the soild behind to spill or wash out, causing settlement in the retained material above.

- Examine previous areas of repair for soundness.

- Check wall façade for evidence of water seepage, efflorescence or rust staining.

- Examine anchorage systems if present. Fasteners and connections to the wall components should be checked for tightness and distress.

- Examine and probe drains for signs of clogging. Examine drainage around ends of wall and note if embankments have been experiencing erosion.

- Examine site grading for any locations that may prohibit proper drainage from behind the wall. Look for evidence of ponding above the wall, such as debris accumulation in the lower spots. Attempt to ascertain why water is not draining properly, and note in the inspection.

- Inspect sidewalk or roadway components above wall for signs or joint separation, potholes and areas of settlement.

- Examine vegetation growth along and above the wall. Root infiltration may create undesirable stresses on the wall and may induce cracking, bulging or failure.

- Examine the wall system for vehicular damage. Document the location and degree of damage.

4. Determine and record the overall rating of structure based on inspection findings.

5. Determine and record all applicable maintenance items and a level of priority.

6. Determine if an underwater dive inspection or an in-depth inspection needs to be scheduled to supplement the routine inspection and provide more information on the condition and performance of the wall. If determined to be needed, schedule in the HSI System.

7. Review of inspection notes to ensure completeness and correctness.

8. Document all CS3 and CS4 defects with a photo and/or a sketch.

9. Remove any traffic control.

4.4.3.3 Element/Assessment Inspections

For all structure types, WisDOT uses concepts adapted from the AASHTO Manual for Bridge Element Inspection to record condition of the components of a structure. Components are divided into Elements (Primary) and Assessments (Secondary). Both require that the inspector quantify specific conditions states, but Elements take it a step further and also require that the inspector define Defects specific to each Element for asset management purposes.

- Elements – Elements are the main structural components of the wall, including the wall face, vertical supports, foundations, anchors, etc. They are subject to distress, movement, and deterioration on a daily basis. They are considered the most important features rated during the inspection and thus, require thorough descriptions of any defects. All wall type elements are recorded in units of linear feet. Each linear foot includes the vertical projection of the exposed height of the wall element. This area is

inspected and evaluated for any distress with deficiencies captured and quantified under the appropriate wall defects.

- Assessments – Assessments include secondary components such as attachments, appurtenances, and surrounding features that can impact the performance of the wall. They include copings, drainage elements, joints, sidewalks, roadways, slopes and backfill. Quantities may either be in units of each, or Lineal feet depending on the specific assessment.

- Defects - WisDOT simplifies wall inspection into these defects:

    - 8903 Wall Deterioration
    - 8902 Wall Movement
    - 1640 Masonry or Panel Displacement
    - 8000 Scour

The defect 8903 Wall Deterioration essentially acts as an umbrella defect which refers the inspector to the unique defects of the material the wall is constructed. While there is only one material defect to code within the inspection report, it is required that the inspector note the specific material defect observed and measured under the defect note. A detailed description of this defect is included for each wall element in the subsequent sections.

For all walls, any lateral, vertical or rotational movement involving more than one pane or panel shall be coded under the defect 8902 Wall Movement. This defect captures the global movement of the wall as discussed in 4.4.2.1. The term global represents a length or portion of wall greater than a single pane or panel of a retaining wall structure. Similar to a cast-in-place or gravity wall, secant walls are essentially monolithic walls. That is, they are poured together with the intent to work integrally, even with precast or cast-in-place panels placed in front of the secant wall. Movement includes lateral movement (whether in or out along the plane of the wall), differential settlement or global settlement. The major function of all retaining walls is to effectively keep soil from movement in order to allow safe passage of the travelling public either on top or along the wall. Therefore, movement is an immediate concern and shall be measured noted and captured within the inspection reports. The inspector shall note the type, location and length of movement that is occurring under the defect notes (e.g. overturning, settlement, sliding, etc.). A detailed description of this defect is included for each wall element in the subsequent sections.

Localized (individual or small groups) panel or masonry block movement and rotation of MSE or Masonry block walls shall be captured under the defect 1640 Masonry or Panel Displacement. In the instance of several MSE panels or masonry blocks exhibiting movement, the inspector should utilize the material defect 8902 Wall Movement. Refer to section 4.4.2.1 for modes of geotechnical failure. A detailed description of this defect is included for each relevant wall element in the subsequent sections.

There may arise the necessity to quantify scour for a retaining structure abutting a body of water. The Scour (8000) is a structural defect. Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with the other wall defects (i.e. structural

defects do not fall under the defect hierarchy). Structural defects represent global or external deterioration that adversely affects the wall. The scour defect should be used to capture the erosion of material along the base of a wall due to a stream or body of water. The defect shall be coded under the primary wall element (i.e. MSE Wall, Sheet Pile Wall, etc.).

4.4.4 Retaining Wall Elements

Retaining wall components are designed to restrain soil and can be made out of steel, concrete, timber, masonry/stone, and other materials. A variety of elements are available for the inspector to utilize during an inspection to best capture the type of wall being evaluated.

The quantity for wall elements is the length of the wall in lineal feet measured from end to end with each linear foot capturing the defects over the entire height of the wall element. Other elements that may be part of the wall, such as bridge railings and copings, are also evaluated as linear feet. All visible portions of the wall shall be considered during the inspection, including all assessments found on the wall.

4.4.4.1 CIP Concrete Wall (Element 8600)

Cast-in-place (CIP) concrete walls are made by placing ready-mix concrete into removable forms that are built around reinforcing steel at the final intended position of the wall. A CIP wall can incorporate tie-backs or prestressed soil anchors for stabilization depending on the height of the wall and characteristics of the soil that is being restrained. Footings with piles may also be used. The weight of the soil on the inner half of the footing slab (heel) provides additional vertical weight for stability. The wall is fully reinforced and relatively thin, allowing for an economical use of materials.

Figure 4.4.4-1: Cast-in-Place Concrete Cantilever Wall.

Figure 4.4.4-2: Cast-in-Place Concrete Cantilever Wall.

The exposed concrete stem may be plain, textured and/or colored. The height on these types of walls is typically less than 28 feet. Walls higher than 28 feet typically require counterforts or tie-backs for additional strength and deflection control. Refer to Figure 4.4.4-1 and Figure 4.4.4-2: Cast-in-Place Concrete Cantilever Wall. for a typical cast-in-place concrete gravity cantilever wall.

Use the below definition(s) to record Defect 8903 Wall Deterioration for this element:

- ***Condition State 1 (Good)***: No spalls, delaminations, abrasions, or patched areas. Cracks, if they exist, have been sealed or are less than 0.012".

- ***Condition State 2 (Fair)***: Delaminations may be present. Spalls, if present, are 1" or less in depth or less than 6" in diameter. Patched areas that are sound. If rebar is exposed, there is no section loss. Cracks, if they exist, are between 0.012" - 0.05". Where efflorescence is present, it's minor and no evidence of rust staining. Abrasions, if they exist, have exposed course aggregate but the aggregate remains secure in the concrete.

- ***Condition State 3 (Poor)***: Spalls greater than 1" deep, or greater than 6" in diameter. Patched areas that are unsound. Exposed rebar, if present, has measurable section loss. Cracks, if present, are greater than 0.05" wide. Where efflorescence is present, there is heavy build-up and/or rust staining. If abrasions are present, the course aggregate is loose or has popped out of the concrete matrix. Conditions are not sufficient to warrant structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Use the below definition(s) to record Defect 8902 Wall Movement for this element:

- ***Condition State 1 (Good)***: Wall elements are as constructed, and/or show no signs of settlement, bulging, bending, heaving, or distortion/deflection beyond normal prescribed post-construction limits. Wall elements are fully bearing against retained soil/rock units.

- ***Condition State 2 (Fair)***: Wall movement has started to occur. Wall shows signs of settlement, bulging, bending, heaving, misalignment, distortion, deflection and/or displacement beyond normal prescribed post construction limits (i.e. wall face rotation, basket budging, anchor head displacement, bin displacement). Wall may be strapped or anchored to prevent further movement, or movement has been arrested through countermeasures. Wall elements are mostly bearing against retained soil/rock units.

- ***Condition State 3 (Poor)***: Wall rotation/sliding/settlement is active and extensive and well beyond normal post-construction limits; sloughing of retained material behind wall is evident. Wall may have been strapped, anchored or stabilized to prevent further movement, but this device has failed. Many or key wall elements are no longer bearing against retained soil/rock units.

- ***Condition State 4 (Severe)***: Wall has failed.

It is important to note that these condition states are for the linear foot unit of measure. Therefore condition states 3 and 4 shall have representative measurements and photographs within the report.

4.4.4.2 Gabion Wall (Element 8601)

Gabion gravity crib walls are constructed from rock-filled wire mesh baskets. The gabions typically have a heavy wire mesh with a nominal 3-inch opening and are formed into rectangular baskets, normally 1.5 to 3 feet in height. Individual baskets are placed on the prepared earthen surface, reinforced with internal tie wires, and filled with a select stone ranging from 4-inch to 10-inch diameter. After the baskets are filled, the wire lids are closed and wired shut to form a relatively rigid block. Succeeding rows of gabions are laced onto the filled underlying gabions and filled in the same manner. Manufacturers typically provide details for the wires, lacing, and lid closure. As wall heights increase, more baskets are wired together to increase the wall depth into the slope. Gabion walls are typically less than 18 feet tall. Geotextile fabric is placed behind the baskets to keep the backfill soils from entering the rock filled gabions. Gabion gravity crib walls are often used in fill sections of a roadway and/or adjacent to waterways. For water installations, the wall typically needs additional protection from scour by the use of riprap or other suitable material.

There are a variety of different types of gabion gravity crib walls on the market. They can be easily constructed without the use of skilled labor or specialized equipment, and they are often built rapidly. Unfortunately, it is difficult to make height adjustments to the wall once in place. Refer to Figure 4.4.4-3 and Figure 4.4.4-4 for views of gabion gravity crib walls.

Figure 4.4.4-3: Gabion Gravity Crib Wall Under Construction.

Use the below definition(s) to record Defect 8903 Wall Deterioration for this element:

- ***Condition State 1 (Good)***: No Defects Noted.
- ***Condition State 2 (Fair)***: Freckled rust with corrosion of the steel basket initiating. Loose fasteners without distortion are present but the connection is in place and functioning as intended. Stones are split, but no shifting.
- ***Condition State 3 (Poor)***: Section loss of the steel basket is evident. Broken or missing fasteners have caused localized distortions. Stones are splitting and spalled with some differential movement but does not warrant a structural review.
- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Use the below definition(s) to record Defect 8902 Wall Movement for this element:

- ***Condition State 1 (Good)***: Wall elements are as constructed, and/or show no signs of settlement, bulging, bending, heaving, or distortion/deflection beyond normal prescribed post-construction limits. Wall elements are fully bearing against retained soil/rock units.
- ***Condition State 2 (Fair)***: Wall movement has started to occur. Wall shows signs of settlement, bulging, bending, heaving, misalignment, distortion, deflection and/or displacement beyond normal prescribed post construction limits (i.e. wall face rotation, basket budging, etc). Wall elements are mostly bearing against retained soil/rock units.
- ***Condition State 3 (Poor)***: Wall rotation/sliding/settlement is active and extensive and well beyond normal post-construction limits; sloughing of retained material behind wall is evident. Wall may have been stabilized to prevent further movement, but this device

has failed. Many or key wall elements are no longer bearing against retained soil/rock units.

- ***Condition State 4 (Severe)***: Wall has failed.

It is important to note that these condition states are for the linear foot unit of measure. Therefore condition states 3 and 4 shall have representative measurements and photographs within the report.

Figure 4.4.4-4: Tiered Gabion Gravity Crib Wall.

4.4.4.3 Gravity (Block & Rubble) Wall (Element 8602)

A gravity wall is a large monolithic structure, which depends entirely on its self-weight and the weight of the soil that rests upon it for stability. The large mass needed to develop adequate weight requires a substantial footprint; therefore available space may limit its use. The walls are typically constructed of concrete with very little, if any, steel reinforcement, masonry or stone block. Concrete gravity walls are typically less than 10 feet tall, and segmented modular block gravity walls are limited by design to an exposed height of 4'-0" without the use of geosynthetic reinforcement behind the wall. Masonry or stone block gravity walls will have varying heights and are entirely dependent on the available footprint. The blocks of these gravity walls are freestanding with joints that may or may not be mortared. Therefore taller walls of this type will require a significant footprint at the base to retain the soil. Refer to Figure 4.4.4-5 for a view of a segmented modular block gravity wall.

Use the below definition(s) to record Defect 8903 Wall Deterioration for this element:

- ***Condition State 1 (Good)***: No Defects Noted.

- ***Condition State 2 (Fair)***: If using mortar, cracking or voids in less than 10% of the joints. Block or stone has split or spalled with no shifting. Patched areas are sound.

- ***Condition State 3 (Poor)***: If using mortar, cracking or voids in 10% or more of the joints. Block or stone has split or spalled with shifting. Patched areas are not sound. Does not require a structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Figure 4.4.4-5: Segmented Modular Block Gravity Wall.

Figure 4.4.4-6: Lannon Stone (Limestone) Block Gravity Wall.

Use the below definition(s) to record Defect 8902 Wall Movement for this element:

- ***Condition State 1 (Good)***: Wall elements are as constructed, and/or show no signs of settlement, bulging, bending, heaving, or distortion/deflection beyond normal prescribed post-construction limits. Wall elements are fully bearing against retained soil/rock units.

- ***Condition State 2 (Fair)***: Wall movement has started to occur. Wall shows signs of settlement, bulging, bending, heaving, misalignment, distortion, deflection and/or displacement beyond normal prescribed post construction limits (i.e. wall face rotation, basket budging, anchor head displacement, bin displacement). Wall may be strapped or anchored to prevent further movement, or movement has been arrested through countermeasures. Wall elements are mostly bearing against retained soil/rock units.

- ***Condition State 3 (Poor)***: Wall rotation/sliding/settlement is active and extensive and well beyond normal post-construction limits; sloughing of retained material behind wall is evident. Wall may have been strapped, anchored or stabilized to prevent further movement, but this device has failed. Many or key wall elements are no longer bearing against retained soil/rock units.

- ***Condition State 4 (Severe)***: Wall has failed.

Use the below definition(s) to record Defect 1640 Masonry or Panel Displacement:

- ***Condition State 1 (Good)***: None. No movement is observed or measured.

- ***Condition State 2 (Fair)***: Block, stone or panel has shifted slightly out of alignment.

- ***Condition State 3 (Poor)***: Block, stone or panel has shifted significantly out of alignment or is missing but does not warrant a structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

It is important to note that these condition states are for the linear foot unit of measure. Therefore condition states 3 and 4 shall have representative measurements and photographs within the report.

4.4.4.4 MSE Wall (Element 8603)

An MSE wall is based on the principle of integrating reinforcing into a granular backfill via means such as metal strips or rods, geosynthetic sheets, or wire grids. This reinforces the soil against shear failure, and allows the soil to act as a single large mass. The friction stresses developed between the granular backfill and the reinforcement resists bond pullout. The reinforcing is tied to precast concrete facing units, which form the vertical face of the wall. The facing units are relatively small and piece together in a geometric pattern. The reinforcing is attached at regular intervals throughout the width and height of the wall.

MSE Walls can be constructed with different materials including precast concrete panel facings (classic vertical faced MSE walls), Modular Block facings (battered segmental retaining walls), and Wire faced walls.

The Segmented Modular Block Retaining Wall is the most common of MSE structure in Wisconsin. Although these walls can act as Gravity walls, they are usually constructed as MSE walls. When constructed as gravity or crib walls, these retention structures are limited to an exposed height of 4'-0" and are embedded 1'-6". Consequently, most of these walls fall under the MSE category. The height to which they can be constructed is a function of the block width, site geometry, setback angle of the wall, angle of the back slope behind the wall, and the design parameters of the retained and foundation soils. Refer to Figure 4.4.4-7 for a view of a Segmented Modular Block Wall. These walls are proprietary and the wall supplier provides their design. Since placement of reinforcing is required, these walls are best used in fill sections. They require good foundation material where little differential settlement is expected.

Use the below definition(s) to record Defect 8903 Wall Deterioration for segmented modular block MSE walls:

- ***Condition State 1 (Good)***: No Defects Noted.

- ***Condition State 2 (Fair)***: If using mortar, cracking or voids in less than 10% of the joints. Block or stone has split or spalled with no shifting. Patched areas are sound.

- ***Condition State 3 (Poor)***: If using mortar, cracking or voids in 10% or more of the joints. Block or stone has split or spalled with shifting. Patched areas are not sound. Does not require a structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Figure 4.4.4-7: Segmented Modular Block MSE Wall.

For the Precast Concrete Panel Facing MSE wall, metallic strips, or wire grids are used to reinforce the soil mass and act as a gravity retaining structure. This type of MSE Wall is best used in fill sections. Refer to Figure 4.4.4-8 for a View of a Precast Concrete Panel MSE Wall. These walls cannot be built over existing utilities, and new utilities cannot be located within the reinforced soil. All reinforcement must lie within the permanent right of way. Foundation materials must have adequate bearing capacity to support the loads. Differential settlement limitations vary based on height and construction. Refer to the WisDOT Bridge Manual Chapter 14 for the maximum differential settlements.

Precast concrete panel faced MSE walls are very competitive economically with other walls when heights exceed 20 feet. They can be used as bridge wingwalls and in conjunction with sill abutments. However, they require a specific type of backfill with non-corrosive properties since there is corrosion potential for metallic strips. The MSE precast panels are founded on an unreinforced concrete leveling pad to provide a level base. However, as the pad is unreinforced, any exposure or undermining of the pad can have major implications to the stability of the fill behind the panels.

Another type of MSE Wall is the MSE Wall with Modular Block Facing. The same general principles apply as the previous type of MSE Wall with some slight variations. These walls typically are not used in the following situations: as a component of an abutment structure; or where traffic barriers/roadway pavements must be vertically supported by the wall. Differential settlement limitations vary based on height and construction. These walls typically have a maximum height of 18 feet.

Use the below definition(s) to record Defect 8903 Wall Deterioration for precast concrete panel facing MSE walls:

- ***Condition State 1 (Good)***: No spalls, delaminations, abrasions, or patched areas. Cracks, if they exist, have been sealed or are less than 0.012”.

- ***Condition State 2 (Fair)***: Delaminations may be present. Spalls, if present, are 1” or less in depth or less than 6” in diameter. Patched areas that are sound. If rebar is

exposed, there is no section loss. Cracks, if they exist, are between 0.012" - 0.05". Where efflorescence is present, it's minor and no evidence of rust staining. Abrasions, if they exist, have exposed course aggregate but the aggregate remains secure in the concrete.

- ***Condition State 3 (Poor)***: Spalls greater than 1" deep, or greater than 6" in diameter. Patched areas that are unsound. Exposed rebar, if present, has measurable section loss. Cracks, if present, are greater than 0.05" wide. Where efflorescence is present, there is heavy build-up and/or rust staining. If abrasions are present, the course aggregate is loose or has popped out of the concrete matrix. Conditions are not sufficient to warrant structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Figure 4.4.4-8: MSE Precast Concrete Panel Facing Wall.

Use the below definition(s) to record Defect 8902 Wall Movement for this element:

- ***Condition State 1 (Good)***: Wall elements are as constructed, and/or show no signs of settlement, bulging, bending, heaving, or distortion/deflection beyond normal prescribed post-construction limits. Wall elements are fully bearing against retained soil/rock units.

- ***Condition State 2 (Fair)***: Wall movement has started to occur. Wall shows signs of settlement, bulging, bending, heaving, misalignment, distortion, deflection and/or displacement beyond normal prescribed post construction limits (i.e. wall face rotation, basket budging, anchor head displacement, bin displacement). Wall may be strapped

or anchored to prevent further movement, or movement has been arrested through countermeasures. Wall elements are mostly bearing against retained soil/rock units.

- ***Condition State 3 (Poor)***: Wall rotation/sliding/settlement is active and extensive and well beyond normal post-construction limits; sloughing of retained material behind wall is evident. Wall may have been strapped, anchored or stabilized to prevent further movement, but this device has failed. Many or key wall elements are no longer bearing against retained soil/rock units.

- ***Condition State 4 (Severe)***: Wall has failed.

Use the below definition(s) to record Defect 1640 Masonry or Panel Displacement:

- ***Condition State 1 (Good)***: None. No movement is observed or measured.

- ***Condition State 2 (Fair)***: Block, stone or panel has shifted slightly out of alignment.

- ***Condition State 3 (Poor)***: Block, stone or panel has shifted significantly out of alignment or is missing but does not warrant a structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

It is important to note that these condition states are for the linear foot unit of measure. Therefore condition states 3 and 4 shall have representative measurements and photographs within the report.

4.4.4.5 Post & Panel Wall (Element 8604)

Post and Panel Walls are comprised of vertical elements (typically H piles) and panels (concrete or timber) which extend between the vertical elements. The panels resist lateral soil pressures by spanning horizontally between the posts. The panels are usually constructed of precast reinforced concrete although precast/prestressed concrete and timber are also possibilities. Refer to Figure 4.4.4-9 and Figure 4.4.4-10 for typical views of post and panel walls using concrete panels and timber panels respectively. The use of Post and Panel walls should be considered if minimal environmental damage or disturbance to the site from construction procedures is critical. These walls also function well in a cut section, where right of way is limited.

Use the below definition(s) to record Defect 8903 Wall Deterioration steel posts with concrete panels:

- ***Condition State 1 (Good)***: No spalls, delaminations, abrasions, or patched areas. Cracks, if they exist, have been sealed or are less than 0.012”.

- ***Condition State 2 (Fair)***: Delaminations may be present. Spalls, if present, are 1” or less in depth or less than 6” in diameter. Patched areas that are sound. If rebar is exposed, there is no section loss. Cracks, if they exist, are between 0.012” - 0.05”. Where efflorescence is present, it’s minor and no evidence of rust staining. Abrasions,

if they exist, have exposed course aggregate but the aggregate remains secure in the concrete. Freckle rust on the posts. Corrosion of the steel has initiated. Cracking of the steel has self-arrested, or has been arrested with effective holes, doubling plates, or similar. Fasteners may be loose, but are performing the intended function.

- ***Condition State 3 (Poor)***: Spalls greater than 1” deep, or greater than 6” in diameter. Patched areas that are unsound. Exposed rebar, if present, has measurable section loss. Cracks, if present, are greater than 0.05” wide. Where efflorescence is present, there is heavy build-up and/or rust staining. If abrasions are present, the course aggregate is loose or has popped out of the concrete matrix. Section loss of the posts is evident. Cracks exist in the steel that have not been arrested. Missing bolts, broken welds, or other fastener damage with some distortion. Conditions are not sufficient to warrant structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Figure 4.4.4-9: Post and Panel Cantilever Wall with Precast Panels and Steel "H" Piles.

Figure 4.4.4-10: Post and Panel Cantilever Wall with Timber Planks and Steel "H" Piles.

Use the below definition(s) to record Defect 8903 Wall Deterioration steel posts with timber planks:

- ***Condition State 1 (Good)***: No spalls, delaminations, abrasions, or patched areas. Cracks, if they exist, have been sealed or are less than 0.012".

- ***Condition State 2 (Fair)***: Decay or section loss of the timber affects <10% of the member section. Checks/Cracks penetrate <5% of the timber member thickness. Timber members do not have splits/shakes/delaminations. Section loss of the timber due to abrasion is < 10% of the member thickness. Freckle rust on the steel posts. Corrosion of the steel has initiated. Cracking of the steel has self-arrested, or has been arrested with effective holes, doubling plates, or similar. Fasteners may be loose, but are performing the intended function.

- ***Condition State 3 (Poor)***: Decay or section loss of the timber affects 10% or more of the member. Checks/cracks penetrate 5%~50% of the member thickness. Timber member has splits/shakes with length less than member depth. Larger cracks/splits/shakes have been arrested with effective repairs. Section loss of the timber member due to abrasion is 10% or more of the member thickness. Section loss of the steel posts is evident. Cracks exist in the steel that have not been arrested. Missing bolts, broken welds, or other fastener damage with some distortion. Conditions are not sufficient to warrant structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Use the below definition(s) to record Defect 8902 Wall Movement for this element:

- ***Condition State 1 (Good)***: Wall elements are as constructed, and/or show no signs of settlement, bulging, bending, heaving, or distortion/deflection beyond normal prescribed post-construction limits. Wall elements are fully bearing against retained soil/rock units.

- ***Condition State 2 (Fair)***: Wall movement has started to occur. Wall shows signs of settlement, bulging, bending, heaving, misalignment, distortion, deflection and/or displacement beyond normal prescribed post construction limits (i.e. wall face rotation, basket budging, anchor head displacement, bin displacement). Wall may be strapped or anchored to prevent further movement, or movement has been arrested through countermeasures. Wall elements are mostly bearing against retained soil/rock units.

- ***Condition State 3 (Poor)***: Wall rotation/sliding/settlement is active and extensive and well beyond normal post-construction limits; sloughing of retained material behind wall is evident. Wall may have been strapped, anchored or stabilized to prevent further movement, but this device has failed. Many or key wall elements are no longer bearing against retained soil/rock units.

- ***Condition State 4 (Severe)***: Wall has failed.

4.4.4.6 Sheet Pile Wall (Element 8605)

A common type retention structure found in Wisconsin is the Steel Sheet Pile Wall. These walls can either be temporary or permanent walls and they may or may not be anchored (often depends on exposed height of wall). Temporary walls can be used in construction to retain fill materials while construction adjacent to the wall is performed. Permanent sheet pile walls will typically be found along rivers and shorelines to protect the shoreline from erosion. Another permanent use for sheet pile walls is around boat docks to allow for proper water depths for boat to draft. The corrosion potential for these walls is high, and site conditions should be taken into account. These walls should not be used in areas where there is shallow bedrock. A steel sheet pile wall can be a maximum of approximately 15 feet high without tiebacks being required. Only the connections at the face of the wall for tiebacks are accessible for inspection. Therefore the inspector should carefully note any distress in fasteners. Refer to Figure 4.4.4-11 and Figure 4.4.4-12 for a view of an anchored sheet pile dockwall and a failed tie-rod, respectively.

Protective coatings, such as paint or impervious water membranes, are provided on steel members, shall be captured under the assessment 9010 Aesthetic Treatments. If, under the inspector's discretion, the coating has become compromised and should be replaced to prevent corrosion, the inspector may create an appropriate maintenance item.

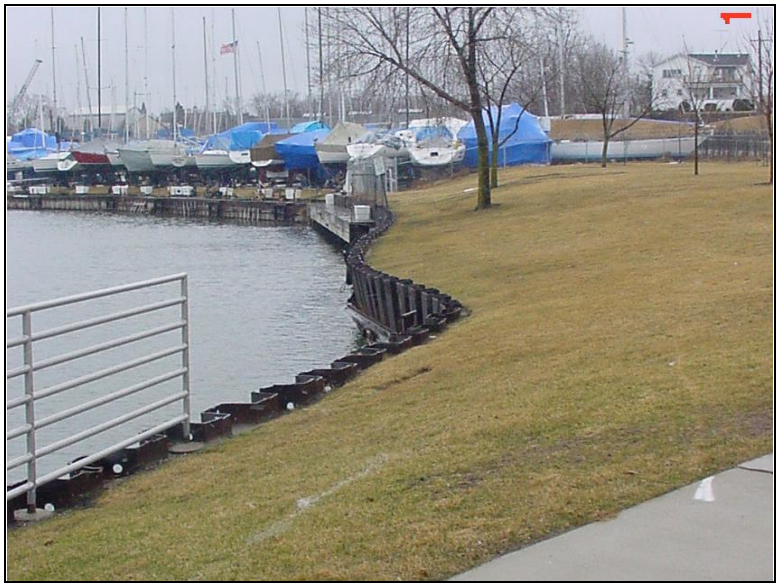


Figure 4.4.4-11: Tilted and Deformed Sheet pile Dockwall.

Use the below definition(s) to record Defect 8903 Wall Deterioration for Steel Sheet Pile Walls:

- ***Condition State 1 (Good)***: No Defects
- ***Condition State 2 (Fair)***: Freckle rust on the steel. Corrosion of the steel has initiated. Cracking of the steel has self-arrested, or has been arrested with effective holes, doubling plates, or similar. Fasteners may be loose, but are performing the intended function.
- ***Condition State 3 (Poor)***: Section loss of the steel posts is evident. Cracks exist in the steel that have not been arrested. Missing bolts, broken welds, or other fastener damage with some distortion. Conditions are not sufficient to warrant structural review.
- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Figure 4.4.4-12: Failed Tie-rod Along a Sheet Pile Dockwall.

Use the below definition(s) to record Defect 8902 Wall Movement for this element:

- ***Condition State 1 (Good)***: Wall elements are as constructed, and/or show no signs of settlement, bulging, bending, heaving, or distortion/deflection beyond normal prescribed post-construction limits. Wall elements are fully bearing against retained soil/rock units.

- ***Condition State 2 (Fair)***: Wall movement has started to occur. Wall shows signs of settlement, bulging, bending, heaving, misalignment, distortion, deflection and/or displacement beyond normal prescribed post construction limits (i.e. wall face rotation, basket budging, anchor head displacement, bin displacement). Wall may be strapped or anchored to prevent further movement, or movement has been arrested through countermeasures. Wall elements are mostly bearing against retained soil/rock units.

- ***Condition State 3 (Poor)***: Wall rotation/sliding/settlement is active and extensive and well beyond normal post-construction limits; sloughing of retained material behind wall is evident. Wall may have been strapped, anchored or stabilized to prevent further movement, but this device has failed. Many or key wall elements are no longer bearing against retained soil/rock units.

- ***Condition State 4 (Severe)***: Wall has failed.

4.4.4.7 Reinforced Soil Slope (Element 8606)

Reinforced soil slope are used when designers are required to maximize the amount of land used on a project. Due to certain slope material conditions, the unreinforced slope is too shallow, thus using too much land to obtain a stable slope. Reinforced soil slops add tensile inclusions with soil to create a composite material. The tensile inclusions are typically

geosythetic reinforcement. Vegetation can be used as the facing of the composite material for slopes less than 45 degrees but more significant armoring may be needed for steeper slopes such as high strength concrete, emulsified asphalt, riprap, stone veneer or articulating modular units. These are not common in Wisconsin and thus no discussion of the defect condition states are available at this time.

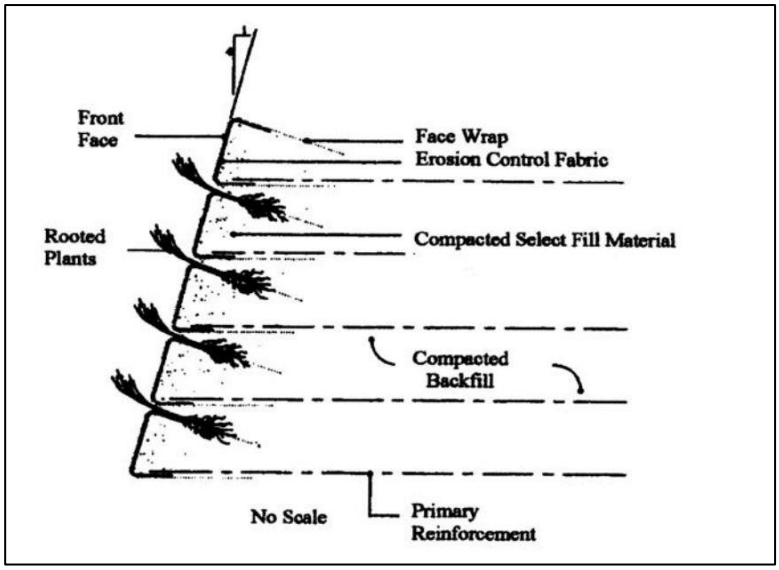


Figure 4.4.4-13: Schematic of a Reinforced Soil Slope with Vegetation Facing.

4.4.4.8 Secant or Tangent Shaft Wall (Element 8607)

Secant and Tangent walls are a series of interlocking concrete drilled shafts. These walls are constructed by drilling a series of reinforced shafts filled with concrete spaced at intervals less than the diameter of the shaft. After the concrete has hardened but before it has completely cured, a reinforced shaft is drilled through both adjacent drilled shafts, thus locking them together. In some instances steel H-piles are placed in the drilled shaft for additional reinforcement. The shafts act together to retain the soil. Once the wall is fully cured, one side of the wall is excavated. Typically precast concrete panels are placed in front of the secant pile wall as an aesthetic façade.

In Wisconsin, Secant Pile Walls used in tunnel construction may be covered with a reinforced shotcrete façade, precast concrete panel façade or other aesthetic feature. Regardless, visual inspection of the secant piles is typically not possible and must be evaluated by distress on the façade paneling. The inspector should look for signs of water seepage, panel distortion, cracking and alignment to gauge how the wall is performing. Refer to Figure 4.4.4-14 for a photo of a Secant Wall being installed and Figure 4.4.4-15 for a photo of the finished wall. Refer to Figure 4.4.4-16 for a photo of distress in a Secant Wall.

Use the below definition(s) to record Defect 8903 Wall Deterioration for this element:

- ***Condition State 1 (Good)***: No spalls, delaminations, abrasions, or patched areas. Cracks, if they exist, have been sealed or are less than 0.012”.

- ***Condition State 2 (Fair)***: Delaminations may be present. Spalls, if present, are 1” or less in depth or less than 6” in diameter. Patched areas that are sound. If rebar is exposed, there is no section loss. Cracks, if they exist, are between 0.012” - 0.05”. Where efflorescence is present, it’s minor and no evidence of rust staining. Abrasions, if they exist, have exposed course aggregate but the aggregate remains secure in the concrete.

- ***Condition State 3 (Poor)***: Spalls greater than 1” deep, or greater than 6” in diameter. Patched areas that are unsound. Exposed rebar, if present, has measurable section loss. Cracks, if present, are greater than 0.05” wide. Where efflorescence is present, there is heavy build-up and/or rust staining. If abrasions are present, the course aggregate is loose or has popped out of the concrete matrix. Conditions are not sufficient to warrant structural review.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall, or the wall has failed due to deterioration.

Figure 4.4.4-14: Secant Wall Construction.

Figure 4.4.4-15: Secant Wall Constructed as Tunnel Wall.

Figure 4.4.4-16: Excessive Cracking within Secant Wall Facade Concrete Panel.

Use the below definition(s) to record Defect 8902 Wall Movement for this element:

- ***Condition State 1 (Good)***: Wall elements are as constructed, and/or show no signs of settlement, bulging, bending, heaving, or distortion/deflection beyond normal prescribed post-construction limits. Wall elements are fully bearing against retained soil/rock units.

- ***Condition State 2 (Fair)***: Wall movement has started to occur. Wall shows signs of settlement, bulging, bending, heaving, misalignment, distortion, deflection and/or

displacement beyond normal prescribed post construction limits (i.e. wall face rotation, basket budging, anchor head displacement, bin displacement). Wall may be strapped or anchored to prevent further movement, or movement has been arrested through countermeasures. Wall elements are mostly bearing against retained soil/rock units.

- ***Condition State 3 (Poor)***: Wall rotation/sliding/settlement is active and extensive and well beyond normal post-construction limits; sloughing of retained material behind wall is evident. Wall may have been strapped, anchored or stabilized to prevent further movement, but this device has failed. Many or key wall elements are no longer bearing against retained soil/rock units.

- ***Condition State 4 (Severe)***: Wall has failed.

4.4.4.9 Other Wall (Element 8608)

This element is intended to be used for any type of earth retention wall that does not fit into any of the previously mentioned elements. Some of the wall types that may be encountered under this element would be Concrete Crib Wall and Timber Crib Wall. Welded wire walls should be evaluated under the MSE Wall element as these are typically vertical walls with attached panel façade. As these are not typical, defects will not be discussed in this section.

Figure 4.4.4-17: Vinyl Sheet Piling Retaining Wall.

4.4.5 Retaining Wall Assessments

Assessments allow for the evaluation of secondary components not necessarily impacting the structural integrity of the structure. The inspector must be familiar with all the available assessments within the HSI system and appropriately capture them within the inspection report for each retaining wall structure. Evaluating assessments provides a more accurate picture of the structure being inspected and alerting the Department of potential future issues including traffic hazards or functionality of the wall. The following section describes all the available assessments that may be reported within HSI.

4.4.5.1 Drainage Approach (9001)

This assessment captures the condition of the soil and slopes at the ends of the retaining structure. The inspector shall note any erosion or loss of fill due to drainage at the ends of the walls. The inspector shall note any remediation recommendations under the maintenance items on the inspection report as well as noting the assessment state of the Drainage Approach. This assessment is quantified as 1 EA for each end of the wall. For unit walls, the inspector shall note this fact, and evaluate the condition of the sidewalk/soils at the walls' interface.

Figure 4.4.5-1: Concrete Flume at End of Wall. Condition State 2 (Fair) – No Erosion Evident, with wide crack.

Refer to the following condition states:

- ***Condition State 1 (Good)***: No slope erosion is evident off the ends of the wall or in the associated ditches.
- ***Condition State 2 (Fair)***: Minor erosion of slopes around the wall. Drainage systems at the end of the wall are plugged or have minor deterioration.

- ***Condition State 3 (Poor)***: Moderate erosion of slopes around the bridge. Drainage systems at the end of the wall are plugged or have moderate deterioration.

- ***Condition State 4 (Severe)***: Major erosion of wall end slopes. Drainage systems at the end of the wall are plugged and have major deterioration.

Deficiencies shall be reported under the assessment notes and state the type, size and location of the defects. This assessment is quantified as 1 EA for each end of the structure.

4.4.5.2 Aesthetic Treatments (9010)

Similar to the bridge inspection assessment, this item is used to evaluate the condition of all the aesthetic treatments located on the retention structure. In certain settings, a structure may contain several aesthetic components, from concrete staining, stamped concrete façade, obelisks, etc. It is necessary for an inspector to review the plan set to determine the type of structure being inspected to determine whether the façade is an aesthetic feature or integral structural component. For instance, secant walls are comprised of main load retaining secant piles typically installed behind precast concrete paneling. During an inspection only the precast paneling can be inspected. The inspector shall use visible distress in the precast panels to determine the condition of the secant piling behind. A similar inspection would be performed with an MSE wall. In this case, any distress noted in the paneling would be noted in the secant pile defects. Any stained concrete or form-lined features would be accounted for under the Aesthetic Treatments assessment.

Figure 4.4.5-2: Failed Concrete Stain.

Figure 4.4.5-3: Aesthetic Treatment Form-Lined Precast Concrete Panels.

Refer to the following condition states:

- ***Condition State 1 (Good)***: System is in good condition, with no notable issues.
- ***Condition State 2 (Fair)***: Aesthetic system is in fair condition, with some fading or discoloration. Minor issues.
- ***Condition State 3 (Poor)***: Aesthetic system is in poor condition, with significant fading or discoloration.
- ***Condition State 4 (Severe)***: Aesthetic system is in severe condition and is not functioning as intended.

Aesthetic deficiencies shall be reported under the assessment notes and state the size and location of the defect. This assessment is quantified as 1 EA for the entire structure. It is the inspector's discretion, based on the assessment's condition over the structure as a whole, to codify the appropriate assessment state.

4.4.5.3 Utilities (9011)

Any utilities noted on the retaining earth structure, not including sign structures. This may include the quantification and evaluation of light poles, luminaires, electrical conduits, or other mechanical/electrical devices that may be attached to or in the earth retaining structure. Drainage pipes, inlets and outlets shall be assessed under 9340 Drainage System.

Refer to the following condition states:

- ***Condition State 1 (Good)***: Utility is in excellent condition, no problems noted.

- ***Condition State 2 (Fair)***: Utility is in fair condition. Some minor problems are noted, but they do not affect the serviceability of the utility.

- ***Condition State 3 (Poor)***: Utility is in poor condition, with moderate problems.

- ***Condition State 4 (Severe)***: Utility is in severe condition. Failures have occurred.

Material deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for an individual component/system. Conduit packages may be quantified as 1 EA or as each individual conduit within the package. It is at the inspector's discretion as to quantify the package or individual conduits. Regardless, the inspector shall note the utilities present on the retaining structure within the assessment notes.

4.4.5.4 Signs Other (9035)

All signs located on the retaining earth structure shall be accounted for, quantified and evaluated under this assessment. This assessment does not includes the Name ID Plaque typically located at the end of a wall. Refer to section 4.4.5.5 Structure ID Plaque (9208) for Name ID plaque assessment. Furthermore, this assessment does not include the quantification or evaluation of any signs located on a sign structure that may be founded on the retaining earth structure.

Refer to the following condition states:

- ***Condition State 1 (Good)***: Sign is present and is in good condition (there may be superficial damage or deterioration).

- ***Condition State 2 (Fair)***: Sign is present – sign may have some damage or deterioration (slightly bent or fading), but remains readable.

- ***Condition State 3 (Poor)***: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

- ***Condition State 4 (Severe)***: Sign is absent, or incorrect, or existing sign is damage or deteriorated to the extent that repair or replacement is required as soon as possible.

Material deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for each sign, not including sign structure signs, observed on the structure.

4.4.5.5 Structure ID Plaque (9208)

All new retaining walls should have a name plaque located on the visible face of the wall at an approach end. This assessment shall be utilized to note the location and condition of the plaque.

Refer to the following condition states:

- ***Condition State 1 (Good)***: Sign is present and is in good condition (there may be superficial damage or deterioration).

- ***Condition State 2 (Fair)***: Sign is present – sign may have some damage or deterioration (slightly bent or fading), but remains readable.

- ***Condition State 3 (Poor)***: Sign is present, but is deteriorated to the point that replacement or repair should be considered in next inspection cycle.

- ***Condition State 4 (Severe)***: Sign is absent, or incorrect, or existing sign is damage or deteriorated to the extent that repair or replacement is required as soon as possible.

Material deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA.

Figure 4.4.5-4: Retaining Wall Name ID Plaque.

4.4.5.6 Decorative Rail (9335)

All extensions fastened to a bridge railing element or horizontal coping on a retaining earth structure and not considered comprised of chain-link fencing shall be quantified and evaluated under the Decorative Rail assessment.

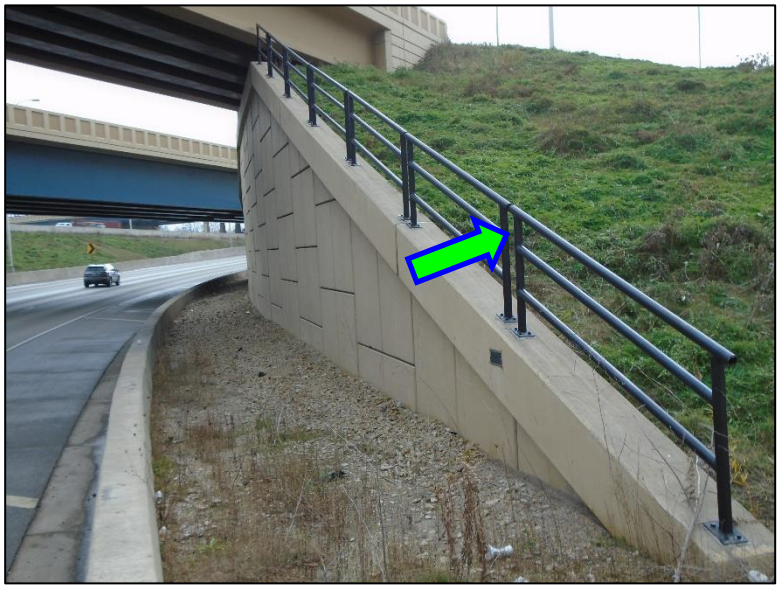


Figure 4.4.5-5: Decorative Rail atop a Horizontal Coping.

Figure 4.4.5-6: Decorative Rail atop a Bridge Rail Element.

Refer to the following condition states:

- ***Condition State 1 (Good)***: Rail has little or no deterioration. Galvanizing or protective coating is sound.
- ***Condition State 2 (Fair)***: Rail has minor deterioration. Coating may have minor failure – surface rust may be present.

- ***Condition State 3 (Poor)***: Rail has moderate deterioration. Coating may have moderate failure – surface rust may be prevalent. Components may be slightly bent or misaligned – connections may be slightly loose.

- ***Condition State 4 (Severe)***: Rail has extensive deterioration. Coating may have extensive failure – there may be section loss. Components may be bent or misaligned connections may be loose.

Material deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for the entire length of the structure. It is the inspector's discretion, based on the assessment's material condition over the whole length, to codify the appropriate assessment state.

4.4.5.7 Luminaire Bases (9336)

Luminaire bases are the bump-outs located along a retaining earth structure that support vertical light posts. These will typically poured monolithic with the bridge railing element atop the horizontal copings. Note that these bases do not include sign structure foundation bump outs. Sign structure bump outs are inspected under a separate inspection for the sign structure itself.

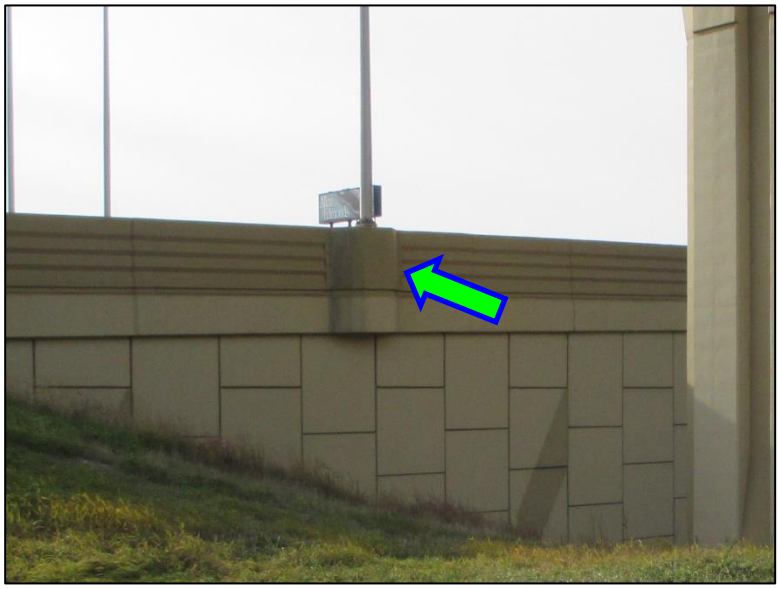


Figure 4.4.5-7: Luminaire Base Placed in Coping and Barrier Wall.

Refer to the following condition states:

- ***Condition State 1 (Good)***: Good condition, with no problems noted.

- ***Condition State 2 (Fair)***: Fair condition, with superficial spalls and/or cracking.

- ***Condition State 3 (Poor)***: Moderate deterioration, with cracking and spalls.

- ***Condition State 4 (Severe)***: Base has failed. Major deterioration noted.

Material deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for each base located along the structure. It is the inspector's discretion, based on the assessment's material condition of each base, to codify the appropriate assessment state.

4.4.5.8 Protective Screening (9337)

In contrast to the Decorative Rail assessment, the Protective Screening assessment shall be utilized for any extension above a bridge railing element or coping that is comprised of chain-link fencing. This is regardless if the screening is the active barrier between the leading edge of a sidewalk or simply on top of the coping/railing.

Figure 4.4.5-8: Protective Screening Fastened to a Horizontal Coping. Note the Screening is Not Adjacent to a Sidewalk.

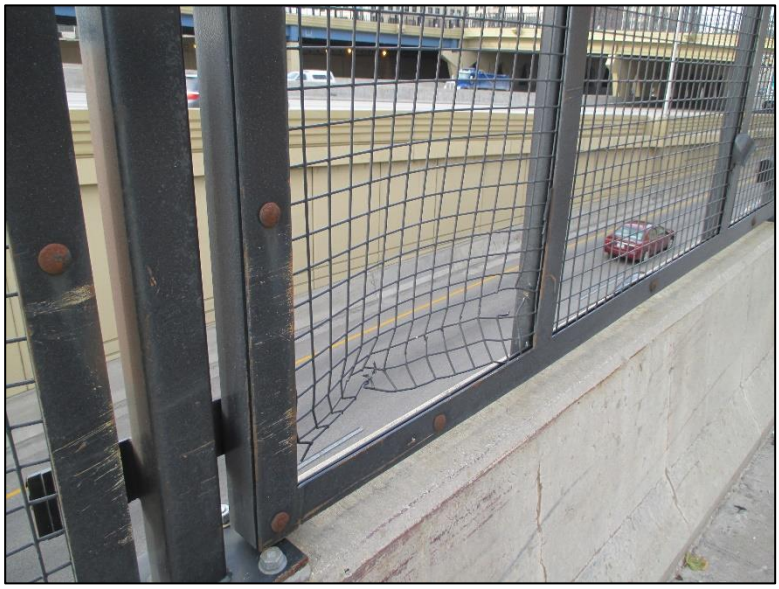


Figure 4.4.5-9: Assessment State 3 Protective Screening atop Bridge Rail. Note Chain-link Fence Deformation but Function Remains.

Refer to the following condition states:

- ***Condition State 1 (Good)***: Chain-link fence has little or no deterioration. Galvanizing or vinyl coating is sound.

- ***Condition State 2 (Fair)***: Chain-link fence has minor deterioration. Coating may have minor failure- surface rust may be present. Fence components are properly aligned (all connections are sound).

- ***Condition State 3 (Poor)***: Chain-link fence has moderate deterioration. Coating may have moderate failure – surface rust may be prevalent. Components may be slightly bent or misaligned – connections may be slightly loos. Fabric may have snags or holes (areas may be slightly stretched or deformed).

- ***Condition State 4 (Severe)***: Chain-link fence has extensive deterioration. Coating may have extensive failure – there may be section loss. Components may be bent or misaligned – connections may be loose. Fabric may have numerous snags or holes (areas may be stretched or deformed).

Material deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for the entire length of the structure. It is the inspector's discretion, based on the assessment's material condition over the whole length, to codify the appropriate assessment state.

4.4.5.9 Horizontal Copings (9338)

This assessment captures the deficiencies associated with the component that is commonly found along the top of a retention structure. This assessment may contribute to the aesthetics of the structure, however it also serves the structural purpose of tying adjacent panels together

at the top of the wall and providing a sound base for placing bridge railing elements. The coping provides protection from deterioration caused by water runoff, snow, freeze/thaw and ice by diverting water away from vertical seams, the top of the wall, and the wall facing. These assessments will typically be comprised of cast-in-place concrete or in some instances precast concrete.

Figure 4.4.5-10: Cast-In-Place Horizontal Coping

Refer to the following condition states:

- ***Condition State 1 (Good)***: No issues. If cracks exist, they are <1/16" in width or sealed.

- ***Condition State 2 (Fair)***: Delaminations or spalls 1" or less, or less than 6" diameter. Patches, if they exist, are sound. HL or narrow cracking may be present. Minor efflorescence may exist, but no rust staining. Abrasion, if present, is minor.

- ***Condition State 3 (Poor)***: Spall greater than 1" deep, or 6" diameter. Patched areas are unsound or showing distress. Medium width cracks. When efflorescence is present, there is heavy build-up and/or rust staining. Abrasion, if it exists, is moderate.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall; or a structural review has been completed and the defects impact the strength and/or serviceability of the wall.

Material deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified in lineal feet LF for the entire length of the structure. It is the inspector's discretion, based on the assessment's material condition over the whole length, to codify the appropriate assessment state for the unit of measure.

4.4.5.10 Features (Roadway/Sidewalk/Etc.) to Wall (9339)

Typically, fill and cut section walls will have roadways, sidewalks, driveways or other facilities adjacent or carried by the wall. Regardless if these features are along the face or along the top of the wall, the inspector shall perform a visual inspection of the features paying particular attention to any settlement, joint separation or excessive cracking. These deficiencies may indicate wall movement or wall distress in the adjacent wall. The slope behind the wall should be stable and adequately supported by the wall. Any backfill losses, indication of water retention, or other defects may indicate performance issues with the wall and should be noted.

Figure 4.4.5-11: Concrete Sidewalk Along Top of Retaining Wall.

Figure 4.4.5-12: Pothole Due to Loss of Fill Within Shoulder Gutter Pan Along Top of Retaining Wall.

Refer to the following for condition states:

- ***Condition State 1 (Good)***: No issues with retained material. The facilities, if they exist, are generally smooth and show no signs of settlement.

- ***Condition State 2 (Fair)***: Settlement exists but within tolerable limits with no structural distress observed. The facility, if they exist, may have minor settlement and/or may be cracked and deteriorated.

- ***Condition State 3 (Poor)***: Settlement exists that exceeds tolerable limits with no structural distress observed, nor structural review required. The facilities, if they exist, may have moderate deterioration and/or may be cracked and deteriorated.

- ***Condition State 4 (Severe)***: The condition warrants a structural review to determine the effect on strength or serviceability of the wall; or a structural review has been completed and the defects impact the strength and/or serviceability of the wall.

Only quantify the assessment if the facility has a direct impact on the wall. Deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for along the top of the wall and 1 EA for along the face of the wall. It is the inspector's discretion, based on the assessment's material condition over the whole length, to codify the appropriate assessment state.

4.4.5.11 Drainage System (9340)

This assessment defines the drainage along the retaining wall, both on top and along the face. Typically systems to note during inspection include vegetation or lack thereof along the wall and whether run off is able to properly drain away or through the wall. Storm sewer pipes including inlets or field inlets along the wall should be inspected and noted if clogged. Debris preventing adequate drainage should be noted or removed by the inspector. The inspector should note any settlement of fill along or above the wall. Settlement localized in the vicinity of storm sewer inlets and outlets may be an indication that the drainage system is not functioning properly or may be broken and eroding fill material away from behind the wall.

The inspector should assess and evaluate fill settlement and erosion in front and along the top of the wall under 9340 Drainage System.

Figure 4.4.5-13: Drainage System Along Top of Retaining Wall. Note Vegetation Exhibits No Signs of Erosion or Settlement.

Figure 4.4.5-14: Drainage System Located along Top of Retaining Wall.

Refer to the following assessment states for Drainage System

- ***Condition State 1 (Good)***: Drainage systems are functioning properly. No slope erosion is evident, nor are any signs of settlement of the slope.

- ***Condition State 2 (Fair)***: Minor erosion of slope. Drainage system is plugged or has minor deterioration.

- ***Condition State 3 (Poor)***: Moderate erosion of the slope. Drainage system is plugged or has moderate deterioration. Minor slope failures have occurred either removing or adding material from the wall area.

- ***Condition State 4 (Severe)***: Major erosion of slopes. Drainage system is plugged and has major deterioration. Substantial slope failures have occurred either removing or adding material to the wall area.

Deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for along the top of the wall and 1 EA for along the face of the wall. It is the inspector's discretion, based on the assessment's material condition over the whole length, to codify the appropriate assessment state.

4.4.5.12 Stairwell (9341)

This assessment defines the treads, risers, nosing, and hand rails that make up the stairway within a retaining wall structure. The sides of the stairwell will be evaluated as part of the retaining wall element.

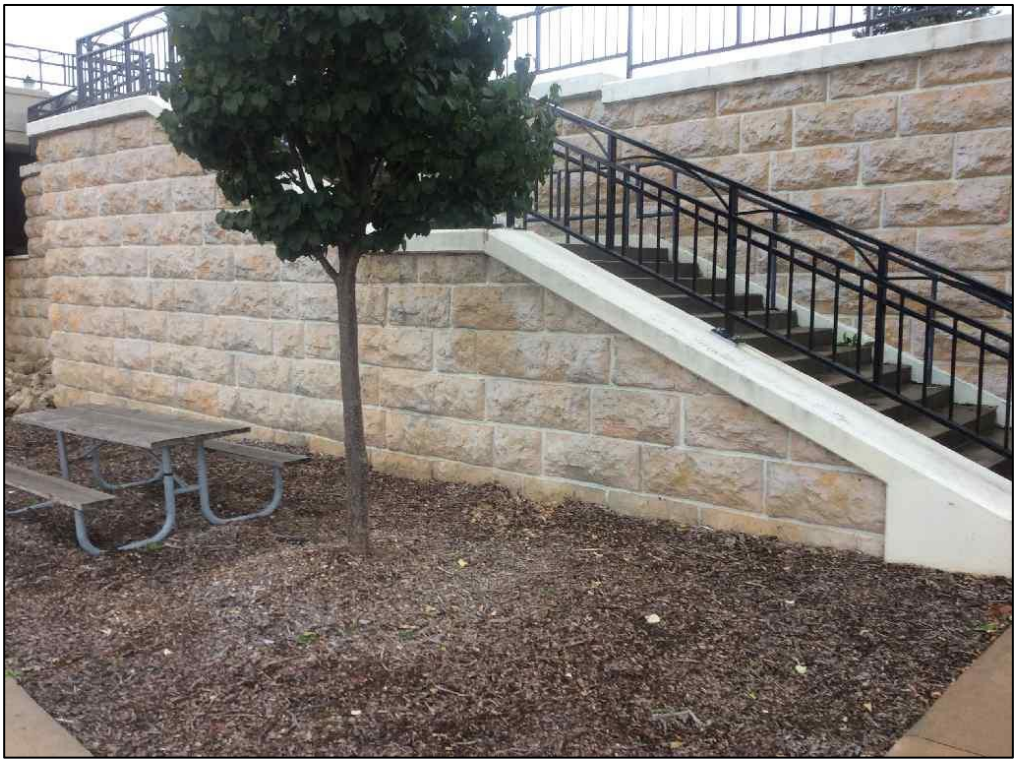


Figure 4.4.5-5: Stairwell integral to Retaining Wall.

Refer to the following assessment states for Stairwell:

- ***Condition State 1 (Good)***: Good condition without defects and functioning as intended. Stairwell may have superficial or cosmetic cracks, spalls, or stains.

- ***Condition State 2 (Fair)***: Fair condition with superficial or cosmetic cracks, spalls, stains, or light corrosion; defects are present, but are not a structural concern.

- ***Condition State 3 (Poor)***: Moderate deterioration or defects that may be a structural issue such as settlement, heaving, tipping, misalignment, scaling, heavy corrosion, etc. Early signs of structural defects.

- ***Condition State 4 (Severe)***: Stairwell has major deterioration or defects that are structurally concerning. Evaluate and close stairwell, as necessary, to restrict access.

Deficiencies shall be reported under the assessment notes and state the type, size and location of the defect. This assessment is quantified as 1 EA for each stairwell. It is the inspector's discretion, based on the assessment's material condition over the whole length, to codify the appropriate assessment state.

4.4.6 Overall Wall Condition Rating

Upon completion of a wall inspection, the inspector is tasked with assigning an overall condition rating to the structure. This is a global evaluation and is used to determine inspection interval and other asset management functions. Therefore the inspector must take into account all elements and assessments noted during the inspection and the functionality of the entire structure. There are four rating levels. The following performance definitions should be reviewed to aid the inspector in assigning the appropriate overall rating for a wall:

- ***Good***: No, or very low distress observed in the wall elements and assessments. Defects are minor, and within the normal range for newly constructed or fabricated elements. Highly functioning wall that is only beginning to show the first signs of distress or weathering.

- ***Fair***: Overall, the condition is satisfactory. Distress is present in wall elements and/or assessments, but does not compromise the wall function. Localized drainage issues, settlement, staining, washing of fines from backfill material that are minor.

- ***Poor***: Overall condition of the wall is poor. Distress is present, but does not pose an immediate threat to wall stability and closure of facilities adjacent to structure is not necessary. Repair and/or replacement is needed in the near future.

- ***Severe***: Critical condition. Major structural defects, or components have rotation, sliding, settlement, and/or overturning that is close to possible collapse. Wall is no longer serving the intended function, or is unstable and needs repair/replacement as soon as possible. Facilities adjacent to wall may need to be closed.

When attempting to rate a significantly long wall (greater than 500 ft) it is important for the inspector to note the functionality of the wall as a whole. Areas of localized severe material distress that do not appear to be affecting the function of the structure will not have as severe an impact on the overall rating. Only when material defects eventually result in large scale wall deterioration, movement or loss of fill from behind the wall should those localized areas be taken into such large consideration when assigning the overall rating.

4.4.7 Maintenance & Repair Items

Upon completion of the inspection report the inspector is relegated to determining the appropriate maintenance & repair actions that should be performed on the structure in order to keep it functioning properly. This includes determining the priority of each particular action. Each maintenance item priority is viewed by the structure owner and provides information for budgetary purposes.

Maintenance activities include items that are of a cyclic or reoccurring nature such as cleaning drains, removing debris, replacing dislodged chinking, painting soldier piles, sealing concrete, etc. Repair activities include non-routine fixing and restoring of wall elements to their intended function: resetting dislodged stonework, repointing stone masonry, grading/reseeding adjacent slopes, patching concrete spalls, mending damaged wire baskets, etc.

Determining the priority level for each item depends significantly on how the functionality of the wall is impacted. Those areas of distress that have a minimal impact should be categorized under a low or medium priority while those areas of distress that are or are eminently impacting the functionality of wall, such as measureable wall movement or global wall failure, should be set at high or critical priority. The following table describes the repair timeline associated with each priority level.

Priority Level	Timeline Expectations
Low	Repair prior to next inspection, as funding allows.
Medium	Repair within one year as funding allows.
High	Repair within 90 days.
Critical	Repair within the timeline specified by the inspector in the notes, but not to exceed 30 calendar days.

The following is the list of all available Maintenance, Repair, and Monitoring Items at the inspector's disposal:

Category	Repair/Maintenance Item	Default Priority
Aesthetics	Aesthetics – Apply Anti-Graffiti Coating	Low
	Aesthetics – Other Work	---
	Aesthetics – Power Wash	Medium
	Aesthetics – Re-paint/ Re-stain	Medium
	Aesthetics – Remove Graffiti	Medium
Drainage	Drainage – Clean Inlets	Medium
	Drainage – Fill Voids in Retained Material	High
	Drainage – Install Riprap/ Geotextile Fabric	Medium
	Drainage – Install/Replace Weep Hole	Medium
	Drainage – Repair Erosion/Scour	Medium
	Drainage – Repair or Replace Inlets	Medium
	Drainage – Repair/Reconnect Drain Tile	High
	Drainage – Seal Horizontal Joints or Cracks	Medium

	Drainage – Seal Vertical Joints or Cracks	Medium
	Drainage – Tuckpoint Inlets	Medium
	Drainage – Unclog Weep Hole	High
	Drainage – Uncover Drain Tile Outfall	Medium

Category	Repair/Maintenance Item	Default Priority
Improvement	Imp – Rehab/Major Repair	---
	Imp – Replace Structure	---
Miscellaneous Maintenance	Misc. – Cut Brush	Low
	Misc. – Other Work	---
	Misc. – Remove Vegetation (Spray)	Low
	Misc. – Remove/Monitor Loose Concrete	Medium
	Misc. – Repair/Replace Fence or Railing	Medium
	Misc. – Sign Remove/Reset/Replace	Low
	Misc. – Utility Repair/Notify Utility	Low
Structural	Structural – Joint Repair or Replacement	Low
	Structural – Other Work	---
	Structural – Patch Concrete Delams/Spalls	Low
	Structural – Repaint Steel Posts or Connections	High
	Structural – Repair Footing	High
	Structural – Repair Timber	High
	Structural – Repair/Replace Steel Connections/Anchors	High
	Structural – Replace Damaged Wall Panel	Critical
	Structural – Replace Deteriorated Modular Block	Medium
	Structural – Replace Shims/ Reset Panels	High
	Structural – Replace Tie-Back	Critical
	Structural – Reset Displaced Masonry/Block	Medium
	Structural – Stabilize Wall	Critical
	Structural – Tighten/Replace Bolts and Nuts	Medium
	Structural – Tuck Point Masonry	Medium
Investigating or Monitoring	Monitor – MSE Wall Settlement	---
	Monitor - Rotation/Tipping	---
	Monitor – Steel Corrosion / Section Loss	---
	Investigate - Cause of MSE Fill Loss	---
	Investigate – Recommend In-Depth Inspection	---
	Investigate – Recommend NDT or MIC Testing	---

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4.5 NOISE BARRIERS

4.5.1 Introduction

Noise barriers have been in use in the United States since 1963 and in Wisconsin since 1983. Most noise barriers have been assigned a unique structure number, similar to how a bridge is assigned a number. Noise barrier structure numbers are identified with the letter 'N' although some of the older structures were originally identified with a 'SN' prefix. Refer to Part 1, Chapter 5 for discussion on identification numbering of noise barriers.

A noise barrier is a constructed appurtenance, either alone or integrated with other systems that alter the normal noise travel at a site. The purpose of a noise barrier inspection is to assess the functionality of the barrier (that is, the ability to actually reduce noise transmission volume and projection from the roadway) and to ascertain the condition of the structure for maintenance and repair needs to prevent local or global structural failure. Very minor breaches in the wall substantially reduce its effectiveness to reduce traffic noise. Therefore the inspector should note any issues with material deterioration, joint separation or other distress that could adversely affect the performance of the wall. Oftentimes noise barriers or portions thereof will be installed on other systems, such as bridge railings, or be placed along property lines without immediate access to one or both sides. Consequently, general maintenance or repair work of these barriers may require extensive preplanning including traffic control and permitting.

Noise barriers are generally classified into two categories: Ground-Mounted and Structure-Mounted. The required inspection interval for Structure-Mounted and Ground-Mounted Noise Barrier Walls is not to exceed six (6) calendar years. Further information on inspection interval can be found in Part 4, Chapter 1 of this manual.

When a noise barrier is present on a supporting bridge or retaining wall structure, and an inspection of that supporting structure is being conducted, the inspector shall perform a cursory inspection of the noise barrier as part of the bridge or retaining wall inspection. If any significant distress is observed, the owner of the noise barrier structure should be contacted.

There are three subcategories for both Structure-Mounted and Ground-Mounted noise barriers: Double-Sided Sound Absorptive Noise Barriers; Single-Sided Sound Absorptive Noise Barriers; and Reflective Noise Barriers. Double-Sided Noise Barriers absorb sound on both sides of the wall, Single-Sided Noise Barriers absorb sound on only one side, and Reflective Noise Barriers reflect any sound transmitted towards them.

This chapter will discuss the different types of noise barriers and specific characteristics common to them, modes of failure, and inspection methodology.

4.5.1.1 Ground-Mounted Noise Barriers

Ground-mounted noise barriers are appurtenances founded on soil. There are three basic types in this category: Noise Barrier Berms; Noise Barrier Walls; and a combination of the two.

Noise Barrier Berms

Noise barrier berms are earthen mounds constructed on-site from natural materials such as soil, stone, rock, rubble, etc., and are constructed in an unsupported manner according to their

naturally stable slope. Noise barrier berms typically require more width than noise barrier walls due to the required slope of the sides, which needs to be gradual enough to maintain stability. Noise protection is typically warranted in more urban settings. Consequently, the available footprint for a structure is limited due to the higher cost associated with it. This results in the construction of a Noise Barrier Wall rather than Noise Barrier Berm in nearly all instances in Wisconsin. WisDOT does not inventory or inspect Noise Barrier Berms. Refer to Figure 4.5.1-1 for a view of a noise barrier berm.

Figure 4.5.1-1: Noise Barrier Berm between Residential Buildings (Foreground) and a Roadway (Background).

Noise Barrier Walls

A noise barrier wall is typically a designed-cantilever structure. The components are typically prefabricated off-site and assembled on-site. A variety of noise barrier wall designs exist including: post & panel; brick & masonry block; freestanding noise walls; direct burial panels; noise walls as earth retention structures; and cast-in-place concrete walls. Refer to Figure 4.5.1-2 for a view of a noise barrier wall.

Figure 4.5.1-2: Ground Mounted Metal Panel Noise Barrier Wall.

Combination Noise Berm & Barrier Wall

Many systems use a combination of types to obtain the required height of the noise barrier. Noise walls are frequently erected on top of a noise berm. Refer to Figure 4.5.1-3 for a view of a berm and noise barrier combination structure.

Figure 4.5.1-3: Combination Noise Barrier Berm and Wall.

4.5.1.2 Structure-Mounted Noise Barrier Walls

Structure-mounted noise barrier walls are systems attached to a structure, or integrally constructed with the structure. Noise barrier walls are frequently mounted on bridges and

retaining walls. For bridge inspections, it is the inspector's task to identify all non-'N' numbered noise barriers, and record them as Element 8610 Noise Barrier – Structure Mounted on the bridge inspection form. Refer to Section 4.5.3 of this manual for further information on Noise Barrier Elements. If the noise barrier has an 'N' number assigned to it, then the noise barrier will not be recorded on the supporting structure inspection form, but will be properly inventoried and inspected under its own report and inspection interval. The inspector shall, however, note within the 'Structure Specific Notes' of the supporting structure report the 'N' number structure, where its located and perform a cursory inspection of the noise barrier and note them within this section. If significant defects affecting the functionality of the barrier or the safety of the travelling public are discovered, the owner of the noise barrier shall be notified immediately. Non 'N' numbered noise barriers will not be assigned 'N' numbers in the future. Therefore, the inspector is required to include non 'N' numbered structure-mounted noise barriers within the supporting structure inspection report with the appropriate element and defects. Refer to Figure 4.5.1-4 through Figure 4.5.1-6 for views of structure-mounted noise barrier walls.

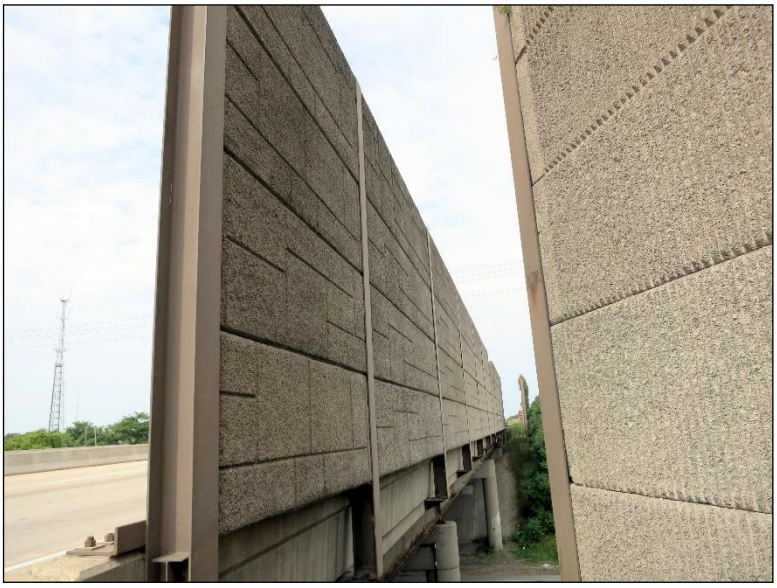

Figure 4.5.1-4: Precast Panel Noise Barrier Wall Mounted on a Bridge.

Figure 4.5.1-5: Precast Concrete Panel Noise Barrier Wall Mounted on a Retaining Wall.

Figure 4.5.1-6: Precast Concrete Noise Barrier Wall Mounted on Concrete Vehicular Crash Barrier.

4.5.2 Noise Barrier Wall Inspection

Noise barriers are inspected using elements and assessments, as described in the following section. Refer to Section 4.6.3 for guidance on the elements and Section 4.5.6 for guidance on assessments. Common defects will be covered in this section.

Element Level Inspection

All noise barrier wall elements are recorded in units of linear feet. Each linear foot includes the top and both sides of the vertical projection of the exposed height of the wall element. This area is inspected and evaluated for any distress with deficiencies captured and quantified under the appropriate noise wall defects. Refer to the next section for the available defects for these elements. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

The same inspection process is conducted for any remaining elements along the noise barrier wall. The inspector must also record and assess the conditions of all existing assessments on the wall. Refer to Section 4.5.6 Assessments for WisDOT noise barrier wall assessments, definitions and assessment states.

The inspector is to carefully note the type, location and measurement of any area exhibiting Condition State 3 or 4 defects. An associated photograph and/or sketch shall be provided within the report for all CS3 and CS4 defects.

Due to the similar material composition as retaining walls, the following list may be used when inspecting noise barrier walls to verify proper material function and wall stability.

- Obtain minimum edge-of-lane to wall distance.
- Verify set elevations along the face of the wall for signs of settlement.
- Inspect the vertical alignment of the wall with a plumb-bob. Most noise barrier walls should be vertical (verify with plans).
- Examine the opening of the construction joints between sections of the wall.
- Inspect joints near ground line for any fill material washing out from between or below the panels.
- Inspect panel joints for differential movement or rotation. Sight down panel face to note individual rotation or tipping out of plane.
- Inspect for erosion of the embankment material in front and in back of the wall.
- Inspect for heaving of the embankment material in front and in back of the wall.
- Inspect for settlement of the fill material along the wall.

- Examine the wall for deterioration of the material, such as cracking, spalling, corrosion, discoloration, etc. noting the width, length, depth, and/or orientation of the deterioration. Provide photographs for Condition State 3 and 4 defects.
- Check wall for evidence of efflorescence or rust staining.
- Examine Panel connections & frame, if applicable.
- Examine post base and anchorage systems if present. Fasteners and connections should be checked for tightness and distress.
- Note any varmint holes around foundation or footing base.
- Note condition of fire hydrant holes or access holes, if applicable.
- Examine and probe drains within the vicinity of the wall for signs of clogging.
- Examine site grading for any locations that may prohibit proper drainage along the wall.
- Inspect sidewalk or roadway components along the wall for signs of joint separation, potholes and areas of settlement which may indicate a more global impact on the noise barrier system.
- Examine vegetation growth along the wall. Root infiltration may create undesirable stresses on the wall and may induce cracking or failure if left untreated.
- Note previous inspection interval and recommend inspection interval.
- Examine the I.D. plaque, if applicable. Note location within the Structure Specific Notes on the inspection report.

Element Defects

WisDOT simplifies noise barrier wall distress into three defects, one concerning wall material defects and two with wall or panel movement. The defect 8903 Wall Deterioration essentially acts as an umbrella defect which refers the inspector to the unique defects of the primary material of which the wall is constructed. While there is only one material defect to code within the inspection report, it is highly recommended the inspector note the specific material defect observed and measured under the defect note. For all non-post and panel noise walls, any lateral, vertical or rotational movement shall be coded under the defect 8902 Wall Movement. Post and panel walls exhibiting movement shall be captured under the defect 1640 Masonry or Panel Displacement. Refer to the following sections for wall defects and condition state definitions. The inspector should refer to sections 4.5.4 Wall Panel Deterioration and 4.5.5 Wall Panel or Post Movement for material deterioration and potential failure modes of noise barrier walls, respectively.

The inspector shall utilize the Defect Hierarchy concerning overlapping defects within a unit of measure when rolling up the total element condition state quantities. Refer to Part 2 Chapter

3 for the Defect Hierarchy. However, only the controlling defect will be counted in the total element condition state quantity.

4.5.2.1 Wall Movement (Defect 8902)

This defect pertains to all wall types with the exception of Post and Panel Noise Barrier Walls and captures the global movement of the wall. Movement includes lateral movement (whether in or out along the plane of the wall), differential settlement or global settlement. The major function of all noise walls is to effectively redirect or absorb sound in order to allow a quieter atmosphere on the opposing side of the wall. Therefore, movement can disrupt this redirection or absorption or may be an indication that a portion of the wall may fail resulting in functional and potential safety concerns. Any movement must therefore be recorded, measured and monitored.

Refer to the following for condition state descriptions:

- Condition State 1: None. No movement is observed or measured.
- Condition State 2: Differential movement has started to occur. Wall may be strapped to prevent further movement, or movement has been arrested through countermeasures.
- Condition State 3: Wall rotation/sliding/settlement is active; if a retaining wall, then sloughing of retained material behind wall is evident. Wall may have been strapped or stabilized to prevent further movement, but this device has failed.
- Condition State 4: Wall has failed.

It is important to note that these condition states are for the linear foot unit of measure. Therefore condition states 3 and 4 shall have representative measurements and photographs within the report.

4.5.2.2 Wall Deterioration (Defect 8903)

This defect reflects the localized material breakdown of the major wall components. It describes the material flaws, defects, etc. in the overall wall system. This defect refers the inspector to the specific material defects comprising the wall. That is, when inspecting reinforced concrete wall components, the inspector shall refer to the reinforced concrete defects, or when inspecting a steel panel noise barrier wall, the inspector shall refer to steel defects. All material defects can be found in the WisDOT Field Manual or within Part 2 Appendix A. Refer to the following for condition state descriptions:

Refer to the following for condition state descriptions:

- Condition State 1: The wall material has deterioration described in the applicable CS1 material defects of section loss and wall integrity.
- Condition State 2: The wall material has deterioration described in the applicable CS2 material defects for section loss and wall integrity.

- Condition State 3: The wall material has deterioration described in the applicable CS3 material defects for section loss and wall integrity.

- Condition State 4: The wall material has deterioration/section loss that has caused the wall to fail and no longer performs the intended function.

It is important to note that these condition states are for the linear foot unit of measure. Therefore condition states 3 and 4 shall have representative measurements and photographs within the report.

4.5.2.3 Masonry or Panel Displacement (Defect 1640)

This defect shall be available only for post and panel noise barrier walls. It describes panels and their movement in relation to the overall wall. Therefore, when individual panels are observed to move, the inspector shall use Masonry or Panel Displacement defect to describe movement rather than Defect 8902. Refer to the following for condition state descriptions:

Refer to the following for condition state descriptions:

- Condition State 1: None. No movement is observed or measured.

- Condition State 2: Block, stone or panel has shifted slightly out of alignment.

- Condition State 3: Block, stone or panel has shifted significantly out of alignment or is missing but does not warrant a structural review.

- Condition State 4: The condition warrants a structural review to determine the effect on strength or serviceability of the element, or a review has been completed and it has been found that the defects impact strength or serviceability.

It is important to note that these condition states are for the linear foot unit of measure. Therefore condition states 3 and 4 shall have representative measurements and photographs within the report.

4.5.3 Noise Barrier Wall Elements

Two noise barrier elements are available during an inspection: Noise Barrier Wall – Ground Mounted (8609) and Noise Barrier Wall – Structure Mounted (8610). Depending on the construction of the wall, the inspector shall choose the appropriate noise barrier wall element and perform an inspection of all exposed surfaces of the element recording all defects and attached assessments along its length.

4.5.3.1 Noise Barrier Wall – Ground Mounted (Element 8609)

Noise walls that are not attached to other systems are considered ground mounted.

Figure 4.5.3-1: Ground Mounted Noise Wall.

4.5.3.2 Noise Barrier Wall – Structure Mounted (Element 8610)

Noise walls that are attached to other systems are considered structure mounted. If the noise barrier is attached to the bridge and has an 'N' number assigned to it, then the noise barrier will not be recorded on the bridge inspection form. However, it is the inspector's due diligence to perform a cursory inspection of the noise barrier to ensure that the travelling public is not threatened by any notable deterioration of the noise barrier. Any areas of concern can be captured as maintenance items, but would not have the element associated with it.

Figure 4.5.3-2: Structure Mounted Noise Wall. Mounted to Retaining Wall Structure.

4.5.3.3 Bridge Rail (Elements 330-334)

Noise walls may have or may be mounted to bridge rail elements. As part of a structure mounted system, these rail elements should already be inspected under the appropriate structure inspection (either a 'B' or 'R' numbered inspection). Therefore, structure mounted 'N' numbered noise barriers need not have bridge rail elements within the inspection report. However, similar to 'N' number structures on support structures, the inspector shall note the bridge rail element within the 'Structure Specific Notes' portion of the report and note any distress or deterioration that may adversely affect the noise wall. In addition, if it is deemed the distress is significant enough to encroach upon the safety of the travelling public, the owner of the bridge railing element shall be notified.

For non-'N' numbered noise barrier wall elements on bridge railing elements, the inspector shall include the appropriate bridge rail element and evaluate its condition along its length similar to a bridge railing element inspection under Part 2 of the Structure Inspection Manual. Refer to Part 2.6.7 Traffic Safety Features for Bridge Rail Element definitions and defects.

4.5.4 Wall Panel Deterioration

Failure of the construction material is frequently observed at older noise barrier structures. The following section is a brief outline of material defects typically found during inspections. Many of the types of defects associated with noise barriers are the same as those associated with retention structures. As such, the discussion of material defects will be relatively similar. Refer to Part 2, Section 1.1.4 of this Manual for a more in-depth description of the defects and causes associated with these materials.

4.5.4.1 Impact Damage

Impact damage is unique to no one noise barrier wall material. Impact damage is the result of a collision between a moving object and the noise barrier structure. This is typically the result of an errant vehicle but may also occur due to forces generated by a severe storm, explosion, or similar such phenomena hurling objects into the wall.

Impact damage is typically confined to an area directly adjacent to the point of contact and is characterized by distortion or crushing of the construction material at the point of contact with cracking, splitting, or splintering radiating from this point. Refer to Figure 4.5.4-1 through Figure 4.5.4-3 for views of impact damage.

Figure 4.5.4-1: Impact Damage to a Noise Barrier Wall.

Figure 4.5.4-2: Impact Damage to a Noise Barrier Wall.

Figure 4.5.4-3: Impact Damage with Failed Post and Paneling of Structure Mounted Noise Wall.

4.5.4.2 Masonry Noise Barrier Walls

The following is a discussion of different types of material defects that may be found when inspecting masonry units. Refer to Figure 4.5.4-4 for an overall view of a masonry noise barrier wall. Refer to Part 4, Section 4.3.1 for photographs of typical Masonry deterioration.

Figure 4.5.4-4: Typical Masonry Noise Barrier Wall.

- **Construction Defects:** Construction flaws are often characterized by damage to the blocks that appear to be due to poor construction technique or errors. Examples might

include evidence of cracked units, open joints in locations other than a bend, or evidence of improper design or construction, such as improper block alignment.

- **Corner Breaks:** One or more of the block corners are cracked or broken off. The plane of fracture is approximately 45 degrees from vertical, and the size of the fracture exceeds 2 inches along all three major axes. Smaller breaks should be considered to be fraying or edge spalls.

- **Cracked Block:** Randomly cracked block units. The direct cause of the crack development is uncertain. Examples might include diagonal or straight crack propagation across the capstone, or vertical crack propagation along the face of the stone.

- **Efflorescence:** Efflorescence, informally referred to as leaching, is a white deposit on the concrete surface caused by the crystallization of soluble salts (calcium chloride, calcium hydroxide) contained within the cement paste. Water traveling through the concrete dissolves these salts and usually deposits them along cracks where the water exits. Efflorescence indicates that water and dissolved chemicals are able to pass through and contaminate the concrete. This is primarily an aesthetic problem, but can serve as evidence of increased block porosity and weakness in extreme cases.

- **Embedded Vegetative Growth:** This is the presence of plant foliage growing from between the block units or in wash-through deposits. The penetration of plant roots into the blocks may cause units to crack either through root growth into pores and small flaws or by extensive plant growth between block units. Neither exposed fine tree and plant roots that have grown through the wall from behind, nor plants present as architectural enhancements are considered Embedded Vegetative Growth.

- **Abrasion:** Abrasion is the loss of surface material due to the action or water or wind-blown particles. This distress may be easily confused with surface scaling and freeze-thaw damage because the latter two are generally more severe in areas of water flow and saturation. Abrasion is typically evidenced by relatively uniform loss of surface mortar along the paths of water flow. Surface scaling and freeze-thaw damage is generally greater in depth.

- **Fraying/Spalling (Block Edges):** Fraying/ spalling is the presence of minor chipping along block edges and corners. Occasionally fraying/ spalling will be the result of improper handling and placement during construction. Other causes include thermal expansion, or the placement of blocks on uneven surfaces. The presence of a large number of spalls in the same place on each block may indicate a problem in manufacturing.

- **Freeze-Thaw Damage:** This is the progressive internal deterioration of saturated block material in the presence of freezing and thawing temperatures. The expansion of water during freezing periods can produce significant internal damage. Resulting defects will appear as areas of crumbling or general deterioration. This type of damage is most often found in areas that are frequently saturated and exposed to freezing conditions. Note if this type of damage is occurring on a vertical or horizontal surface.

- **Manufacturing Flaws:** These are systematic flaws in the block units that are the result of a design or manufacturing problem.

- **Popouts:** Aggregate particles near the surface of the block units that have expanded and caused concrete to flake or chip off. Typically caused by water trapped near the surface of the concrete that freezes and expands.

- **Scaling:** This is a special type of freeze-thaw damage. It is generally characterized by significant exterior damage and crumbling, more so than typical freeze-thaw damage. Refer to Figure 4.5.4-5 for a view of heavy scaling on a masonry noise barrier wall.

- **Staining:** This is the discoloration of block units caused by exposure to elements such as surface runoff containing dark clays or organic material, deicing chemicals, mold growth, moss, and other sources. While staining in general is aesthetic, on other materials such as vinyl or other plastics, it could be an indication of an area of weakening material.

- **Structural Distress:** Any evidence of structural failure of the entire block unit such as shifting and wall movement.

Figure 4.5.4-5: Heavy Scaling with Efflorescence on a Masonry Noise Barrier Wall.

4.5.4.3 Concrete Noise Barrier Walls

The following is a discussion of different types of material defects that may be found when inspecting pre-fabricated concrete panels and cast-in-place concrete. Refer to Figure 4.5.4-6 for an overall view of a concrete noise barrier wall.

Figure 4.5.4-6: Overall View of a Concrete Noise Barrier Wall Comprised of Reinforced Concrete Core and Durisol (wood/cement fiber) Cladding.

- **Fraying/Spalling/Exposed Reinforcing Steel:** Fraying/spalling is the presence of minor chipping along the concrete edges and corners. Occasionally fraying/spalling will be the result of improper handling and placement during construction. Other causes include thermal expansion, or the placement of concrete sections on uneven surfaces, or lowering into place in a cocked position. The presence of a large number of spalls in the same place on each block may indicate a problem in manufacturing. Consistent spalling over time can result in exposed reinforcing steel.

- **Scaling**: This is a special type of freeze-thaw damage. It is generally characterized by significant exterior damage and crumbling.

- **Cracking:** Random cracks in the structure. Examples might include diagonal, straight, or horizontal crack propagation. Cracking may also occur longitudinally within the concrete core of the Durisol type panels which may not be visible except along the bottom face of the bottom panel of structure-mounted noise walls. Refer to Figure 4.5.4-7 for a view of a concrete panel with a horizontal crack. Refer to Figure 4.5.4-8 for a view of cracking along the underside of a bottom panel of a structure mounted noise wall.

Figure 4.5.4-7: Concrete Panel Noise Barrier Wall with a Horizontal Crack at Mid-Panel.

Figure 4.5.4-8: Cracking with Efflorescence Along Midline of Underside Panel.

- **Corrosion:** Corrosion of the steel posts between panels will typically occur along the side facing the roadway spray and especially at the base where debris and deicing agents will accumulate. The inspector should check for scaling, pack rust and section loss. Digging out a few inches of material around the steel posts may be required to fully expose corrosion occurring at the groundline. Refer to Figure 4.5.4-13.

- **Crushing:** Crushing of Durisol type panels is common at the bottom corners where the wall is supported on bearing pads. This is often attributed to inadequate bearing area due to undersized bearing pads or shims.

- **Misalignment:** Improper shimming at the base of the panels may magnify to the upper panels leading to the top panels no longer being held within the flanges of the steel support posts.

4.5.4.4 Timber Noise Barrier Walls

The following is a discussion of different types of material defects that may be found when inspecting timber. Refer to Figure 4.5.4-9 for an overall view of a timber plank noise barrier wall and Figure 4.5.4-10 for a view of missing timber planks on a noise barrier wall.

Figure 4.5.4-9: Timber Plank and Post Noise Barrier Wall.

Figure 4.5.4-10: View of Broken and Missing Timber Planks.

- **Decay:** This is the breaking down of a material as a result of bacteria or fungi attack.
- **Insect Infestation:** Often, insects such as termites use timber as food and shelter. These and other insects can be detrimental to the integrity of a timber wall and can cause significant internal damage.
- **Vermin Damage:** Damage by small animals and birds using the timber for shelter can also be significant.
- **Fire Damage:** If the structure appears black, a piece of it has disappeared, or a large amount of ashes are present at the site, it is likely the structure was damaged by fire.

4.5.4.5 Metal Noise Barrier Walls

The following is a discussion of different types of material defects that may be found when inspecting metal. Refer to Figure 4.5.4-11 for an overall view of a metal noise barrier wall.

Figure 4.5.4-11: Steel Noise Barrier Wall.

- **Fatigue Cracking:** Cracks that have formed as a result of cyclic wind loading.

- **Corrosion:** This type of deterioration is the slow, steady deterioration of ferrous material due to exposure of bare metal, moisture and oxygen. Exposure to deicing chemicals act as a catalyst and accelerate metal corrosion. Refer to Figure 4.5.4-12 and Figure 4.5.4-13 for views of metal corrosion.

Figure 4.5.4-12: Severe Corrosion of a Metal Noise Barrier Wall.

Figure 4.5.4-13: View of Through Section Loss Along the Base of a Post for a Post and Panel Noise Wall.

4.5.4.6 Noise Barrier Walls of Other Materials

Several other materials gaining greater use in the construction of noise barriers include transparent panels; plastics; recycled rubber; and FRP composites. Refer to Figure 4.5.4-14 for a view of a plastic composite noise barrier wall. These materials typically exhibit the following types of deterioration:

- **Ultraviolet Degradation:** The material may exhibit cracking, discoloration or show signs of disintegration. This is typically the result of continuous exposure of sun light on materials that are not UV-stable.

- **Material Incompatibility:** In certain instances two or more materials in contact with one another may induce a chemical reaction that is detrimental to one or all of the materials. This is more common with two different metal types touching such as galvanized steel and aluminum however it may occur with other materials as well. The inspector should look for any type of indications at the interfaces between two differing materials. The buildup of scale or leachate may be an indication of distress between the materials.

- **Corrosion Damage:** Similar to steel corrosion, other metals also undergo the same electro-chemical process when exposed to moisture, air and deicing agents.

- **Overstress Damage:** Concrete and steel can withstand tremendous stresses relative to member sizes. Other materials when shaped similarly to their concrete or steel counterparts may not be capable of this same resistance and may exhibit undesirable behavior such as creep or buckling under weight. The inspector should note any warping or rippling occurring within a material surface as this may be an indication of overstressing or over application of a material type.

Figure 4.5.4-14: Plastic Composite Noise Barrier Wall.

4.5.5 Wall Panel or Post Movement

Wall movement may be the result of numerous factors, including soil settlement or material or component failure. Refer to Figure 4.4.7-1 through Figure 4.4.7-6 in Part 4, Chapter 5 Retaining Walls for types of wall movement.

4.5.5.1 Vertical Movement

Vertical movement can occur in the form of uniform settlement or differential settlement. Uniform settlement will have little effect on the structural stability of the wall. Differential settlement, on the other hand, can create serious problems in the wall. Differential settlement may cause the opening of joints or cause wall cracking or transverse tipping of the wall.

The most common causes of vertical movement consist of soil bearing failure; soil consolidation; erosion; and deterioration of the foundation material. On structure-mounted noise walls, the inspector should pay close attention to the panels nearest the transition from bridge to retaining wall. Bridges tend to not settle due to foundations on piling whereas retaining walls will not typically bear on such a robust substructure and are, in fact, designed to settle, often 3 inches or more. Newer designs eliminate this problem by using double posts and ending the noise wall at the bridge/retaining wall joint.

4.5.5.2 Lateral and Rotational Movement

Noise barrier structures that are structure mounted to retaining walls are susceptible to lateral movements or sliding and rotational movements. Lateral movement occurs when the lateral soil pressures behind the retaining wall exceed the resisting soil frictional and shearing forces or anchorages that hold the retaining wall in place. Rotational movement, or tipping, is generally the result of asymmetrical settlements or lateral movements; however, it may result from increased soil pressure behind the retaining wall. As the retaining wall begins to rotate

or tip, the noise wall supported to its top will follow suit. This resulting rotation is magnified by the fact the noise wall extends much higher than the retaining wall.

Noise walls secured to footings in the ground may also exhibit lateral and rotational movement if there is improper drainage or excessive freeze/thaw in the soils. When drainage systems along the wall are not properly removing water from the adjacent soils, the soils can become saturated forcing any additional water to "find" another means of outletting. This can result in slope damage or erosion around the base of the footings. Saturated soils of less cohesive make ups could slide potentially failing the noise wall.

The most common causes of lateral movement are slope failures; seepage; changes in soil characteristics (e.g., frost action and ice); and long term consolidation of the original soil. The most common causes of rotational movement are saturation of backfill due to clogged drains; erosion of the embankment along the front of the wall; and improper design. Refer to Figure 4.5.5-1 for a view of lateral wall movement and its effects on a wall. Refer to Figure 4.5.5-2 for a view of a failed drain tile system resulting in the washout out of material below a noise wall.

Figure 4.5.5-1: Lateral Movement of a Noise Barrier Wall.

Figure 4.5.5-2: Drain tile system failure below noise wall resulting in loss of material and exposure of noise wall footing.

4.5.6 Assessments

Assessments allow for the evaluation of secondary components not necessarily impacting the structural integrity of the structure. The inspector must be familiar with all the available assessments within the HSI system and appropriately capture them within the inspection report for each noise barrier wall structure. Evaluating assessments provides a more accurate picture of the structure being inspected and alerting the Department of potential future issues including traffic hazards or functionality of the wall. All available assessments for noise barrier walls are the same as described for retaining walls. Refer to Part 4 – Chapter 5 Retaining Walls, Section 4.5.4 Retaining Wall Assessments.

4.5.7 Recommended Inspection Procedures

Each structure should be inspected for component deterioration, impact damage and site conditions that may potentially contribute to failure of the barrier. Erosion of the berm material is the only type of deterioration that will compromise the effectiveness of the noise berm. A visual Inspection of Noise Barrier Walls can usually be completed without special access equipment. However, the inspector may need to utilize a bucket truck or ladder to obtain additional information on significant defects above ground, especially at connection or panels of structure-mounted systems. For noise walls that have significant vegetative growth that obscures the view of the wall (often due to wild or intentionally planted ivy), scheduling the inspection in the winter months after the leaves have fallen is recommended.

Inspection reports with documented condition state 3 or 4 defects require accompanying photographs. Each location of condition state 4 distress must have a photograph while an overall condition photo may be permitted for condition state 3.

During an inspection, the inspector is responsible for reviewing and verifying all inventory data such as but not limited to:

- Wall dimensions
- GPS coordinates
- Location
- Inspection procedures

While inspecting of steel components, the inspector should have access to non-destructive testing equipment. Additional evaluation may be required on anchorage systems or fasteners exhibiting excessive section loss to determine whether the component is compromised or in need of repair or complete replacement. D-meter readings and measurements should be performed on steel members in those locations exhibiting heavy scale, such as the post base or at fasteners.

The following general sequence can be used for Noise Barrier Wall inspections:

1. Mobilize to site and set-up traffic control.
2. Identify structure number, if applicable.
3. Perform Inspection.
4. Review inspection notes by inspector to ensure completeness and correctness.
5. Determine previous inspection interval.
6. Recommend new inspection interval (if different from the old).
7. Notify the noise barrier owner if there are any significant safety issues that require immediate correction.
8. Remove traffic control.

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4.6 SMALL BRIDGES (C-STRUCTURES)

4.6.1 Introduction

This chapter will discuss small bridge structures related to transportation facilities, commonly known as c-structures by the Wisconsin Department of Transportation. These structures can service waterway conveyance, provide pedestrian access under roadways, or act as a cattle/animal pass under roadways. Sections 4.6.2 through 4.6.4 discuss specific types of box and pipe culverts, which are widely used throughout the State. Culverts primarily serve to transfer open channel storm water drainage flow. Utility ducts and sewers are similar structures, which are not covered in this Manual. When box or pipe culvert are 20 feet or more in span, they are classified as bridges and are documented with element level inspection terminology discussed in Part 2 of this Manual.

4.6.2 Small Bridge Structures

As discussed in Part 1 of this Manual, any structure greater than 20 feet in length is defined as a bridge per the Code of Federal Regulations. The State of Wisconsin categorizes smaller structures under two categories, Small Bridges (C-Structures) and Roadway Culverts. In general, C-Structures have a unique structural design and are inventoried and inspected under the direction of the Bureau of Structures. Structures not meeting this criteria (mainly pipe culverts) are inventoried and inspected under the direction of the Bureau of Highway Maintenance. This chapter will cover C-Structures in great detail.

Many structure types make up the small bridge inventory; In general, c-structures have clear openings of 20 feet or less measured along the centerline of the roadway. Refer to Figure 4.6.2-1 for structure length definition. The following structures shall be categorized as c-structures:

- Bridge-like structures (deck girders, flat slabs, rigid frames, etc.)
- Single and multi-cell box culverts (opening 20 square feet or greater)
- Arches
- Structures without a floor slab (including arches on footings)
- Structural plate pipe culvert or arches

Though rare, a Small Bridge may include multiple structural plate pipe culverts where the clear distance between openings is less than half of the smaller contiguous opening. In the case of multiple pipes, the structure length will be the distance from the extreme edges of the two outermost openings. A series of structural plate pipes are considered to be one structure when the clear distance between the openings is less than half the width of the smallest opening.

Nearly all pipe structures (except structural plate pipe culverts) are not considered c-structures and are inventoried by the Bureau of Highway Maintenance.

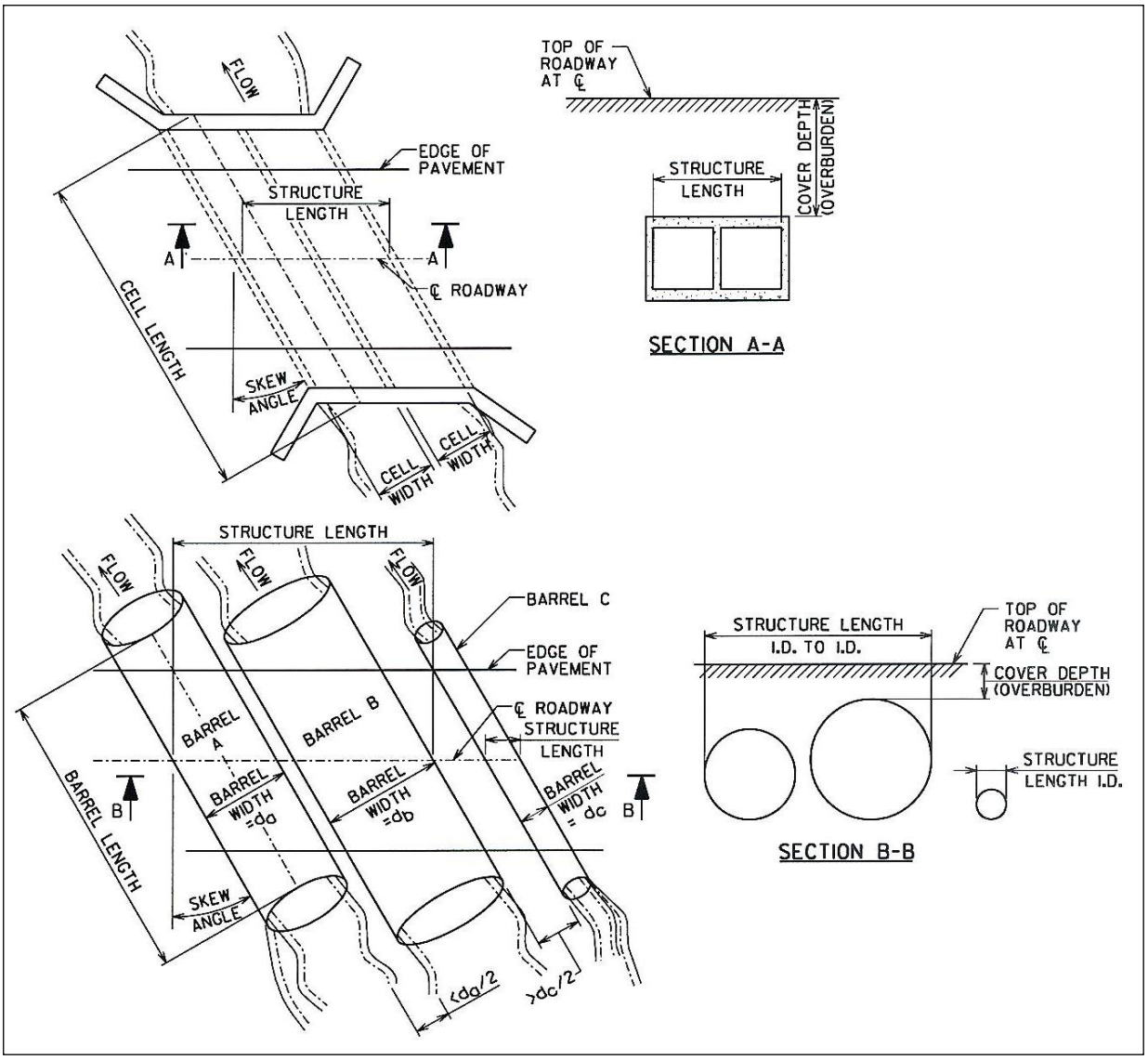

Figure 4.6.2-1: Small bridge span definitions. Note the pipes would only be considered if at least one was a structural plate pipe culvert.

Small bridge box/pipe culverts are hydraulically designed to efficiently deliver peak water flows underneath the roadway above. The hydraulic efficiency is improved when these flows submerge the inlet end of the structure. Bridges, on the other hand, are not always designed for optimum hydraulic efficiency, and may even be designed to allow water to flow over the deck under extreme flood conditions.

Small bridge box/pipe culverts generally do not have a roadway deck, and there is no distinction between substructure and superstructure. Small bridge box/pipe culverts are usually covered by roadway fill material, typically called the overburden, or an embankment. They are designed to carry the soil dead load above the structure, and also for the live loads due to traffic above. Above certain fill heights, however, live loads will arch over the structure, leaving only the soil dead loads to be carried by the structure. A small bridge box/pipe culvert's

strength is achieved through its own material properties and lateral pressures from the surrounding soil.

Small Bridge Structures may be constructed in a manner similar to a bridge (such as using multiple rolled steel beams and a deck). Though spanning less than 20 feet, structural elements such as these should be inspected in a manner similar to a bridge as discussed in Part 2 of this Manual.

Small Bridges shall have an inspection interval not to exceed 4 calendar years (see Part 4, Chapter 1 for more information). The structures shall be inventoried and reported within HSI. It should be noted that these structural small bridge inspections may be supplemented by hydraulic opening and debris removal maintenance inspections in accordance with Section 48 of the State Highway Maintenance Manual.

Small Bridges (C-Structures) are given "C-number" designations indicating they are not considered bridges per the FHWA guidelines, but are still inventoried and inspected with FHWA Bridge Guidelines. The inspector shall note the location (or lack thereof) of the Name ID plaque on the structure under the Structure Specific Notes within the inspection report and under the Structure ID Plaque Assessment.

4.6.3 Small Bridge – Bridge Like Structures

For those small bridges (clear span as measured along centerline roadway less than 20') that are constructed similar to a bridge (i.e. do not have a structural bottom), the structure shall be inspected as a bridge with all pertinent bridge elements (i.e. abutment, girders, deck, bridge railing, etc.). Refer to Part 2 for discussion and elaboration on bridge elements. Note that these structures shall be inventoried and inspected as C-structures.

The remainder of this chapter shall discuss both box and pipe culverts that qualify as small bridges.

Figure 4.6.3-1: View of Name ID Plaque located on the Wing of a Culvert End Treatment.

Figure 4.6.3-2: A bridge like structure with total length less than 20 ft. This structure is designated as a small bridge in HSI, and inspected with bridge elements.

4.6.4 Small Bridge – Box and Pipe Culverts

Box and Pipe culverts are constructed of a wide variety of materials, such as concrete, steel, timber, masonry, plastic, or aluminum. This section highlights the types of culvert shapes, the structural categories of culverts, and the variety of culvert end treatments that the inspector may encounter. Discussions of the various materials used to build culverts follow in Section 4.6.6.

4.6.4.1 Culvert Shapes

Many culvert shapes are present throughout Wisconsin. The shape chosen for a location may or may not be dependent upon hydraulic performance requirements. Other factors that may affect the shape of culvert chosen are structural requirements, potential for clogging by debris, and the need for a natural stream bottom. Typical culvert shapes are described below.

- **Circular:** This is the most common culvert shape. Although they are hydraulically and structurally efficient, they can reduce a stream's width, and they are more prone to clogging than other shapes. Soil pressures around circular shaped culverts are directed towards the middle of the shape, and are fairly uniform all around. Most of these assets are covered under the BHM inventory/inspection program.

Figure 4.6.4-1: View of circular structural plate pipe culvert.

- **Pipe Arch:** This shape is used when the distance from the stream bottom to roadway is limited. Their shape is arched on top, and flattened on the bottom. The flattened bottom allows a wider stream to flow through the culvert. As with circular culverts, they are prone to clogging. Since these shapes closely approximate a true arch, soil pressures on the underside of the culvert are fairly low, with high reaction pressures occurring at the bottom corners of the shape. They are not as structurally efficient as a circular shape. Most of these assets are covered under the BHM inventory/inspection program.

- **Elliptical:** This shape has the same advantages and disadvantages as pipe arch culverts. Most of these assets are covered under the BHM inventory/inspection program.

- **Arch:** Arches are open shapes used when limited obstruction to stream flow is required, and where the natural stream bottom is inherently resistant to scour. They offer less obstruction to stream flow than pipe arches. The spring lines of arch culverts bear on footing foundations, and rip rap is commonly used to protect them from erosion when the stream bottom is not resistant to scour. Soil pressures on the sides of an arch culvert try to push the sides towards the center of the stream, resulting in the top of the arch being pushed up. Soil dead loads above the top of the arch help to restrain this “peaking” tendency.

Figure 4.6.4-2: View of a corrugated steel arch culvert with masonry headwalls.

- **Tied Arch:** These are similar to arch culverts, but differ in that tied arch culverts have a floor (the natural stream bed is covered).

- **Box:** These are square or rectangular shaped structures. They are commonly chosen because they are adaptable for many site conditions, and they can be used when the distance from the stream bottom to roadway is limited. Box culverts always have a floor. Some have stream bed material placed within them either manually or filled in from sediment migration in the stream. These structures are not as structurally efficient as other culvert shapes.

Figure 4.6.4-3: View of a single cell reinforced concrete box culvert.

- **Frame:** These are similar to box culverts, but differ in that they do not have a floor (the natural stream bed is exposed). The walls of these culverts bear on footing foundations.

- **Multiple Barrel:** Though not a "shape" in the strict definition, multiple barrel or cell culverts are simply a series of pipes, arches, or boxes placed side by side. Their use is common when the distance from the stream bottom to roadway is limited. Their major disadvantage is that waterway debris is easily snagged by the cell walls or soil between the openings. Refer to Figure 4.6.4-4 for a view of a multiple barrel culverts.

Figure 4.6.4-4: Two-cell reinforced concrete culvert.

- **Various:** It is not uncommon for a culvert structure to be comprised of several different types and shapes of culverts. In this instance, the inspector shall include all elements described in Section 4.6.611 that comprise the culvert. Refer to Figure 4.6.4-5 for a view of two different culvert types acting as a single structure.

Figure 4.6.4-5: View of reinforced concrete single cell box culvert with corrugated steel pipe arch extension

4.6.4.2 Structural Categories

There are two structural categories of culverts, flexible and rigid. It is important for an inspector to understand the structural behavior of a culvert so that a proper assessment can be made as to the cause of any damage found. Structural behavior is dependent upon the material from which a culvert is built.

Flexible Culverts

Culverts constructed of metal are flexible, and can be easily deformed under load if not properly installed. Because the thinness of the corrugated metal plates leaves these culverts with little bending strength of their own; they must rely on the surrounding soil/backfill material for support and shape retention.

A metal structural plate pipe culvert with no lateral soil support will “squash” or flatten into an oval shape under vertical loads. The top of the pipe will deflect downward, and the sides of the pipe will bulge outward. Properly compacted soil surrounding the pipe will restrain the sides from bulging. This keeps the culvert ring in compression, and allows the culvert to support vertical loads with acceptable deflections by way of arching action.

Rigid Box Culverts

Cast-in-place concrete, precast concrete, and masonry box culverts are considered to be rigid. The culvert material itself is stiff enough to resist bending and to support vertical loads independent of the surrounding soil. As such, tension and compression stresses due to bending are created in the pipe. When viewed from the inside of a closed pipe, a culvert will experience tensile stresses on the top and bottom surfaces, while compressive stresses will be generated on the sides. These stresses will be opposite on the soil side of the pipe. Rigid culverts do not appreciably deflect before they crack or fracture.

4.6.5 Small Bridge Inspection

Many steel culverts are large enough so that the entire length can be inspected from the inside. Small culverts that cannot safely or practically be entered should be examined by looking through the opening from both ends. Locations along sectional culverts may be conveniently referred to by using pipe joints as station numbers from one end of the pipe. Defect positions along the circumference of the culvert barrel may be referenced like hours on a clock.

A routine inspection of any structure will include several items. First, an element level inspection must be performed not only on the culvert itself, but also on the roadway over the structure. An inspection must also be carried out on the wingwalls, curbs, sidewalks, and railings if these elements or assessments exist. Second, an NBI condition rating must be assigned to the culvert structure, and to the channel and channel protection. Third, an NBI appraisal rating must be assigned for the waterway adequacy and approach roadway alignment. An NBI appraisal rating must also be assigned for the railing, approach guardrail, guardrail transitions, and guardrail ends if these items exist.

Prior to inspecting a culvert, the inspector should review all available information about the culvert. This will help establish equipment needs, inspection procedure, and flag areas of existing distress. At a minimum, most culvert inspections will require rubber boots/hip waders and a flashlight. Also, a range pole/probing rod should be brought along to not only check for scour or deterioration below the water level, but also to help keep the inspector's balance while wading through muck or over loose rip rap.

4.6.5.1 Inspection Hazards

Although culverts are low level structures, they do have a number of hazards with which the inspector must contend.

Inadequate Venting

Long culverts or culverts with blocked ends may have inadequate oxygen levels, or high concentrations of toxic or explosive gasses. Many longer culverts can be considered confined space. In this event, applicable OSHA regulations should be adhered to for confined space entry. Two people should perform the inspection in these situations. When air quality is suspect, gas monitors should be carried by the inspectors during the inspection to alert them of a hazardous environment. In addition, continuous ventilation may be required throughout the inspection to provide a safe working environment. Life line ropes clipped to the inspectors may also be used so that a third inspector stationed at the entry point may pull a disabled person out of the pipe.

Drowning

Soft muck can often be a problem during an inspection. After stepping into it, it is often difficult to pull one's foot out due the water and soil pressures surrounding it. Scour holes may be present within the streambed inside of a culvert. Since it is often difficult to visually detect these, it is easy for an inspector to accidentally step into one. High water velocities, flows, and slippery culvert floors may knock an inspector off of their feet. Probing rods, life vests, and hard hats should always be used while wading through these hazardous conditions.

Some culverts have limited or no headroom. When this situation is encountered, wading should not be attempted until the water level has lowered. If that is unlikely, underwater inspection methods are required to properly perform the inspection. Refer to Part 1 for underwater inspection equipment and personnel requirements.

Toxic Chemicals

Hazardous chemicals may be present in the stream water itself. A fire or explosion may result from careless behavior (smoking) in these conditions. An inspector should be aware of his work environment.

Animals

Debris accumulation and low water flows within culverts may provide ideal shelter for animals. The inspector should be aware that snakes, rodents, or dead animals may be present within the culvert.

Quick Sand Hazards

Quick sand is produced when water flows upward through sand. When the flow is strong enough, sand particles are lifted up out of the stream bed, and kept suspended by the turbulent nature of the upward rising water. A similar situation may be found at the outlet of a culvert conveying large quantities of water. The falling water creates a scour hole and constant turbulence to keep the streambed sand particles suspended. An inspector may accidentally step into the scour hole. Contrary to what Hollywood movies suggest, people do not sink in quick sand. The density of quick sand is greater than that of water, and people will actually float higher in the sand/water mixture than in pure water itself. An inspector found in this situation can simply swim away from the scour hole.

4.6.6 Small Bridge Elements

Elements are provided to break the structure down into its main structural components. Typically, when a structure has a structural concrete or steel floor, it may be considered a culvert and the appropriate AASHTO culvert elements shall be utilized. For small bridges – bridge like structures where the structure has no floor and does not support an overburden load, the structure should be evaluated using AASHTO bridge elements. Refer to Section 4.6.3 for information.

Culvert elements are affected by appropriate material defects structural elements. Other typical structural defects associated with culvert elements include scour and settlement.

The quantity for culvert elements is the sum of the lengths of all the barrels comprising the structure in lineal feet with each linear foot capturing the defects over the entire circumference of the culvert barrel or cell. It is not uncommon for a culvert to be comprised of several different types of culverts. The sum of each barrel of each type is summed and totaled under the appropriate culvert element. The successful dissemination of the structure is based on the inspector's notes, measurements and photographs. All visible portions of the culvert should be considered during inspection, including all assessments.

Element Defects

Refer to Part 2 Appendix A for defect descriptions. The defects listed are unique to the element and element material (i.e. concrete, steel, timber, etc.).

The inspector shall utilize the Defect Hierarchy concerning overlapping defects within a unit of measure when rolling up the total element condition state quantities. Refer to Part 2 Chapter 3 for the Defect Hierarchy.

When structural defects are present, the inspector shall report the defect in its entirety on the inspection report regardless if overlapping with material defects. However, only the controlling defect will be counted in the total element condition state quantity.

Structural defects shall be coded in their entirety on the inspection report regardless if overlapping with the other material defects. Structural defects represent global or external deterioration that adversely affects the structure. The scour defect should be used to capture the erosion of material along the base of a structure due to a stream or body of water. Settlement should be used for extended lengths of the structure exhibiting displacement vertically or out of plane. Structural defects should only be reported under structural elements, i.e. culvert elements, abutment elements, etc.

When assessing an element, the inspector shall utilize the specific material defects delegated to those elements. Furthermore, these defects are set into four Condition States. Each Condition State describes the progression of deterioration and severity for each defect. The Condition States are comprised of general descriptions and are uniquely colored to follow the severity the description represents.

- Condition State 1 Good Green
- Condition State 2 Fair Yellow
- Condition State 3 Poor Orange
- Condition State 4 Severe Red

The Condition States for each defect are summed up to the overall Element Condition State which provides the general condition rating for the element in the element's particular unit of measure. The sum of the defects must equal the total quantity for the element. All overlapping defects that were not included within the defect quantities (due to Defect Hierarchy), shall be included within the element or defect notes. Depending on the severity of the defect, photographic documentation may be required.

4.6.6.1 Steel Culvert (Element 240)

For c-structures, the primary type of structure using this element are structural plate steel pipe culverts. Structural plate steel pipe culverts are field assembled, built-up of several pre-curved corrugated plates. The field assembled plate sections are fastened together with bolts to form a continuous pipe. Fastened plate edges oriented along the barrel length are called seams, while plate edges transverse to the barrel are called joints. Circular, pipe arch, and arch

underpass shapes are commonly found in sizes up to 26 feet. They are galvanized for corrosion protection.

Corrugated steel box culverts approximate a rectangular shape, though their corners are rounded, sides tilted slightly, and tops slightly curved. They are reinforced in areas of maximum bending, and are available in spans up to about 20 feet.

Long span corrugated steel structures are similar to structural plate steel pipes, but long span culverts require longitudinal and circumferential stiffening members due to their great lengths. Because their lengths are normally greater than 20 feet, they are classified as bridges. They should, however, be inspected in a manner described in this section.

Element Level Inspection

This element describes steel culverts, including arched, round or elliptical pipes. Pipe culverts made of a metal other than steel shall be captured under element 243 Other Material Culvert.

Barrel misalignment, joint and seam condition, localized damage, and corrosion are areas of concern during any steel culvert inspection. However, due to the flexible nature of pipe, the primary area of concern is its shape.

Because steel culverts depend upon a well compacted soil to help retain their shape, excessive flattening or widening suggests a soil failure. As a minimum, the height and widest opening should be measured at both culvert ends. If culvert size and water flow permit, measurements in the middle of the barrel should also be recorded. These measurements should also be taken at 25-foot intervals between these three locations. Bolted joints are excellent places to take measurements, because they can be easily found for re-measurements on subsequent inspections. Measurements should always be taken at the inside crest of the corrugations. Critical measurements to be taken during an inspection for various barrel shapes have been reproduced from the *Culvert Inspection Manual*. Refer to Part 4, Appendix C, for these figures.

In the inspection report, steel culvert elements are recorded in units of linear feet. Although the culvert span, found in the inventory data, is measured along the centerline of the road, the culvert element quantity is calculated by taking the flow line length of the barrel times the number of barrels, or the sum of all barrel lengths if barrels differ in length. It is important to note that the culvert element may exhibit more than one defect along its length. It is the inspector's task to examine each element, determine the applicable defects and reasonably assign the appropriate Condition State to each defect. Where multiple defects exist within a unit of measure, the inspector shall refer to the Defect Hierarchy as described in Part 2 Chapter 3. The sum of all of the reported material defect condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of steel culverts should include the following items:

- Measuring the height and width of the barrel at the ends, middle, and at 25 foot intervals in between. These measurements should be compared to the original dimensions shown on the design drawings, if available. Excessive flattening suggests a soil failure.

- Indicating on the report the extent of any flattening found. Include the affected length along the barrel, and location of the distress.

- Examination of the roadway above for any dips, cracking, settlement, or patching that would suggest a failing culvert below.

- Checking for horizontal misalignments by sighting down the barrel length.

- Checking for vertical misalignments sighting down the barrel length, and by noting the presence of localized deep water or sediment accumulation at low spots within the barrel.

- Looking for localized damage in the form of dented, flattened, or bulged areas. Bulged areas on the bottom of a round barrel suggest culvert settlement over a local hard spot. Bulged areas on the bottom of an arch pipe suggest the corners have settled. Small areas of localized damage are usually not critical. Localized damage may jeopardize the barrel's ability to carry its ring compression loads.

- Looking for corrugation misalignments and "cusped" seams along the fastener lines. Cusped seams are an opening up of the plate lap as a result of loose fasteners, resulting in a discontinuity of the barrel curvature. They may allow moisture and backfill to leak inside of the culvert.

- Examining all seams and joints for plate separation. Measure any separations found. A small rod or flat rule should be used to probe through any separation to check for voids in the backfill.

- Checking for cracking at the bolt holes. These are usually caused by excessive barrel strains due to deflections.

- Verification of loose bolts by tapping with a hammer.

- Checking the entire barrel for the extent and severity of corrosion. The heaviest corrosion will normally appear near the water surface. Corrosion is the leading cause of steel culvert replacement. A sharp blow with the pointed end of a geologist's hammer is the accepted method for the inspector to evaluate the general integrity of the steel.

- Examining the inlet and outlet of the barrel for signs of scour or undermining underneath of a closed pipe. This could lead to piping.

- Examining for scour or footing undermining along the length of arch culverts. Undermining may allow the concrete footing to rotate, failing the arch.

- Looking for signs of abrasion along the barrel. Abrasion can accelerate the corrosion process by scraping away the metal's protective coating.

- Examination of end treatments or wingwalls for damage, deterioration, debris accumulation, streambed scour, undermining, settlement, tipping, and embankment erosion. These deficiencies can cause hydraulic inefficiencies of the culvert, and roadway settlements.

- Checking for excessive joint openings between headwalls, wingwalls, aprons, etc.

Figure 4.6.6-1: Barrel with good alignment - Condition State 1.

Figure 4.6.6-2: Surface corrosion near the waterline – Corrosion Condition State 2.

Figure 4.6.6-3: Light to moderate bolt corrosion – Connection Condition State 2.

Figure 4.6.6-4: Severe bulging in numerous locations – Distortion Condition State 3.

Figure 4.6.6-5: Severely distorted barrel – Distortion Condition State 4.

4.6.6.2 Reinforced Concrete (RC) Culvert (Element 241)

Concrete box culverts may be cast-in-place (CIP), or precast. CIP concrete culverts are typically box or arch shaped with the box shape being more common. The advantage of CIP culverts is that they can be designed to fit the specific needs of a site. The hydraulic openings of box shaped culverts are commonly referred to as cells. Multiple cell box culverts are common.

Precast culverts are available in several standard shapes including circular, box, elliptical, pipe arch, and arch. The hydraulic openings of circular, elliptical, pipe arch, and arch-shaped culverts are commonly referred to as barrels. Sizes up to 12 feet wide are commonly available for closed shapes.

Proper bedding compaction and preparation during construction is critical if precast closed sections are to perform well in service. Properly prepared bedding material (usually crushed stone) under the sections will evenly support the culvert sections. Voids within the bedding will force the rigid culvert sections to span over these soft spots, often resulting in transverse cracks. Voids under the mid-length of a section will lead to cracks on the bottom half of the pipe at mid-length. Voids under the ends of a section will lead to cracks on the top half of the pipe at mid-length.

Improper compaction of fill material on the sides of a culvert may lead to longitudinal cracks. Without proper soil side support, the pipe will want to flatten under vertical loads. Because concrete is a rigid material, it will not flex, and longitudinal cracks along the top, bottom, and sides of the pipe will result.

The soil surrounding concrete culverts is very effective in protecting them from severe temperature fluctuations, deicing chemicals, and impact loading. As a result, they tend to hold up fairly well, and do not deteriorate as quickly as concrete bridge elements.

Element Level Inspection

Barrel misalignment, joint condition, and localized wall damage are areas of concern during any concrete culvert inspection. Since they are classified as rigid culverts, they will not deform under load as do corrugated steel culverts. However, they will show signs of distress due to overloading or differential settlement. The distress usually shows up as longitudinal or transverse cracks.

On the inspection report form, reinforced concrete culvert elements are recorded in units of lineal feet. Although the culvert span, found in the inventory data, is measured along the centerline of the road, the culvert element quantity is calculated by taking the flow line length of the barrel times the number of barrels, or the sum of all barrel lengths if barrels differ in length. It is important to note that the culvert element may exhibit more than one defect along its length. It is the inspector's task to examine each element, determine the applicable defects and reasonably assign the appropriate Condition State to each defect. Where multiple defects exist within a unit of measure, the inspector shall refer to the Defect Hierarchy as described in Part 2 Chapter 3. The sum of all of the reported material defect condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of concrete culverts should include the following items:

- Examination of the roadway above for any dips, cracking, settlement, or patching that would suggest a failing culvert below.
- Checking for horizontal misalignments by sighting down the culvert length.
- Checking for vertical misalignments by sighting down the culvert length and noting the presence of localized deep water or sediment accumulation at low spots within the culvert.
- Looking for open joints. Open joints may allow water infiltration into the culvert, possible allowing the backfill material to enter the culvert as well. This can lead to roadway settlement above, and debris accumulation within the culvert. Open joints that allow water to flow out of the culvert (exfiltration) may cause soil erosion around the culvert. This can lead to culvert (and roadway) settlements.
- Looking for cracks, delaminations, or spalls at expansion joints or precast section joints. This type of deterioration suggests improperly functioning joints, possible due to culvert settlements.
- Looking for longitudinal cracks along the barrel or cell length. These may be caused by excessive overloads or improper fill compaction of barrel sections, or lateral differential settlements of box sections. Medium to wide crack widths should be measured and recorded.
- Examination of box culvert tops when these elements double as the riding surface of traffic above. Longitudinal cracks in the culvert top may be caused by traffic overloads.

- Looking for transverse cracks. These may be caused by differential settlements along the culvert length, often the result of improper bedding material preparation.
- Checking the entire barrel or cell for delaminating, spalls, exposed reinforcing steel, cracking and leaching. Note the location of such defects.
- Examining the inlet and outlet of the barrel for signs of scour or undermining underneath of a closed pipe. This could lead to piping.
- Examining for scour or footing undermining along the length of arch culverts. Undermining may allow the concrete footing to rotate, failing the arch.
- Looking for signs of abrasion along the length of the culvert.
- Examining any anchor bolt conditions at the springline of precast culverts.
- Examination of end treatments or wingwalls for damage, deterioration, debris accumulation, streambed scour, undermining, and embankment erosion. These deficiencies can cause hydraulic inefficiencies of the culvert, and roadway settlements.
- Checking for excessive joint openings between headwalls, wingwalls, apron, etc.

Figure 4.6.6-6: Reinforced Concrete Culvert - Condition State 1.

Figure 4.6.6-7: Deterioration on Culvert Wall – Heavy Density Cracking with Efflorescence Condition State 4.

Figure 4.6.6-8: Differential Settlements Between Box Segments – Settlement Condition State 3.

Figure 4.6.6-9: Joint Separation Between Pipe Segments – Settlement Condition State 3.

4.6.6.3 Timber Culvert (Element 242)

Timber culverts are usually in the form of box culverts and are constructed of individual small dimension timbers. They are not common in Wisconsin, but for the proper location, they can be an economical and easy to construct installation.

Element Level Inspection

Material deterioration and decay, differential settlement, and impact damage for flood debris are the primary concerns of any timber culvert. In addition overloading may show up as wall or ceiling bulging, and by splitting and protrusion of the individual laminations.

On the inspection report form, timber culvert elements are recorded in units of lineal feet. Although the culvert span, found in the inventory data, is measured along the centerline of the road, the culvert element quantity is calculated by taking the flow line length of the barrel times the number of barrels, or the sum of all barrel lengths if barrels differ in length. It is important to note that the culvert element may exhibit more than one defect along its length. It is the inspector's task to examine each element, determine the applicable defects and reasonably assign the appropriate Condition State to each defect. Where multiple defects exist within a unit of measure, the inspector shall refer to the Defect Hierarchy as described in Part 2 Chapter 3. The sum of all of the reported material defect condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of timber culverts should include the following items:

- Timber culverts more than twenty years old or any showings signs of distress should be completely sounded with a hammer. This is a preliminary, quick check for decay by listening for a resulting hollow sound.

- The preservative treatment of the elements and its conditions should be noted. Because of small dimension laminations, their preservative treatment extends throughout the timber resulting in little decay. But examine the entire element for signs of decay. Signs include discolored wood, rotted texture; section loss, fruiting (fungi) bodies; and insect infestation.

- Examination of the roadway above for any dips, cracking, settlement, or patching that would suggest a failing culvert below.

- Checking for horizontal misalignments by sighting down the culvert length.

- Checking for vertical misalignments by sighting across the tops of the culvert bays. Excessive deflections indicate that the culvert is experiencing differential settlements

- Checking the walls for plumbness. A plumb bob or smart level may be used. The measured amount of deflection should be recorded

- Looking for signs of decay due to fungi or parasites.

- Looking for any wall or ceiling bulges, and note any tipped walls.

- Checking for broken or missing timbers. These may be allowing backfill material to enter the culvert, causing roadway settlement above.

- Examination of any exposed fasteners for deterioration.

- Examining the inlet and outlet of the barrel for signs of scour or undermining underneath of a culvert with a floor.

- Looking for signs of abrasion along the length of the culvert.

- Examination of end treatments for damage, deterioration, debris accumulation, streambed scour, undermining, and embankment erosion. These deficiencies can cause hydraulic inefficiencies of the culvert, and roadway settlements.

- Checking for excessive joint openings between headwalls, wingwalls, apron, etc.

- Checking for missing laminations at the openings and inverts.

- If the claw or pointed end of a hammer penetrates the wood, that is a sign of decay. The extent of decay can also be estimated by trying to insert a chisels or other probe into suspected areas.

- All suspected decay areas may be further analyzed with a Resistograph or other similar NDE method to determine the extent of the decay. See Chapter 5 for further information. The location and the extent of such decay should be estimated and should be reported to the Program Manager.

4.6.6.4 Masonry Culvert (Element 244)

Masonry culverts are not constructed anymore, but can still be found in-service. They are built of stone or brick formed into arch or box shapes. Joints between the masonry units may be mortared, set in concrete, or mortarless. Refer to Figure 4.6.6-10 for a view of a masonry culvert. Masonry is still sometimes used to construct headwalls for culverts of other material types when aesthetics is important.

Figure 4.6.6-10: Masonry Culvert with Severe Deterioration of Mortar Joints.

Element Level Inspection

Material deterioration is the primary concern of any masonry culvert. Refer back to Figure 4.6.6-10 for a view of typical deterioration. In addition, signs of soil distress or overloading may show up as wall or arch bulging.

On the inspection report form, masonry culvert elements are recorded in units of linear feet. Although the culvert span, found in the inventory data, is measured along the centerline of the road, the culvert element quantity is calculated by taking the flow line length of the barrel times the number of barrels, or the sum of all barrel lengths if barrels differ in length. It is important to note that the culvert element may exhibit more than one defect along its length. It is the inspector's task to examine each element, determine the applicable defects and reasonably assign the appropriate Condition State to each defect. Where multiple defects exist within a unit of measure, the inspector shall refer to the Defect Hierarchy as described in Part 2 Chapter 3. The sum of all of the reported material defect condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

Maintenance inspection of other culverts should include the following items:

- Examination of the roadway above for any dips, cracking, settlement, or patching that would suggest a failing culvert below.
- Checking for horizontal misalignments by sighting down the culvert length.
- Checking for vertical misalignments by noting the presence of localized deep water or sediment accumulation at low spots within the culvert.
- Looking for cracked, crushed, or missing masonry units or mortar.
- Looking for signs of excessive masonry weathering.
- Looking for any masonry culvert wall or arch bulges. This suggests a soil failure. The location and size of such distress should be recorded.
- Indicating on the report the extent of any flattening found on plastic culverts. Include the affected length along the barrel, and location of the distress.
- Examining the inlet and outlet for signs of scour or undermining underneath of masonry culverts with a floor, or for plastic culverts.
- Examining for scour or footing undermining along the length of masonry culverts without floors. Undermining may allow the footing to rotate, failing the culvert.
- Looking for signs of abrasion along the length of the culvert.
- Examination of end treatments for damage, deterioration, debris accumulation, streambed scour, undermining, and embankment erosion. These deficiencies can cause hydraulic inefficiencies of the culvert, and roadway settlements.
- Checking for excessive joint openings between headwalls, wingwalls, apron, etc.

4.6.6.5 Prestressed Concrete (PS) Culvert (Element 245)

A prestressed concrete culvert is similar to a reinforced concrete culvert in terms of inspection. Cracking will typically be less prevalent on a prestressed member due to design but any cracks found during inspection should be heavily scrutinized. Prestressing strands are subjected to higher forces when compared to mild reinforcing steel and are therefore less resilient to corrosion from water intrusion.

Element Level Inspection

On the inspection report form, prestressed concrete culvert elements are recorded in units of linear feet. Although the culvert span, found in the inventory data, is measured along the centerline of the road, the culvert element quantity is calculated by taking the flow line length of the barrel times the number of barrels, or the sum of all barrel lengths if barrels differ in length. It is important to note that the culvert element may exhibit more than one defect along its length. It is the inspector's task to examine each element, determine the applicable defects

and reasonably assign the appropriate Condition State to each defect. Where multiple defects exist within a unit of measure, the inspector shall refer to the Defect Hierarchy as described in Part 2 Chapter 3. The sum of all of the reported material defect condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

As noted previously, any cracking found in a prestressed concrete culvert should be closely observed given the susceptibility of prestressing strands to corrosion. The limits on crack width and spacing are much less with prestressed concrete when compared to reinforced concrete.

Maintenance inspection of concrete culverts should include the following items:

- Examination of the roadway above for any dips, cracking, settlement, or patching that would suggest a failing culvert below.
- Checking for horizontal misalignments by sighting down the culvert length.
- Checking for vertical misalignments by sighting down the culvert length and noting the presence of localized deep water or sediment accumulation at low spots within the culvert.
- Looking for open joints. Open joints may allow water infiltration into the culvert, possible allowing the backfill material to enter the culvert as well. This can lead to roadway settlement above, and debris accumulation within the culvert. Open joints that allow water to flow out of the culvert (exfiltration) may cause soil erosion around the culvert. This can lead to culvert (and roadway) settlements.
- Looking for cracks, delaminations, or spalls at expansion joints or precast section joints. This type of deterioration suggests improperly functioning joints, possible due to culvert settlements.
- Looking for longitudinal cracks along the barrel or cell length. These may be caused by excessive overloads or improper fill compaction of barrel sections, or lateral differential settlements of box sections. Medium to wide crack widths should be measured and recorded.
- Examination of box culvert tops when these elements double as the riding surface of traffic above. Longitudinal cracks in the culvert top may be caused by traffic overloads.
- Looking for transverse cracks. These may be caused by differential settlements along the culvert length, often the result of improper bedding material preparation.
- Checking the entire barrel or cell for delaminating, spalls, exposed strands, cracking and leaching. Note the location of such defects.
- Examining the inlet and outlet of the barrel for signs of scour or undermining underneath of a closed pipe. This could lead to piping.
- Examining for scour or footing undermining along the length of arch culverts. Undermining may allow the concrete footing to rotate, failing the arch.

- Looking for signs of abrasion along the length of the culvert.

- Examination of end treatments or wingwalls for damage, deterioration, debris accumulation, streambed scour, undermining, and embankment erosion. These deficiencies can cause hydraulic inefficiencies of the culvert, and roadway settlements.

- Checking for excessive joint openings between headwalls, wingwalls, apron, etc.

4.6.6.6 Culvert Liner (Element 8802)

Culvert Liner is a Strengthening and Repair Element available only under Culvert Elements. They add structural stability to a culvert. This element will be used in conjunction with the main culvert element as a "protective system". Refer to Section 2.6.5.3 for additional information on this element.

In most cases, a culvert liner will not increase the Condition State of the culvert element itself. For instance, a corrugated steel pipe that is corroded through the bottom (Condition State 4 Corrosion) with a culvert liner is installed would have the Steel Culvert element remain in Condition State 4 on the inspection report while the Culvert Liner element would now become the structural element to evaluate during inspections. The liner essentially becomes the main structural component while the previous culvert element remains as a place holder.

Element Level Inspection

On the inspection report form, culvert liners are recorded in units of linear feet. It is important to note that the culvert liner may exhibit more than one defect along its length. It is the inspector's task to examine each element, determine the applicable defects and reasonably assign the appropriate Condition State to each defect. Where multiple defects exist within a unit of measure, the inspector shall refer to the Defect Hierarchy as described in Part 2 Chapter 3. The sum of all of the reported material defect condition states must equal the total quantity of the element. This will quantify the element's state of deterioration and help generate quantity/cost estimates for future remedial work.

As a strengthening and repair system element, Culvert Liner essentially breaks down into more simply whether or not the structure is remaining effective. The inspector should take care to note any distortion or bending that may occur in the liner after its installation as this may be a sign of impending failure.

When Culvert Liners are inspected in the field, the inspector should focus his/her energies on the liner itself and no longer on the material condition of the previous culvert element. Once the liner is installed, it is assumed the liner is the new culvert element. Existing headwall configurations, unless also repaired, should still be inspected thoroughly by the inspector as these retain the roadway fill and provide protection to the embankments.

4.6.6.7 Integral Wingwall (Element 8400)

This element defines the wingwalls integral with the box culvert apron, headwall or barrel. The intent of wingwalls is to retain the roadway or embankment fills adjacent to the barrel opening. A wingwall not functioning as intended may permit the loss of fill around the barrel and may occlude the barrel opening reducing hydraulic capacity. The wingwalls extend past the end of

the barrel typically flared out, however wingwalls may also be found running parallel with the channel or parallel to the roadway. Typically a joint is found between the end of the barrel and the wingwall. The inspector shall report the wingwall as integral so long as it is anchored to apron of the end treatment. In the cases where the wingwalls are comprised of different materials and consequently not anchored to the apron or barrel of the culvert, the wingwalls shall be inspected and noted under the Culvert End Treatment assessment.

Element Level Inspection

On the inspection report form, integral wingwalls are recorded in units of “each”. There are two material defects for integral wingwalls, Wingwall Movement and Wingwall Deterioration. As in all cases, the most severe defect Condition State, as determined by the Defect Hierarchy, is assigned to the entire element. This will quantify the wingwall’s condition and help to generate quantity/cost estimates for future remedial work.

In general, wingwalls slope down from the top of the headwall. In some instances, the wingwalls may be incorporated as retaining structures that continue well beyond the box culvert structure. While the inspector should perform his/her due diligence in the evalulation of the retaining structure adjacent to the box culvert, the inspector need only inspect a portion of the wall under the Integral Wingwall element. The length inspected should be 20 feet from the end of the barrel or the length of a 2:1 slope from the top of the headwall, whichever is greater. The remaining length should then be considered retaining wall and shall be inspected under a separate R-number structure inspection.

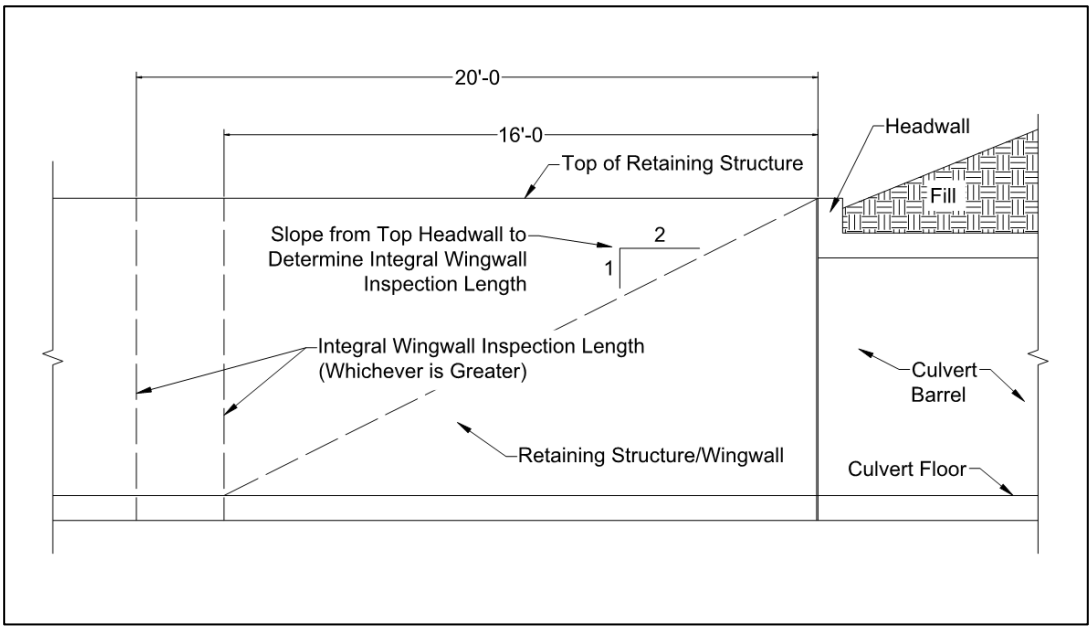

Figure 4.6.6-11: Determination of Integral Wingwall Inspection Limit When Wingwall Part of Larger Retaining Structure. For This Example, the Inspector Would Evaluate From Joint to 20’-0 Out (as it is greater than the 2:1 slope from the top of headwall) as Integral Wingwall.

Figure 4.6.6-12: View of Integral Wingwalls Parallel with Roadway.

Figure 4.6.6-13: View of Retaining walls extending beyond limits of integral wingwalls.

Maintenance inspection of integral wingwalls should include the following items:

- Visual examination of the material condition of the wingwall. Look for cracking, efflorescence, rust staining, spalling, corrosion, etc.
- Verify the vertical plumbness of the wingwall wall. Common tools for verification may include a level or plumb bob.

- Inspect the joint between the barrel and wingwall. Check for loss of filler material within the joint. The caulking sealant should be adhered to the concrete on both sides.

- Check for rotation about the apron base. Rotation of the wingwall would be indicated at the joint between the barrel and wingwall. Measurements should be taken near the apron and near the headwall.

- Check for settlement of the apron and wingwall. This will be indicated by an observable drop between the barrel floor and the apron.

Figure 4.6.6-14: View of Wingwall Movement. Note loss of fill behind wingwall and culvert barrel – Condition State 3.

4.6.7 Small Bridge Material Deterioration

There are four general types of distress that any c-structure can experience. Refer to Figure 4.6.7-1 through Figure 4.6.7-3 for views of material distress. High earth loads, water velocities, long slender sections, and age can produce the following types of distress:

- **Shear or Bending Failures:** High embankments impose high loads on all sides of a culvert and can cause shear or bending failures. This may show up as excessive global flattening, sheared off bolts, or localized dents/bulges in steel culverts. Concrete culverts will exhibit longitudinal cracking, concrete crushing, or excessive deflections.

- **Settlement:** Foundation settlement may be seen as a sag or rise (when only the pipe ends settle) in steel culverts. Differential vertical or lateral displacements at construction joints and pipe segment joints, or pipe segment joints that have opened up suggest settlement of concrete culverts. Lateral movement of concrete box sections may also be observed.

- **Scour, Undermining, and Piping:** High water velocities within long, slender openings can result in scouring and undermining of the culvert ends. Scouring or undermining

may also occur along the footings when the culvert has no bottom. Undermining can lead to a loss of bearing, and subsequent structure settlement. Piping is the unintended flow of water around the outside of a culvert. It can begin at the inlet end where undermining under the barrel or end treatments allow water to flow. It may also occur at interior joints that have opened up due to soil settlements. Water inside of the culvert may flow out through the open joints. When this occurs, water flow can erode the soil surrounding the culvert, creating an artificial pipe. Piping can become a serious problem in that the piping area will start to erode, a culvert can lose its lateral soil support, and roadway settlements can occur.

- **General Deterioration:** Factors such as weathering, abrasion, chemical exposure, etc., cause deterioration of any construction material over time.

Figure 4.6.7-1: Concrete headwall failure. Note: Headwall deterioration would be captured under Assessment 9248 – Culvert End Treatment

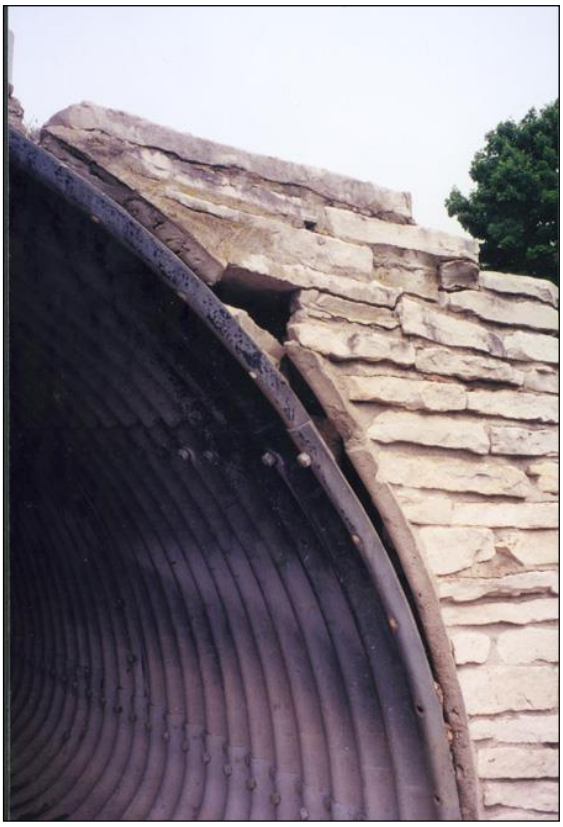

Figure 4.6.7-2: Masonry headwall separating from the barrel.

Figure 4.6.7-3: Through thickness section loss at water line of wall separating two cells.

4.6.8 Small Bridge Assessments

Assessments allow for the evaluation of secondary components not necessarily impacting the structural integrity of the structure. The inspector must be familiar with all the available assessments within the HSI system and appropriately capture them within the inspection report

for each culvert or small bridge structure. Evaluating assessments provides a more accurate picture of the structure being inspected and alerting the Department of potential future issues including traffic hazards or functionality of the structure. The following section describes the two assessments unique only to culvert inspections. All other assessments must be recorded and inspected. Refer to Part 2 Chapter 7 for the definitions and assessment states for all the available assessments within HSI.

4.6.8.1 Culvert End Treatments (Assessment 9248)

Culvert end treatments will encompass the headwall and apron on one end of the culvert. Culvert end treatments can be made of steel, concrete, timber and other materials and may be a combination of materials. Regardless of material composition, the headwall and/or apron shall be included under the end treatment assessment rating and evaluated based on its condition and function. Refer to Section 2.7.1.26 for additional information on this assessment. Wingwalls shall be evaluated separately under Integral Wingwall (Element 8400).

4.6.8.2 Roadway Over Structure (Assessment 9325)

Culverts and closed spandrel filled arch bridges can have several inches to several feet of soil overburden placed on top. Because of this, a deck is not required to carry the live loads to the structure. A conventional roadway pavement is simply built over the fill, and traffic live loads are delivered to the structure through the soil. Roadways over structures are important elements to inspect because distress found on a roadway often indicates more serious distress to the structure below.

The assessment “roadway over structure” is to be used only if there is more than 9 inches of fill (including the thickness of pavement, if present) over the structure at any point. For example, if there is only 6 inches of fill over the crown of a structural plate arch, but 8 feet 6 inches of fill at the footings, “roadway over structure” would need to be recorded. Refer to Section 2.7.1.32 for additional information on this assessment.

4.6.9 National Bridge Inventory (NBI) Overall Ratings

Part of every Routine inspection is rating the main structure according to the Federal Highway Administration (FHWA) General Condition Rating Guidelines. The numeric condition ratings of these guidelines describe the existing culvert as compared to its as-built condition. Ratings range from 9 to 0, with 9 describing components in excellent condition, and 0 describing failed components. Note that the NBI ratings are for small bridge – culvert structures and not small bridge – bridge like structures.

Because only a single number is used to rate the box/pipe culvert, the rating must characterize its overall general condition. The rating should not be used to describe local areas of deterioration, such as isolated heavy spalling. However, widespread heavy spalling would certainly influence the rating. A proper rating will therefore consider deterioration severity plus the extent to which it is distributed throughout the culvert.

NBI ratings are used to evaluate the state of deterioration of the culvert material. Since material condition is independent of a culvert’s load carrying capacity, postings or original design capacities less than current legal loads will not influence the rating. Similarly, temporary

substructure support does not change or improve the condition of the culvert material. Temporary strengthening methods will therefore not influence the culvert rating.

Because the NBI general condition ratings apply to a wide range of components and materials, the FHWA has developed supplemental rating guidelines within the *Culvert Inspection Manual*. These supplemental rating guidelines are used to assist the inspector in properly assigning condition ratings to culverts constructed of the most commonly used materials. The general condition ratings, along with the FHWA supplemental rating guidelines for culvert conditions, are given below. It should be noted that the supplemental rating guidelines only address corrugated steel and aluminum round pipe and pipe arches. The reader is referred to the *Culvert Inspection Manual* for supplemental rating guidelines for structural plate arch, metal box, low profile long span arch, high profile long span arch, pear shaped long span, and horizontal ellipse long span barrels.

Condition Rating	Description
N	Not applicable. Use if structure is not a culvert.
9	No deficiencies. **FHWA Supplemental Rating Guidelines:** **Steel Culvert –** New condition. **Cast-In-Place Concrete Culverts –** New condition. **Masonry Culvert Barrels –** New condition.
8	There are no noticeable or noteworthy deficiencies which affect the condition of the culvert. Insignificant scrape marks caused by drift. **FHWA Supplemental Rating Guidelines:** **Steel Culvert –** Good, superficial rust, no pitting, minor construction defects, protective coating intact. There is smooth curvature in the barrel, and horizontal shape measurements are within 10 percent of design for round pipes, 3 percent for pipe arches. Seams and joints are tight with no openings. **Cast-In-Place Concrete Culverts –** Good, no settlement or misalignment. Joints are tight with no defects apparent. No cracking, spalling, or scaling present; surface in good condition. The invert shows no signs of scour. **Masonry Culvert Barrels –** Good, no settlement or misalignment. Mortar is tight with no defects apparent. No cracking, missing, or dislocated masonry present; surface in good condition. The invert shows no signs of scour.

7 Shrinkage cracks, light scaling, and insignificant spalling which does not expose reinforcing steel. Insignificant damage caused by drift with no misalignment and not requiring corrective action. Some minor scouring has occurred near curtain walls, wingwalls, or pipes. Metal culverts have a smooth symmetrical curvature with superficial corrosion and no pitting.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Generally good, moderate rust, slight pitting. The protective coating is ineffective. The top half of the pipe is smooth but minor flattening of the bottom may exist for round pipe, and the bottom half is flattened but still curved for pipe arches. The horizontal shape measurements are within 10 percent of design for round pipes, and within 3 to 5 percent greater than design for pipe arches. Seams and joints have minor cracking at a few bolt holes, minor openings, and a potential for backfill infiltration.

Cast-In-Place Concrete Culverts – Generally good, no settlement, minor misalignment at joints. Joint material is deteriorated at isolated locations. Minor hairline cracking at isolated locations; slight spalling or scaling present on invert or bottom of top slab. Minor scour at the invert.

Masonry Culvert Barrels – Generally good, no settlement, minor misalignment at joints. Shallow mortar deterioration at isolated locations. Surface deterioration or masonry at isolated locations. Minor scour at the invert.

6 Deterioration or initial disintegration, minor chloride contamination, cracking with some leaching, or spalls on concrete or masonry walls and slabs. Local minor scouring at curtain walls, wingwalls, or pipes. Metal culverts have a smooth curvature, non-symmetrical shape, significant corrosion or moderate pitting.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Fair, fairly heavy rust, moderate pitting, slight thinning. The top half of the pipe has a smooth curvature but the bottom half has flattened significantly for round pipe, or has flattened for pipe arches. The horizontal shape measurements are within 10 percent of design for round pipe, and 5 percent greater than design for pipe arches. Minor cracking at the bolts is prevalent in one seam in the lower half of the pipe. Evidence of backfill infiltration at the joints and seams.

Cast-In-Place Concrete Culverts – Fair, minor misalignment and settlement at isolated locations. Joint material generally deteriorated, minor separation, possible infiltration or exfiltration; minor cracking or spalling at joints allowing exfiltration. Extensive hairline cracks, some with minor delaminations; invert or bottom of top slab scaling less than ¼ inch deep or small spalls present. Minor scour near footings.

Masonry Culvert Barrels – Fair, minor misalignment and settlement. Mortar has extensive areas of shallow deterioration; missing mortar at isolated locations; possible infiltration or exfiltration; minor cracking. Minor cracking of masonry units. Minor scour near footings.

5 Moderate to major deterioration or disintegration, extensive cracking and leaching, or spalls on concrete or masonry walls and slabs. Minor settlement or misalignment. Noticeable scouring or erosion at curtain walls, wingwalls, or pipes. Metal culverts have significant distortion and deflection in one section, significant corrosion or deep pitting.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Generally fair, extensive heavy rust, deep pitting, moderate thinning. Significant distortion at isolated locations in the top half and extreme flattening of the invert for round pipe, or slight reverse curvature in one location for pipe arches. The horizontal shape measurements are between 10 to 15 percent greater than design for round pipe, or within 5 to 7 percent for pipe arches. Moderate cracking at the bolt holes in one seam near the bottom of pipe. Deflection of pipe caused by backfill infiltration through joints and seams.

Cast-In-Place Concrete Culverts – Generally fair, minor misalignment or settlement; possible piping. Joints are open and allowing backfill to infiltrate; significant cracking or spalling at joints. Crack widths greater than 1/8 inch with significant delamination and moderate spalling exposing reinforcing steel; large areas of surface scaling greater than ¼ inch deep. Moderate scour along footing; protective measures may be required.

Masonry Culvert Barrels – Generally fair, minor misalignment or settlement. Mortar generally deteriorated, loose or missing mortar at isolated locations, infiltration staining apparent. Minor cracking and slight dislocation of masonry units; large areas of surface scaling. Moderate scour along footing; protective measures may be required.

4 Large spalls, heavy scaling, wide cracks, considerable efflorescence, or opened construction joint permitting loss of backfill. Considerable settlement or misalignment. Considerable scouring or erosion at curtain walls, wingwalls, or pipes. Metal culverts have significant distortion and deflection throughout, extensive corrosion or deep pitting.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Marginal, pronounced thinning (some deflection or penetration when struck with pick hammer). For round pipes, significant distortion throughout length of pipe, lower third may be kinked. For pipe arches, significant distortion along the top of arch, bottom has reverse

curvature. The horizontal shape measurements are between 10 to 15 percent greater than design for round pipe, and more that 7 percent greater for pipe arches. Moderate cracking at the bolt holes on one seam near top of pipe. Deflection caused by backfill infiltration through open joints and seams.

Cast-In-Place Concrete Culverts – Marginal, significant misalignment and settlement of the pipe; evidence of piping. Differential movement and separation of the joints, significant infiltration and exfiltration at joints. Extensive cracking with crack widths greater than 1/8 inch with efflorescence; spalling has caused exposure of reinforcing steel which are heavily corroded; extensive surface scaling on invert greater than 1/2 inch deep. Scour along footing with slight undermining, protection required.

Masonry Culvert Barrels – Marginal, significant misalignment and settlement. Mortar severely deteriorated, significant loss of mortar, significant infiltration and exfiltration between masonry units. Significant displacement of individual masonry units. Scour along footing with slight undermining, protection required.

3 Any condition described in Condition Rating 4 but which is excessive in scope. Severe movement of differential settlement of the segments, or loss of fill. Holes may exist in walls or slabs. Integral wingwalls nearly severed from culvert. Severe scour or erosion at curtain walls, wingwalls or pipes. Metal culverts have extreme distortion and deflection, in one section , extensive corrosion, or deep pitting with scattered perforations.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Poor, extensive heavy rust, deep pitting, scattered perforations. For circular pipes, extreme deflection at isolated locations, flattening of crown, crown radius 20 to 30 feet. For pipe arches, extreme deflection in top arch in one section; bottom has reverse curvature throughout. The horizontal shape measurements are in excess of 15 percent greater than design for circular pipes, and over 7 percent greater for pipe arches. Three inch long cracks at bolt holes on one seam.

Cast-In-Place Concrete Culverts – Poor, significant ponding of water due to sagging and misalignment of the pipe; end section drop off has occurred. Significant joint openings and differential movement, infiltration or exfiltration causing misalignment of culvert and depressions or settlement in the roadway. Extensive cracking with spalling, delamination and slight differential movement; scaling has exposed reinforcing steel in bottom of top slab or invert. Severe footing undermining with slight differential settlement causing minor cracking or spalling in footing and walls.

Masonry Culvert Barrels – Poor, significant ponding of water due to sagging and misalignment of the pipe; end section drop off has occurred. Extensive areas of missing mortar, infiltration or exfiltration causing misalignment of

culvert and depressions or settlement in the roadway. Individual masonry units in lower part of structure missing or crushed. Severe footing undermining with slight differential settlement causing minor cracking or spalling in footing and minor distress in walls.

2 Integral wingwalls collapsed, severe settlement of roadway due to loss of fill. Section of culvert may have failed and can no longer support embankment. Complete undermining at curtain walls and pipes. Corrective action required to maintain traffic. Metal culverts have extreme distortion and deflection throughout with extensive perforations due to corrosion.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Critical, extensive perforations due to rust. For circular pipe, extreme deflection and distortion throughout pipe, flattening of crown, crown radius over 30 feet. For pipe arches, extreme deflection along top of pipe. The horizontal shape measurements are in excess of 20 percent greater than design for circular pipes, and 7 percent greater for pipe arches. Plate cracks from bolt to bolt on one seam.

Cast-In-Place Concrete Culverts – Critical, culvert not functioning due to misalignment. Severe cracks with significant differential movement; concrete completely deteriorated in isolated locations in top slab or invert. Severe footing undermining with significant differential settlement causing severe cracks.

Masonry Culvert Barrels – Critical, culvert not functioning due to misalignment. Individual masonry units in top of culvert missing or crushed. Severe footing undermining with significant differential settlement causing severe cracks in footing and distress in walls.

1 Bridge closed. Corrective action may put it back in light service.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Critical, invert completely deteriorated. For circular pipes, partially collapsed crown in reverse curvature. For pipe arches, the structure is partially collapsed. Seams have failed.

Cast-In-Place Concrete Culverts – Culvert is partially collapsed, roadway is closed to traffic. Severe footing undermining resulting in partial collapse of the structure.

Masonry Culvert Barrels – Culvert is partially collapsed, roadway is closed to traffic. Severe footing undermining resulting in partial collapse of the structure.

0 Bridge closed. Replacement is necessary.

FHWA Supplemental Rating Guidelines:

Steel Culvert – Critical, partial or complete collapse.

Cast-In-Place Concrete Culverts – Total failure of culvert and fill, roadway is closed to traffic.

Masonry Culvert Barrels – Total failure of culvert and fill, roadway is closed to traffic.

The above FHWA supplemental rating guidelines for steel and aluminum culvert shapes apply only to the commonly found circular and pipe arch sections. Additional shape guidelines for structural plate arches, corrugated metal box culverts, low profile arch long-span culverts, high profile arch long-span culverts, pear shaped long-span culverts, and horizontal ellipse long-span culverts are beyond the scope of this Manual, but may be found in the Culvert Inspection Manual.

One suggested method for establishing a culvert rating is to identify phrases within the general condition/FHWA supplemental guideline language that describes a culvert condition rating more severe than what actually exists. The correct rating number will be one number higher than the one describing the more severe condition.

For example, suppose a structural steel plate pipe culvert has surface corrosion at the invert and waterline, plus random joint bolts that have minor to moderate corrosion. The seams are tight. The culvert is an 8 foot wide arch pipe, with a measured horizontal dimension of 8 feet 4 inches (4.2 percent greater than the design value). Condition Rating 7 indicates that there is moderate rust or pitting, and the horizontal dimension of the culvert is between 3 to 5 percent of the design value. There may be minor openings at the seams. Condition Rating 6 indicates that there is fairly heavy rust with minor pitting and slight thinning. There may be evidence of backfill infiltration at the seams, and the horizontal dimension of the culvert is greater than 5 percent of the design value. Using the method described above, Condition Rating 6 describes a situation more severe than what actually exists on the substructure. Therefore, a rating of 7 would be appropriate.

Another method to help narrow down the culvert condition rating number is to group the numbers in more general categories. Ratings of 9 to 7 apply to culverts in good condition, 6 to 5 suggest fair condition, 4 to 3 suggest poor condition, 2 suggest poor/critical condition, and 1 to 0 suggest critical condition.

4.6.10 Inspection of Roadway Approach and Embankment

Deficiencies in the roadway and embankment may be indicators of structural or hydraulic problems in the culvert. Refer to Section 4.6.5.5 of this Chapter and Part 2, Chapter 6 for additional information on the inspection of Roadway Approach and Embankment.

4.6.11 Inspection of Waterway

The channel conditions and waterway adequacy should be evaluated during the inspection. Refer to Part 2, Chapter 8 for a description of these inspection procedures. Culverts, or structures with a structural floor do not require a streambed profile. The ends of each Culvert End Treatment, however shall receive an underwater probe inspection to determine streambed stability or scour hole development.

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4.8 OTHER STRUCTURES OVER ROADWAYS

4.8.1 Introduction

This chapter deals with structures that have not already been discussed but require review since their condition may affect the traveling public on the roadway underneath the structure. These structures include railway bridges, pedestrian bridges, and pipeline bridges (utility cable structures, high voltage towers, and telecommunication towers are not covered in this Manual). Although the Wisconsin Department of Transportation (WisDOT) or local government agencies may not own these structures, their safety must be assessed if they encroach the travelled right of way. Many WisDOT-owned structures will have an Identification Plaque mounted to the structure. However, private owners may also have identification plaques attached to their structures. The inspector should be familiar with the standard WisDOT ID plaque format and be able to differentiate these from private owner ID Plaques. Refer to Part 1, Chapter 5, for an explanation of the WisDOT numbering system.

The structures discussed in this chapter are an additional safety check for the traveling public, and do not limit the ultimate responsibility of the structure owner to ensure a safe structure. WisDOT may perform the inspection of the structures through cooperative inspection efforts with the owning agency or through independent inspections. The level of inspection effort for these structures may vary, and will typically be owner-specific and site-specific. Cursory inspections from the underside of the structure in conjunction with reviewing the owner's latest inspection report may be sufficient to confirm the safety of the structure. However, the inspector should never enter private property without the owner's consent.

4.8.2 Railway Bridges

The inspection of the materials for a railway bridge is similar to that of a highway bridge. Refer to Part 2 of this Manual for a discussion on bridge inspection. Additionally, this Section will focus on the inspection of elements and evaluation of problems which are unique to a railway bridge. Refer to Figure 4.8.2-1 and Figure 4.8.2-2 for views of a typical railway bridge.

Figure 4.8.2-1: Railroad Bridge Top View.

Figure 4.8.2-2: Railroad Bridge Elevation View.

4.8.2.1 Railway Bridge Loads

All bridges, whether railway or vehicular incur many of the same types of loading. Of course, the magnitude of these loads will differ for a railway bridge due to its specific type of usage. The greatest difference between railway and roadway bridges is the increased vertical live, impact, centrifugal, and longitudinal (braking) load. Furthermore, railway bridges often experience larger thermal forces and vibration.

4.8.2.2 Railway Bridge Inspection Reporting

Although WisDOT owns several railway bridges, private railroad companies own most of the railway bridges in Wisconsin. The American Railway Engineering and Maintenance-Of-Way Association (AREMA) recommend a 12-month annual inspection program be completed by the owners of railway bridges. At times, because of poor condition or low rating capacity, inspections may be scheduled at 3, 4, or 6-month intervals. Railway owners may provide only cursory inspections, instead of detailed inspections. Additionally, these inspection reports vary in format and content from one company to another company. The owner of the railway structure is responsible for the safety associated with the bridge. Any additional inspections by others related to the safety of the traveling public under the bridge should be considered a supplemental bridge inspection. This supplemental bridge inspection data should be forwarded to the bridge owner if a serious safety condition exists.

During a supplemental inspection, the inspector should report indications of failure in any portion of the structure and any conditions that could contribute to a future failure. Likewise, any loose or corroded elements in danger of falling onto traffic lanes below should be noted. Most frequently, the Wisconsin Department of Transportation (WisDOT) became involved in private railway bridge inspections after receiving complaints from passing motorists about falling rivets, bolts, and/or nuts. If possible, structures should be observed during passage of a train, so that the effects of vibrations, sideways, and deflections may be noted. All pertinent

defects should be noted and recorded, and reference points established for notification of the railway bridge owner.

4.8.2.3 Railway Bridge Inspections

The railway track approach immediately adjacent to the abutment and back wall is one of the most important areas of a railway bridge and is probably one that receives the least attention. On bridges having inner guardrails, the track bed is never tamped because the tamper blades will not fit within the guardrails. Typically this area never receives the attention that it should and the track surface remains low. If there is no inner guardrail and the bridge is located within the limits of a surfacing program there is a tendency to raise the track to the back wall and leave the bridge in a "hole". When surfacing is properly performed, the tamper is turned and the approach is raised away from the bridge, providing a smooth transition from the level bridge to the newly surfaced track.

When the approach surface is allowed to remain low, large forces are applied to the back wall and bridge seat by passing trains. The results are broken back wall ties, cracked and failed back walls, damaged bridge seats, etc. The secondary batter from the mechanical equipment tends to damage bridge ties and cause additional impact to the bridge seat. It is vitally important that the approaches to the bridge be kept level and smooth.

When inspecting bridges, the approaches should be observed for the following serious conditions:

1. The approach is excessively low or out of cross level or both.
2. Broken track and back wall ties in the approach area.
3. Severe muddy condition in the track at the approach and back wall (suggesting approach settlement).
4. Bent or broken rails.
5. Missing or broken track components, angle bars, bolts, spikes, etc.

4.8.3 Pedestrian Bridges

While this section is located under Chapter 8 (Other Structures over Roadways), it is also applicable for pedestrian bridges over other obstacles (e.g., a waterway) and can be implemented by the maintaining agency/owner as deemed appropriate. The inspection and evaluation of a pedestrian bridge is similar to that of the bridges discussed in Part 2 of this Manual. Pedestrian bridges over roadways should be inspected at a 48-month maximum interval with special inspections for problem areas. Often times fencing is provided on the sides of the bridge. This makes it difficult or impossible for the inspector to look over the side. Therefore, additional access equipment may be necessary to properly inspect the underside of the bridge. Traffic control is often necessary when additional access equipment is used. Refer to Figure 4.8.3-1 through Figure 4.8.3-2 for an overall view of a pedestrian bridge and the bridge's approach ramp.

Figure 4.8.3-1: Overall View of Pedestrian Bridge.

Figure 4.8.3-2: Pedestrian Bridge Entrance/Exit Ramp.

Since the pedestrian bridge will not encounter the same loadings that vehicle or railway bridges encounter, some pedestrian bridges are constructed with a non-redundant girder system. A non-redundant system will only have a one or two girder superstructure, thus making the bridge a nonredundant steel tension member (NSTM). Since some pedestrian bridges are non-redundant, the inspector should pay extra attention to the bridge girder system to ensure excessive corrosion or cracking is not occurring. Refer to Figure 4.8.3-3 and Figure 4.8.3-4 for examples of NSTM pedestrian bridges.

Figure 4.8.3-3: Non-Redundant Steel Girder Pedestrian Bridge.

Figure 4.8.3-4: Steel Thru Truss Pedestrian Bridge.

4.8.4 Pipelines

Non-Wisconsin Department of Transportation (WisDOT) pipelines are typically owned by either a transport company that moves a bulk product, or a distribution company that sells the product. Both of these types of owners operate pipelines in the State. Their product normally consists of a fuel, such as gasoline, fuel oils, propane, or natural gas.

The inspector should familiarize himself with the products carried in these pipelines and be aware of the signs of a leak. Not all these products will have a tell-tale style “natural gas smell”, especially bulk natural gas which does not have this odor added until it is processed by the

distribution company. If a leak is detected the inspector should immediately leave the site by foot, and then contact the owner. It is important to leave the site without introducing an ignition source, such as sparks created by starting a vehicle or using a cell phone. The inspector should also be familiar with safety procedures with respect to leaks involving these fuels. The inspection should not continue until the owner has contained the leak and given notice to proceed.

The inspection of a pipeline bridge is similar to that of the bridges discussed in Part 2 of this Manual. Pipelines should be inspected at 24-month intervals with special inspections for problem areas.

Pipelines should be inspected for material deterioration and any loose bolts or pieces of material that could fall on to the roadway below. In addition, the inspector should look for sags in the pipe and inspect the structure's cables (for cable supported pipes) and foundations. Refer to Figure 4.8.4-1 for an end view of a pipeline bridge.

Figure 4.8.4-1: End View of a Pipeline Bridge.

Pipelines are most often connected to a bridge using a saddle/clamping system. Refer to Figure 4.8.4-2 for a view of a typical pipeline-to-bridge connection. This bridge uses U-bolts to hold the pipeline to the bridge. All connections should be inspected to ensure the pipeline is securely fastened. Any loose or missing connections should be noted in the inspection report.

Figure 4.8.4-2: Pipeline to Bridge Connection.

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5.1 GENERAL OVERVIEW AND SUMMARY TABLE

5.1.1 Introduction

Structures of all types are susceptible to a range of defects (flaws and discontinuities). Some defects are unknowingly built into a structure, while others develop with time due to natural or manmade events. While some of these defects may be detected by visual or tactile means, others may be hidden within the structure's members or components. In some cases, an indication of a defect may be readily observed, but the true extent or cause of the defect is not evident. For these cases, the use of nondestructive evaluation (NDE) methods or partially destructive evaluation (PDE) methods can be used to more thoroughly investigate a structure's condition. NDEs permit the inspection of an element without inflicting damage, while PDEs typically cause minor localized, repairable damage. Specifically, NDE and PDE are used to investigate the material integrity of the test component and not the function of the component beyond material failure.

Part 5 presents an overview of NDE techniques as they apply to bridges and miscellaneous support systems. Although NDE is often performed by specialists, all structure inspectors should be familiar with available NDE techniques so they can recommend appropriate testing procedures and recognize the limitations of the data.

A variety of NDE and PDE techniques have been developed and commonly employed in the inspection of structures. While several of these techniques are described in Part 5, the inspector should be aware that many other methods are also currently available and new techniques are constantly being developed. Refer to Figure 5.1.1-1 for a brief comparison of the techniques covered in Part 5.

Chapter	Method	Nondestructive	Partially Destructive	Material			Desired Information, Measured Defect or Material Property
				Concrete	Steel	Timber	
2	Visual Inspection	X		X	X	X	Overall external material deterioration.
3	Audible Inspection	X		X	X	X	Concrete delamination; steel bolt and rivet looseness; timber integrity.
4	Liquid Penetrant	X			X		Crack or flaw detection in non-porous metals. Including Aluminum.
5	Ultrasonic Testing (Incl. Phased Array)	X		X	X	X	Non-homogenous investigation, delamination, surface crack and weld discontinuity detection. Including thickness reading to determine the extent of section loss. Steel welds and bridge pins volumetric tests.
6	Acoustic Emission	X		X	X	X	Crack, corrosion, weld defects and material embrittlement.
7	Magnetic Particle	X			X		Crack or flaw detection in ferrous metals.
8	Eddy Current	X			X		Crack or near surface defect detection in conductive metals. Can be performed through paint, but is limited on rough surfaces.
9	Radiography	X		X	X		Crack or flaw detection in ferrous metals. Internal defects, reinforcing layout in concrete.
10	Ground Penetrating Radar	X		X			Concrete deck voids, overlay thickness, and reinforcing steel location; soil/foundation engineering; and underwater profiling/scour location.
11	Infrared Thermography	X		X			Concrete deck delamination. Other material debondment (FRP wraps)
12	Impact Echo Testing	X		X			Internal concrete flaw detection.
13	Ultrasonic Pulse Velocity						Internal concrete flaw detection.
14	R-Meter Testing	X		X			Reinforcing steel concrete cover, location and size.
15	Half-Cell Testing	X		X			Corrosion of reinforcing steel in concrete.
16	Chloride Ion Testing		X	X			Corrosion of reinforcing steel in concrete.
17	Carbonation Testing		X	X			Depth of pH decrease and susceptibility of steel reinforcement to corrosion.
18	Penetration Hammer (Schmidt Hammer)	X		X			In-situ, relative surface hardness of concrete.
19	Windsor Probe		X	X			In-Situ, relative penetration resistance.
20	Resistance micro drill	X				X	Extent of decay in Timber
21	Stress Wave Timers	X				X	Extent of decay in Timber
22	Monitoring Systems	X		X	X	X	Measurement of loads and strains to calculate stresses in members.
23	Material Sampling		X	X	X	X	Various material properties and defects.
24	Unknown Foundation Investigation	X	X	X	X	X	Determination of foundation type or length of piles.
25	Hydrographic Surveys and side scanning sonar	X					Hydrographic surveys map a channel bottom and do not specifically test a structural component. However, locating areas of scour that can undermine substructure units allows the inspector to assess the integrity of the structure.

Figure 5.1.1-1: Summary of NDT & PDT Methods in Part 5.

5.1.2 NDE and PDE Inspector Qualifications

Nondestructive and partially destructive evaluation methods range from simple chain drags and timber coring to complex methods such as ultrasonic and Windsor Probe Testing. It is important that the person conducting the test, as well as the personnel interpreting the test data, be properly trained in the applied method. Additional qualifications should include both an understanding of the theory behind the test and practical experience. Inspection methods should be conducted in accordance with applicable American Society for Nondestructive Testing (ASNT) written practice in accordance with document (SNT-TC-1a) and written and approved procedures. Other testing methods should follow written procedures and sound engineering practices as found in American Society for Testing and Materials (ASTM) standards, and American Association of State Highway and Transportation Officials (AASHTO) specifications.

Refer to Part 1 of this Manual for the minimum standards for inspector qualification. Furthermore, Part 1 of this Manual discusses the requirements for submitting written practices and personnel certifications to WisDOT for review. Nationally recognized certifications in NDE are provided through ASNT and require a written practice in accordance with SNT-TC-1a. The SNT-TC-1a is a guideline for an employer based written practice that includes the qualification and education requirements to be certified to the employers written practice. The SNT-TC-1a allows for training requirements to be modified as allowed by the written practice, which is an agreement between the NDT provider and the customer.

Many tests are particular to bridges, such as thermography and ground penetrating radar testing of bridge decks, are not covered by ASNT certifications; however, these tests are covered by the ASTM standards and experience can be documented.

5.1.2.1 Specialists

NDE: NDE personnel shall be qualified in accordance with nationally recognized NDE personnel qualifications practice or standards such as ANSI/ANST-CP-189, SNT-TC-1A, MIL STD 410, NAS-410 or a similar document. The practice or standard used and the applicable revision shall be specified in the contractual agreement between the structure owner and the NDE inspector.

NDE Requirements: Consultants, contractors, and their subcontractors performing NDE magnetic particle testing (MT), liquid (dye) penetrant testing (PT) or ultrasonic testing (UT)] on bridges or miscellaneous structures as defined in the Wisconsin Structure Inspection Manual shall maintain their written practice and personnel certifications. And are required to produce them for review..

Consultants, contractors, and their subcontractors performing NDE (visual inspection) on bridges or miscellaneous structures, as defined in the Wisconsin Structure Inspection Manual, shall have taken and passed the NHI 80-hour course (Safety Inspection of In-service Bridges) or be certified as an American Welding Society Certified Welding Inspector (AWS/CWI) or submit their written practice and certifications documenting the equivalent training that qualifies their personnel to perform visual inspection on structures in the state of Wisconsin. The third option must be approved by the Wisconsin Department of Transportation's (WisDOT)

Statewide Inspection Program Manager, or their designee, for approval prior to performing any work.

5.1.3 Data Collection and Interpretation

Upon embarking on a nondestructive or partially destructive evaluation program, a plan should be developed which details the type(s) of testing to be performed, amount of data needed, test locations, criteria for data interpretation, and follow-up procedures for handling unanticipated test results. Many testing methods produce considerable test data. Data should be collected on the applicable forms, which include the location and test results. When possible, copies of all field data should be retained as part of the structure's official file.

Interpretation of nondestructive evaluation (NDE)/partially destructive evaluation (PDE) data should be performed by persons knowledgeable both in the test theory and in the analysis or evaluation of the structure being tested. For some tests, such as ultrasonic weld inspection, recognized criteria exists for evaluating the effects of any detected anomalies. However, for many other test methods, the NDT/PDT data must be evaluated based on each individual structure's behavior.

Most nondestructive programs detect and assist in the evaluation of flaws and discontinuities, as well as determining the strength or serviceability, by indirect methods. The tests typically indicate the existence, extent, and location of discontinuities. However, the influence of the discontinuity on the strength or serviceability of the structural component is often more difficult to determine. The validity of an NDE test or NDT depends on good engineering judgment based on experience, additional NDT, or fully destructive testing results. Note: NDE is nondestructive evaluation and may use many tools in the toolbox such as additional test of NDT. The information collected by NDE/PDE is typically raw data and the specialist must interpret the information to correlate it to a usable parameter. Also, certain techniques may provide false data under certain conditions; therefore, the specialist must be familiar with a technique and recognize the false readings. False positives from inexperience persons and poor procedures can result in over testing and repair of components that have no real problems. The understanding of how structures load paths, AASHTO fatigue details and the interaction of various loads can also play into the decision making process which often results in the extent of repairs and life expectance of a structure. Poor procedure and inexperience and non-qualified personnel can lead to false assurance and unanticipated failures. Likewise, the inspector should not recommend testing, or accept testing results, without being familiar with the technique. Adverse conditions and human factors such or rushed or incomplete NDE can result in missed flaws or discontinuities that could lead to failures. Critical members that cannot be tested due to bird droppings, inaccessibility, or hidden feature should be carefully considered for more extraordinary measure to ensure quality and integrity of the member in question. These efforts may include steam cleaning, sand blasting, paint removal, remote inspection methods, strain gages and finite element analysis. These used in addition to systematic and sound engineering practices and experienced personnel and diligent research may be necessary to ensure reasonable safety, and integrity.

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5.2 VISUAL INSPECTION

5.2.1 Introduction

Visual Examination is the most basic of nondestructive inspection techniques, and utilizes the human eye. It allows for detecting and inspecting a wide variety of surface flaws such as cracks, discontinuities, corrosion, and contamination. Detection of surface cracks is particularly important due to their relationship to structural failure mechanisms. Even when other nondestructive evaluation (NDE) techniques are employed, visual examination is a needed supplement, such as in Liquid Penetrant Testing. When conducted under proper conditions, liquid penetrant testing is capable of highlighting minor surface flaws and provides a basis for detailed visual examination. Close visual examination requires proper access to the element being inspected and appropriate tools. Refer to Figure 5.2.1-1 for a view of an inspector conducting a visual inspection.

Figure 5.2.1-1: Inspector Conducting Visual Inspection.

5.2.2 Applications

Visual examination is applicable to all structures, either as a Routine Inspection or as the first step in any In-depth Inspection. It can identify where a failure is most likely to occur and identify when a failure has commenced. Visual examination is often enhanced by other surface methods of inspection, which can identify defects that are not easily seen by the unaided eye. Furthermore, visual examination may be aided with magnifying lens equipment or tools.

Any inspection should always proceed in a logical manner from element to element. Proper access is necessary so that the inspector can examine elements from a reasonable distance during Routine Inspections and from a maximum “arm’s length” (approximately 2 feet) distance during Nonredundant Steel Tension Member (NSTM) and In-depth Inspections. Surfaces must be properly cleaned to expose the base material. The element being

inspected should also be well lit, either naturally or by the use of a portable light source. The inspector should have good vision (corrected if necessary) and color acuity. The inspector should also possess knowledge of the types of failures to look for. A variety of tools should be employed, when necessary, to aid the inspector. These tools may include binoculars, low power magnifying glasses, crack gages, and boroscopes to view inaccessible areas. Access equipment should also be used to allow the inspector reasonable proximity to the element being inspected.

5.2.3 Limitations

Because visual examination can only detect defects visible to the eye, the internal condition of an element remains unexamined. In addition, some small surface flaws may be difficult to locate, or once found, it may be difficult to accurately determine their extent. Badly deteriorated elements may be difficult to examine effectively due to heavy corrosion buildup. Furthermore, access to visually examine an item may be difficult even with the use of specialized equipment.

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5.3 AUDIBLE INSPECTIONS

5.3.1 Introduction

The two most common types of audible inspections are “Chain Dragging” and “Hammer Sounding.” Chain Dragging is normally used on large concrete surface areas, such as bridge decks, while Hammer Sounding can be used on a number of materials in random locations. Both methods typically rely on the experience of the inspector to differentiate the relative sounds of similar materials.

The Chain Dragging is a simple method allowing the inspector to drag several lengths of heavy chain over a concrete surface. The chains contact the concrete surface and produce an audible indication of delaminated areas, much like tapping with a hammer. These areas are marked and mapped for further evaluation.

Hammer Sounding can be an extension of visual inspection by incorporating an additional sense, hearing. Hammer Sounding can aid in detecting impending spalls and existing delaminations, which when struck with a hammer, give off a dull sound or loud pop as opposed to the sharp ring of hard concrete without any internal discontinuities. This sound is easily noted when progressing from solid areas to delaminated areas. Similarly, hammer sounding of bolts and rivets can also serve as an aid to detecting loose fasteners. Bolts and rivets should be struck sideways as well as on their ends. In the inspection of timber elements, hammer sounding is used to detect the presence of significant decay. When a seriously decayed member is struck with the hammer, a dull or hollow sound is produced. However, when suspected decay is encountered, it must be verified by other means such as boring or coring. In general, hammer sounding can be satisfactorily accomplished using a relatively light hammer. Refer to Figure 5.3.1-1 for a view of an inspector hammer sounding a concrete abutment.

The audible inspection procedure for bridge decks is covered in American Society for Testing and Materials (ASTM) D4580-86 (1997) “Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding.”

Figure 5.3.1-1: Inspector Hammer Sounding a Bridge Abutment.

5.3.2 Applications

Chain Dragging is most commonly used as an aid in inspecting concrete bridge decks. However, it can be used on other horizontal surfaces. Because several chains can be incorporated into the drag, large areas can be quickly examined. This method is most efficiently conducted using a two-person team.

Hammer Sounding is most commonly used as an aid in inspecting concrete, but can also be used on metal fasteners and timber members. This technique works well on both horizontal and vertical surfaces. Large areas can be inspected in a reasonable amount of time only if spacing is random; however, the inspector should take care to thoroughly cover the entire surface of the element since the hammer only provides information on the local area under the point of impact. A single inspector can conduct this method of investigation, but access equipment may be needed.

Both techniques require minimal training; however, the inspector should be familiar with the tonal differences between sound and delaminated concrete.

5.3.3 Limitations

Chain dragging is limited to locating areas of delamination in exposed concrete bridge decks. It is not effective on asphalt-overlaid decks, since there is no difference in sounds between delaminated concrete and debonded overlays. Hammer sounding is limited to areas visually identified for possible deterioration. Deterioration may include delamination and impending spalls in exposed concrete; rotting timber; and loosened fasteners. Although chain dragging and hammer sounding are inexpensive, it can be physically demanding and time consuming. Likewise, traffic control is often needed for bridge decks. Furthermore, data recording must be done manually using field sketches and photographs.

Although the use of chain dragging and hammer sounding is relatively simple, the extent of the noted delamination is subject to the inspector's interpretation of the tonal differences in the produced sound. Likewise, areas with high levels of background noise, such as large traffic volume or adjacent airports, industry, or construction sites will make the tonal differences of the produced sounds difficult to distinguish. While the horizontal extents of a defect may be detected using chain-dragging or hammer sounding, the depth of the noted defect cannot be determined from these methods.

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5.4 LIQUID PENETRANT

5.4.1 Introduction

Liquid Penetrant Testing is used to confirm the presence of a crack or flaw. It relies on the ability of a liquid to enter into a discontinuity. Therefore it can only find discontinuities, which are open to the surface of the material. It can be applied to any material provided it is non-porous, and is not adversely affected by the penetrant material.

The basic procedure requires that the material be pre-cleaned to remove all surface contaminants and the application of a liquid (penetrating oil) to the surface being tested. The penetrant will seek out and enter small surface openings. Penetrant is then removed from the test surface by wiping or water rinsing. A drying developer is next applied. The penetrant remaining in the discontinuity bleeds out forming a highly visible, contrasting indication on the test surface.

The characteristics of a good penetrant relate to the ability of the fluid to be drawn into small openings even against gravity. This penetrating ability is affected by many variables including surface tension of the liquid, wetting ability, surface condition, surface contamination, and temperature.

There are two major types of penetrants used: (1) visible dye penetrants, and (2) fluorescent-visible penetrants. Visible dye penetrants are normally red, providing contrast with the applied white developer. Fluorescent penetrants contain dyes, which fluoresce brilliantly when viewed under black light in a darkened area. The ability to see penetrant indications on the test surface relates to the contrast provided between the penetrant and the test surface. Fluorescent penetrants provide better contrast than visible dye penetrants. For this reason, fluorescent penetrants are more accurate than visible dye penetrants. Refer to Figure 5.4.1-1 for a view of a casting with the dye and developer applied to the surface. Refer to Figure 5.4.1-2 for a view of a gusset plate with applied dye penetrant.

Interpretation of the liquid penetrant indication involves determining what condition is present to have caused the indication, evaluating the condition as to its effect and seriousness from the standpoint of usability of the part, and reporting of inspection results accurately and clearly.

Proper interpretation and evaluation of liquid penetrant indications requires knowledge of the types, causes, and appearance of indications, knowledge of the test method and material fabrication process, adequate illumination, good eyesight, and experience.

This testing method is covered in American Society for Testing and Materials (ASTM) E165-95 “Standard Test Method for Liquid Penetrant Examination” and ASTM E1417-99 “Standard Practice for Liquid Penetrant Examination.”

Figure 5.4.1-1: Liquid Penetrant and Developer Applied to a Casting.

Figure 5.4.1-2: Gusset Plate with Applied Dye Penetrant.

5.4.2 Applications

Liquid penetrant tests can be conducted on a wide variety of non-porous materials including metallic and non-metallic, magnetic and non-magnetic, as well as conductive and non-conductive. This method is highly sensitive to small surface discontinuities and produces indications directly on the surface of the component providing a visual representation of the flaw.

The penetrant materials typically come in aerosol form making them very portable and well adapted to field use. This also allows large areas of a component to be tested rapidly even if

the component has a complex geometric shape. Powder penetrant materials are also available, but are typically cumbersome in the field.

Finally, penetrant materials and the associated equipment are relatively inexpensive, especially when compared to most other nondestructive evaluation (NDE) methods.

5.4.3 Limitations

Liquid penetrant testing does have several limitations. This method only works on non-porous materials. Also, surface finish and roughness can affect the sensitivity of the test.

It can only detect discontinuities which are open to the surface. Discontinuities filled with contaminants, paint, rust, oxidation, or corrosion products may not be detected. Therefore, pre-cleaning is critical. The process also requires multiple steps of cleaning, applying the dye/fluorescent, cleaning off the dye/fluorescent, applying the developer, and cleaning off the developer after the test is completed. This effort requires the safe handling of chemicals and the proper disposal of saturated cleaning rags and empty aerosol cans.

Finally, the test sensitivity is lowered at reduced temperatures since crack widths are typically reduced and the test medium is less fluid.

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5.5 ULTRASONIC TESTING

5.5.1 Introduction

Ultrasonic Testing (UT) is used to evaluate the internal (volumetric) condition of materials. Specifically, it is used to confirm suspected discontinuities or cracks, as well as check questionable material thicknesses or lengths. Typical discontinuities, which are detectable by use of UT, include laminations, surface cracks, and many surface and subsurface weld related discontinuities (lack of fusion, porosity, etc).

The use of sound to determine the internal properties of a member is not new; audible sound has been used as a nondestructive method for centuries. For instance, striking a porcelain bowl to listen for either a ring or dull tone is an old way to detect a crack. Today, shear stud connectors used for composite bridge beams are still crudely tested by striking them with a hammer and listening to the change in ringing note.

With Ultrasonic Testing, the transducer can be thought of as replacing both the hammer and ear. The transducer directs a wave of high frequency vibrations, inaudible to the human ear, into the test specimen and then receives the returning echoes. The ultrasonic instrument provides the necessary electronics to produce these waves and display the returning echoes for interpretation.

A transducer is a device that can convert energy from one form to another. In the case of UT, electrical energy is changed to mechanical energy and vice versa. Ultrasonic testing transducers convert electrical energy into mechanical vibrations, which in turn produce high frequency sound waves. They also convert high frequency sound back into electrical energy upon receiving the return echoes.

The most common ultrasonic technique currently in use in the United States is called pulse echo. The pulse echo method employs short bursts, or pulses, of waves, which are transmitted into the specimen by the transducer which must be in integral contact with the specimen. This is typically accomplished through the use of a substance called coupling gel. Any returning unexpected echo from these pulses is evaluated for the determination of reflector location and size.

With pulse echo, a single transducer can be utilized as both the emitter and receiver. This type is commonly used in steel inspection applications. Another method utilizes two transducers. One acts as the emitter and the second, located very close to the emitter, acts solely as the receiver. This method is called “pitch-catch” and is more commonly used in the evaluation of other materials such as concrete.

The signal height or amplitude is related to the amount of reflected sound energy. Large reflectors, causing total reflection of sound, produce signal responses of higher amplitude than smaller reflectors, which only reflect a portion of sound energy. Larger return echo amplitudes suggest larger sized flaws. Echo indications are normally retested from another position to confirm flaw size and position. Refer to Figure 5.5.1-1 for an illustration of pulse echo ultrasonic testing.

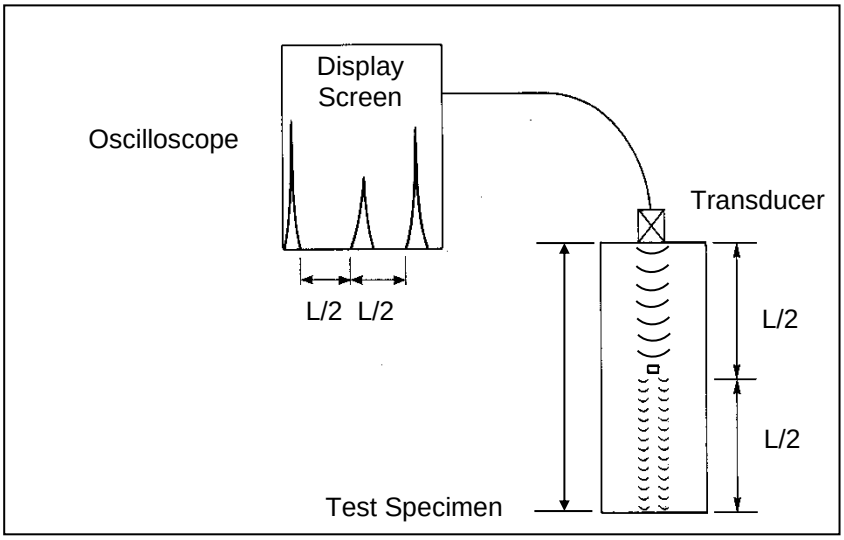

Figure 5.5.1-1: Illustration of Pulse Echo Ultrasonic Testing.

Basic ultrasonic pulse echo systems include the following functions:

1. Power Supply
2. Pulser
3. Receiver/Amplifier
4. Oscilloscope (Cathode Ray Tube) (CRT)
5. Timer (Clock)
6. Transducer

Power for the testing equipment is supplied by portable battery packs or by an external AC source. The pulser, also called the pulse generator, produces the short duration burst of voltage, which is applied to the transducer. The rate of these voltage bursts is controlled by a clock or timer. Sound echoes returning to the transducer are relayed to the receiver, amplified, filtered and sent to the cathode ray tube for display on the screen. Pulse echo methods include compression, shear, and surface wave modes.

1. Compression Wave Testing, (also called straight beam testing), is used for flaw detection, particularly laminations, and for thickness measuring. It directs waves into the material perpendicular to the specimen's surface. Refer to Figure 5.5.1-2 for a schematic of the Compression Wave mode.

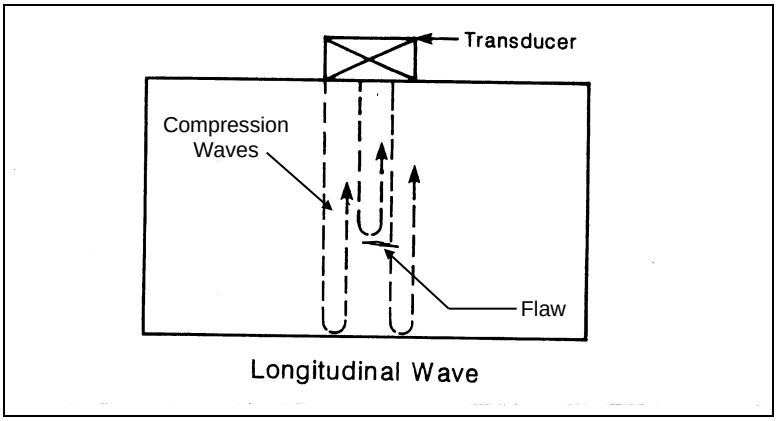

Figure 5.5.1-2: Pulse Echo UT Compression Wave Schematic.

2. Shear Wave Testing, (also called angle beam testing), is ideally suited for weld testing. Waves are directed into the material at an angle other than 90 degrees to the specimen surface. The shorter wavelength (lower velocity) increases sensitivity, and angular capability allows for weld examination at predetermined angles. Refer to Figure 5.5.1-3 for a schematic of the Shear Wave mode.

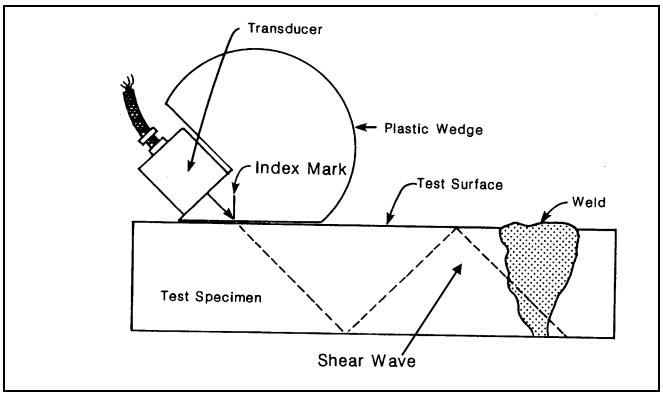

Figure 5.5.1-3: Pulse Echo UT Shear Wave Schematic.

3. Phased Array Testing is an advanced ultrasonic method that uses several elements within a single transducer to “steer” the stress beam. Phased Array combines both straight and angled beam testing by emitting a stress beam at several angles from one location. The stress beam covers a much larger area and allows the inspector to view an internal defect from more perspectives using different displays. Phased array provides better volumetric coverage and is especially advantageous for weld inspections. Phased Array is typically used in steel inspection however there is technology used in concrete inspection that performs similarly.

A variety American Society for Testing and Materials (ASTM) Standards cover ultrasonic testing methods dependent on the material and type of structural component. American Welding Society (AWS) Standards cover ultrasonic testing of welds.

5.5.2 Applications

Perhaps the most advantageous aspect of Ultrasonic Testing (UT) is its ability to examine the internal structure of a material when accessibility is limited to one side. It is an ideal method for the detection of flaws, which are generally not readily detectable by visual means. Because of the basic characteristics of UT, it is used to inspect a variety of both metallic and nonmetallic members such as welds, forgings, castings, plastics, ceramics, concrete, steel sheeting, aluminum tubing, fiberglass, timber, etc. Since UT is capable of economically revealing subsurface discontinuities (variation in material composition) in a variety of dissimilar materials, it is an extremely effective and useful tool. Furthermore, relatively thick specimens can also be examined. Used on bridge decks, slabs, and foundations, UT results are definitive for both bare and covered concrete decks. Penetration of asphalt thicknesses of up to 6 inches has been quite successful. Used on caissons and piles, the length and integrity of piles and caissons can be determined. Ultrasonic tests on concrete, wood, and steel piles have been generally successful to lengths of 100 inches.

UT is most successful for detecting discontinuities, which are oriented perpendicular to the direction of the propagating stress beam. It is also often used as a complimentary method to other nondestructive evaluation (NDE) procedures such as radiography.

The method is readily adaptable to field testing, as portable lightweight units containing a rechargeable battery having a typical 8-hour battery life. Refer to Figure 5.5.2-1 for a view of an inspector conducting UT in the field.

Figure 5.5.2-1: Inspector Conducting UT on a Sign Structure Anchor Bolt.

5.5.3 Limitations

Ultrasonic testing (UT) should not be performed on rough surfaces, on parts with complicated geometries, on highly attenuated materials, or where the discontinuity size is expected to be smaller than one half of the wavelength. Rough surfaces may require grinding in the surface preparation. Other factors, which limit the successful application of UT are: lack of properly trained personnel, over estimation of the accuracy of flaw locating and sizing, and poorly

written testing procedures. Typically, a certified Level III nondestructive evaluation (NDE) specialist should evaluate and develop written testing procedures for uncommon applications.

Steel anisotropy and difference between the acoustic velocity of the test object and the calibration standard can lead to discontinuity mis location, or totally reflected ultrasound in the inspected part. This phenomenon is described in FHWA memorandum *"Recommendations to Account for Acoustic Dissimilarities in Steel Ultrasonic Weld Inspection"* from December 2024. This phenomenon is prevalent in steels produced by the thermo-mechanical control process (TMCP) where the differences in shear wave velocity in the rolled direction vs. the transverse-to-rolled direction are more pronounced.

Special procedures and possibly additional calibration standards may be needed prior to the test being conducted on steel with notable anisotropy or a notable velocity difference between the calibration standard and the test specimen.

Furthermore, it is important that the UT equipment be calibrated prior to each use, and that certified personnel are operating the equipment, or are under the direct supervision of a certified individual.

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5.6 ACOUSTIC EMISSION

5.6.1 Introduction

Noises occurring in nature are accompanied by, and are probably the result of, an energy release of some kind. Fatigue cracks, weld discontinuities, and many other failure causing mechanisms also produce sound energy. Although a portion of the sound produced by materials under stress may exist as audible sound, most is low energy and inaudible. This depends on deformation magnitude and type, and upon flaw growth or failure. An example of the audible stress release occurred at the Mianus River Bridge in Greenwich, CT, where various newspapers reported loud noises being heard by residents days before the actual collapse occurred. This was also the case at the Hoan Bridge in Milwaukee, where witnesses reported a loud noise at the time of the brittle fracture of the steel girders.

For the purposes of this discussion, an Acoustic Emission (AE) is defined as inaudible sound energy released within a material undergoing deformation or flaw growth. An AE test is described as a method used to detect this sound energy. This is a "passive" method. The inspector or a system of transducers simply listens for any sounds emanating from the source location.

To detect acoustic emissions, one or more "listening" transducers are attached to the test object. Positioning of AE transducers in the path of anticipated sound propagation enables detection. The detected signals are then electronically processed to derive information or the location and severity of growing flaws. It should be noted that "guard" transducers are also used in conjunction with the "listening" transducers to differentiate the flaws from just normal bridge noises. Refer to Figure 5.6.1-1 for a schematic of a basic acoustic emission test.

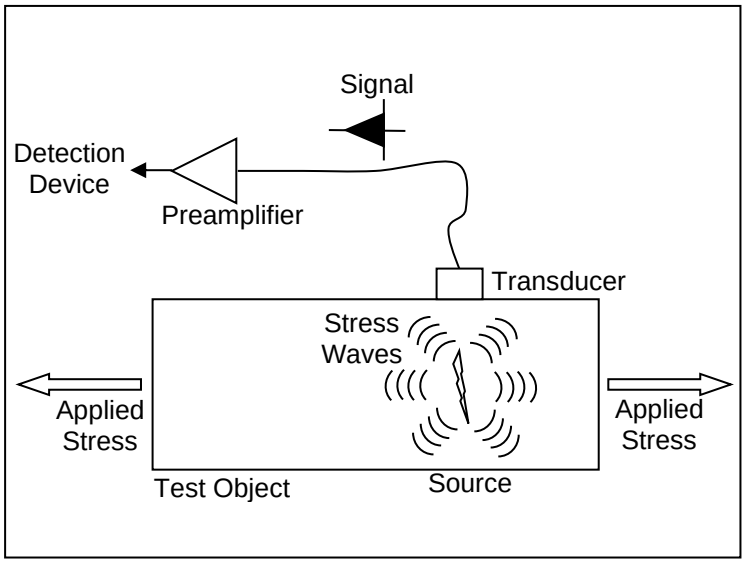

Figure 5.6.1-1: Acoustic Emission Test Configuration (Guard Transducers are Not Shown for Clarity).

AE testing differs significantly from the other nondestructive evaluation (NDE) methods discussed in Part 5. Perhaps the most notable differences are:

1. The detected signal is produced by the test material itself, not by an external source. The AE transducers need only act as receivers;

2. AE tests detect movement, where most other methods typically only detect existing geometrical discontinuities; and

3. An applied stress is required to cause flaw growth, and hence, the acoustic emission. The applied stress can be the result of the components service and dead loads or an induced load used specifically for the AE test. In many tests, a combination of the two is necessary.

Various American Society for Testing and Materials (ASTM) Standards cover AE testing and are dependent on the material and type of structural component being tested.

5.6.2 Applications

AE testing is used to detect cracks, corrosion, weld defects, and material embrittlement. This method can be used on a wide variety of materials, such as metal, timber, concrete, fiberglass, composites, and ceramic. However, it is most extensively used in steel structures, particularly on retrofitted connections and previously noted cracked members.

An entire structure can be monitored with AE testing from a few locations, reducing the amount of access required. AE testing can also be conducted while the structure is in-service. Furthermore the structure can be inspected remotely. That is, the inspector can obtain the data from the web rather than at the site.

AE testing is a real-time NDE method. In other words, it is monitoring the actual condition of the component during the test. The AE test method can also be used to record an accumulation of damage occurring within a structure. The data obtained can be used as history for a structure, and possibly to predict failure.

5.6.3 Limitations

A primary limitation of AE testing of structures, such as bridges, is the requirement to differentiate the sound energy released by a growing flaw from that which is called background noise. Many background noise generators such as bolts, joint friction, traffic, and others can mimic or mask the sound energy released from growing cracks. Some AE test methods avoid this problem by isolating areas known to contain possible background noise generators.

When a global AE inspection is conducted to determine areas where structural problems exist, it will not provide the inspector with the type or size of the defect. Additional NDE or partially destructive evaluation (PDE) methods may be required to identify the exact nature of the emission source defect.

Due to the necessity of listening to the flaw grow, AE systems are typically set up and run 24/7, which can be very costly depending on the size of the structure, the number of transducers on the structure, and the amount of data the inspector must analyze.

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MAGNETIC PARTICLE

5.7.1 Introduction

The magnetic particle test (MT) is used for testing ferromagnetic materials (steel, wrought iron, cast iron, etc.). MT is used to confirm suspected cracks or test suspect-details. The primary advantage of MT is the high sensitivity in the detection of tight surface cracks and other small discontinuities. Typical detectable discontinuities include cracks, lack of fusion, and other weld related surface discontinuities. Base metal discontinuities such as seams, laps, and "stringers" are also easily detected.

As defined by American National Standards Institute (ANSI), the magnetic particle method utilizes the principle that magnetic lines of force, when present in a ferromagnetic material, will be distorted by a change in material continuity, such as a sharp dimensional change or a discontinuity. If the discontinuity is open to, or close to, the surface of a magnetized material, flux lines will be distorted at the surface; a condition termed flux leakage. When fine magnetic particles are distributed over the area of the discontinuity where the flux leakage exists, the particles will converge at the discontinuity and the accumulation of particles will be visible for the inspector.

The objective of magnetic particle testing is to cause a magnetic field of sufficient strength and predetermined direction to leak if discontinuities are present. The inspector detects these leaks by sprinkling the test area with particles, blowing away the excess, and then looking for areas where the particles have accumulated. These areas of accumulation indicate a surface or possibly a subsurface discontinuity.

MT methods and implementation procedures are fully described by the following terms:

1. **Dry method:** The dry method describes the type of indicating medium, the particles, as dry. Commercial powders are available in various colors including red, black, grey, or yellow. The color selection should be based on the maximum color contrast with the material to be tested. Refer to Figure 5.7.1-1 for a view of the test powder colors. Dry fluorescent particles are also available for use with a black light. Dry particles are finely divided ferromagnetic material with high permeability and low retentivity. The powder consists of a mixture of particle sizes, smaller ones being attracted by weak leakage fields, and larger ones for detecting larger discontinuities.

Figure 5.7.1-1: Sample Powder Colors.

2. **Wet Method:** If powders or particles are suspended in oil or water, the method is considered wet. Wet suspensions are also available in various colors and fluorescent. They can be sprayed onto the part, or the part can be bathed in a suspension. Wet fluorescent particles provide maximum sensitivity (superior visibility) if used with the proper current, lighting, and surface preparation. Wet particles are mixed with the suspension in predetermined concentrations and particle sizes. The concentration will affect the test sensitivity. Light concentrations will produce faint indications and heavy concentrations may provide too much coverage. They are generally smaller in size and lower in permeability than dry particles.

3. **Continuous Procedure:** This term is used when the magnetizing force is applied prior to the application of the particles and terminated only after excess particles have been lightly blown away.

4. **Residual Procedure:** This term residual is used where the particles are applied after the part has been magnetized, and the magnetizing current terminated.

Field-testing of bridges and related structures use portable units, which include the small portable prod or yoke units with alternating current (AC), half-wave direct current (HWDC) capability. Portable prod equipment is commonly available in amperages up to 1,500. However, it can also be powered from 115-volt single-phase AC. Prods directly induce the electrical current into the part. **The inspector should note that direct current (DC) prods may cause arc strikes and therefore, they should never be used on nonredundant steel tension members (NSTMs).** Refer to Figure 5.7.1.2 for a view of prod equipment. Prods are used with a separate power unit that generates the current for testing. Both prods can be attached to a single fixture, similar to a yoke.

Figure 5.7.1.2: Prods used for MT

Yokes are lightweight portable units easily carried to the job site. On some yokes the legs are fixed at a set distance. On others the legs are adjustable for various pole spacing. Yokes operate with 115 volt AC. The potential for electrical arc strikes is normally not an issue with a yoke, even when used with half-wave direct current (HWDC) current. Refer to Figure 5.7.1-3 for a view of a portable field kit.

This testing method is covered in American Society for Testing and Materials (ASTM) Test E709, “Standard Guide for Magnetic Particle Examination.”

Figure 5.7.1-3: Magnetic Particle Inspection Kit.

5.7.2 Applications

MT is a sensitive means of locating small and shallow surface cracks and can locate near surface discontinuities with direct current. Unlike with liquid penetrant, cracks filled with foreign material can be detected and no elaborate pre-cleaning is necessary. This test is also effective on painted surfaces.

This method is reasonably fast, inexpensive especially compared to some other nondestructive evaluation (NDE) methods, and the equipment is very portable. There is also little or no limitation due to size or shape of the part being inspected. Refer to Figures 5.7.2-1 and 5.7.2-2 for views of magnetic particle tests being performed in the field. Refer to Figure 5.7.2-3 for a view of a crack that was identified using magnetic particle testing.

Figure 5.7.2-1: Yoke MT on a Built-Up Plate Bridge Girder Butt-Weld.

Figure 5.7.2-2: Prod MT on a Sign Structure Base.

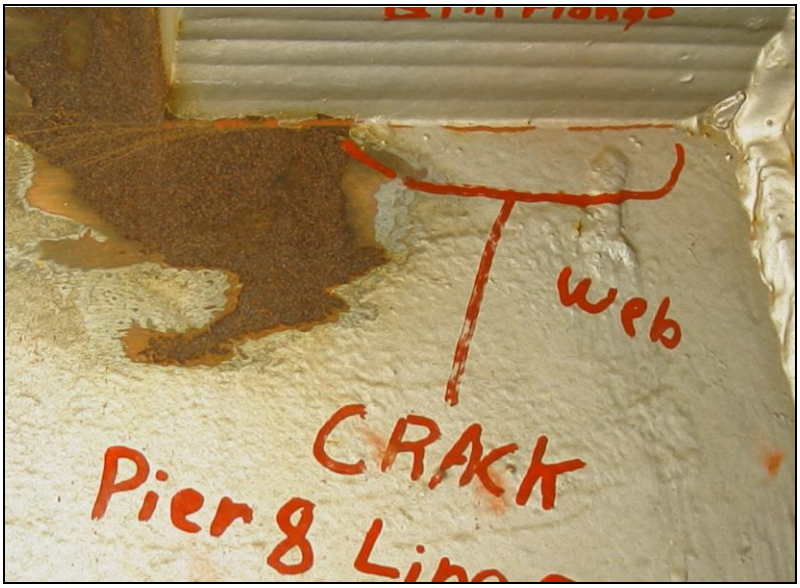

Figure 5.7.2-3: Crack Identified by MT.

5.7.3 Limitations

MT does have some limitations. This test will work only on ferromagnetic material and the magnetic field must be in a direction perpendicular to the principal plane of the discontinuity for best detection. MT will not disclose fine porosity. The deeper the discontinuity lies below the test surface, the larger the discontinuity must be to provide a readable indication.

As previously mentioned, DC prods (not the same as the Half Wave Direct Current (HWDC) yoke) should not be used on NSTM's. Test surfaces should be clean and paint removed for highest sensitivity. Also, residual magnetism may need to be removed. Although the method appears simple, the need for experienced and knowledgeable operators is required.

Permanent magnets can be used in certain applications where the access to 115V AC power is not available or not available for practical access. This method has many limitations. Permanent magnets are only properly used when these limitations preventing the formation of adequate leakage fields at the discontinuities is addressed and accounted for in the test procedure.

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5.8 EDDY CURRENT

5.8.1 Introduction

The Eddy Current (EC) test is used for testing ferromagnetic and conductive materials. EC is used to confirm and detect cracking or near-surface defects. EC is highly effective at detecting cracking and is commonly employed to detect and map out fatigue cracking on structural members. EC is also used to determine the thicknesses of coatings, such as galvanizing or thin materials.

The Eddy Current test works off the theory of mutual inductance. That is a moving electric field will induce a magnetic field within a nearby conductive material and a moving magnetic field will induce an electric field in a nearby conductive material. In Eddy Current testing a probe is pressed to the surface of the test material. The probe is typically a coiled wire with an electric current running through it. The electric current has an associated magnetic field wrapped around it. At the tip of the probe, this magnetic field induces a circular electric current on the surface of the test material. When the current comes into contact with a discontinuity (crack), the eddy current is disrupted and a characteristic signal is displayed. The circulating current (named eddy currents after similar observed fluid dynamic behavior in streams) has an associated magnetic field which opposes the magnetic field of the probe. This is what is referred to as mutual inductance. Any change in the eddy currents in the test material will affect the resulting impedance of the coil.

Because the magnetic field surround the probe, direct contact with the test material is not essential. However, the magnetic fields attenuate rapidly in the air so the probe must be near the test specimen. Because the “lift-off” distance does not need to be zero and dielectric or non-conductive materials have little effect on the magnetic fields, Eddy Current testing can be performed through coatings.

Figure 5.8.1-1: Image of Eddy Current testing device with pencil probe in the foreground.

Field-testing of bridges and related structures use portable units. Commercial units offer a variety of display options and are available in analog or digital. Needle devices (analog) are "go, no-go" devices, meaning the device simply shows if a discontinuity is present. Refer to figure 5.7.1-2 for an image of a needle eddy current unit. The more common display used in bridge inspection is the digital display which typically set up as time vs. signal amplitude. This allows the inspector to more readily pin point the ends of cracks, for example. Refer to Figure 5.8.1-3 for an image of a digital eddy current device.

This testing method is covered in American Society for Testing and Materials (ASTM) Test E709, "Standard Guide for Magnetic Particle Examination."

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Figure 5.8.1-2: Magnetic Particle Inspection Kit

5.8.2 Applications

Eddy Current devices are available with testing frequencies ranging from 10Hz to 10Mhz depending on the material, probe type, probe geometry and particular testing need.

The test is suitable for any conductive material and is a highly sensitive means of locating cracking on or near the surface of a member. It is used often on testing fatigue prone details at welds and bolt holes.

Eddy Current testing is also a method for testing coating thickness, such as galvanizing. The device can detect coating thicknesses up to 0.2-1.0 mils. It can also be used to detect the thicknesses of thin conductive members from <25 microns – 3 mm.

This method is reasonably fast, inexpensive especially compared to some other nondestructive evaluation (NDE) methods, and the equipment is very portable. There is also little or no limitation due to size or shape of the part being inspected.

5.8.3 Limitations

Eddy Current testing does have some limitations. This test will work only on conductive material and the discontinuity or defect must be perpendicular to the plane of the induced eddy currents. Subsurface defects may be detected by eddy current testing but the depth of the defect is extremely limited. Furthermore, EC will not afford the inspector with the depth of a crack if detected.

While Eddy Current testing is used to determine the thicknesses of galvanizing coating, the test will not detect defects below the coating due to its conductive characteristics. Any changes to the test materials magnetic characteristics will greatly affect the results of EC.

While EC is an effective and fast test, it does require training and inspector interpretation due to all the factors that can affect test results. Changes in a materials geometry or properties can create complex signals or background noise that an experienced inspector must be able to filter out.

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5.9 RADIOGRAPHY

5.9.1 Introduction

Radiography (RT) is used for evaluating the internal structure of a material. Similarly to an X-Ray, the method uses high energy high frequency electromagnetic particles to pass through the material and capture the particles on the opposite side on a film. The film is developed and offers an actual picture of the internal characteristics of the member being tested.

Radiography can use two types of sources for the electromagnetic radiation, man-made or naturally occurring. The man-made sources are high energy X-rays that are created through a device called an X-ray tube. Naturally occurring sources emit gamma rays. Two common sources include Iridium 192 and Cobalt 60. These sources require no outside power.

The method relies on the theory of electromagnetic radiation in that high energy photons (particles of electromagnetic radiation) will be able to pass through material but will be attenuated (slowed down or scattered) by changes in the material's properties. These properties include density, geometry, defects, holes or voids, etc. Higher density material will attenuate the photons more and therefore result in less photons passing through to the film. Generally on the film, denser materials or areas show up as lighter spots on the film, while areas of less density or less material will allow more photons to pass through and show up as darker areas on the film. Refer to Figure 5.9.1-1 for conceptual rendering of a radiographic test of a weld.

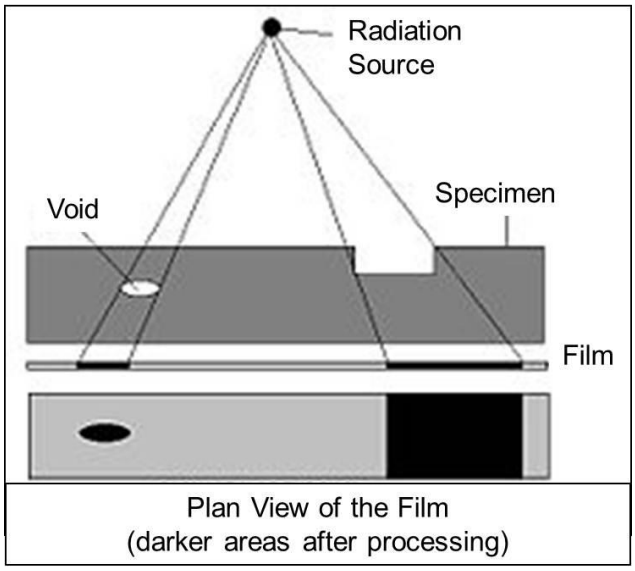

Figure 5.9.1- 1: RT of material with a void and notch. Note the darker areas on the film located below the test material.

RT can be performed on most materials that require internal evaluation, however the nondestructive evaluation test is typically performed on steel members and in particular welds, and on concrete members to determine reinforcing layout. Refer to Figure 5.9.1-2 for

an RT image of a weld with internal defects. Refer to Figure 5.9.1-3 for an RT image of a concrete slab with reinforcing steel shown.

Figure 5.9.1- 2: RT image of weld. Note slag inclusions located in the center of the image and a lack of fusion located at right center.

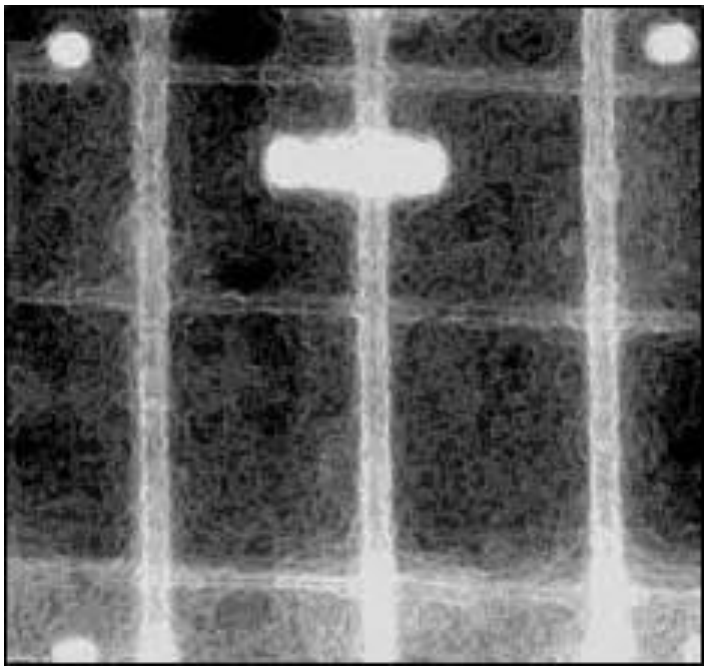

Figure 5.9.1- 3: RT image of concrete slab. Note reinforcing layout shown on film.

5.9.2 Applications

Radiography is used to determine the internal makeup of a member or portion of a member. Gamma ray radiography typically consists of a much smaller device since the source requires no power. The device itself is to protect the inspector from the high levels of radiation and to focus the radiation beam at the member. This type of RT is useful for field work as it is portable and easy to use.

X-Ray tubes can produce much higher energy photons and are typically used when high penetration through a material is required, that is highly dense material or very large members.

RT is most commonly used on concrete and steel members. It is highly effective at determining internal flaws within weldments. RT can detect slag inclusions, porosity, lack of fusion, and cracking. In order for RT to detect cracking, the length or depth of the crack must be oriented in line with the radiation beam from the source, otherwise the photons will pass across the crack rather than through the crack. To some degree RT can also be used to detect section loss as the less material will show up slightly darker than the surrounding material.

In concrete applications, RT is commonly used to determine reinforcing layouts. RT can also be used on concrete members to determine voids.

5.9.3 Limitations

Radiographic testing does have some limitations. The testing itself is rather expensive especially when dealing with thicker members. Furthermore, there are only a few licensed firms that can handle radioactive isotopes.

While the test will provide the inspector with an internal view of the member being inspected, it is only a 2D representation of a 3D object and therefore will not provide the depth dimension of where the defect is located. Lead tape and indicators are used as reference points on the film to try and remediate this issue.

Radiography testing also requires access to both sides of a member. The source must be on one side and the film on the other. In certain situations, this may be an impossible test due to inaccessibility.

Radiography is also a complex method that includes the safety hazard of working with a radioactive material which requires its own personal protective equipment. The equipment itself is also highly specialized and requires an experienced inspector to operate effectively.

Chapter 10 – GROUND PENETRATING RADAR

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5.10 GROUND PENETRATING RADAR

5.10.1 Introduction

Ground Penetrating Radar (GPR) is a recognized non-destructive evaluation technique with many applications. Applications on bridges include bridge deck condition evaluation, overlay thickness, voiding under bridge approach slabs, reinforcing steel location, foundation investigation, and underwater profiling. Chapter 5 will discuss the use of GPR on bridge decks and approaches, while Chapters 24 and 25 will discuss additional uses of GPR.

A radar system typically consists of a control unit, radar antenna, and display unit. The control unit generates a radar pulse (in the microwave spectrum) and sends it through a cable to the antenna. The antenna transmits the pulse into the surface. When this energy encounters an interface between two materials of differing dielectric properties, such as reinforcing steel, air, moisture, or the base-course material, a portion of the energy is reflected back to the radar antenna. The received pulse is sent back to the control unit for processing/storage. The display unit (video or chart recorder) presents the data.

The reflected energy is received by the transducer, amplified, and recorded. The electromagnetic pulse is repeated at a rapid rate and the resultant stream of radar data produces a continuous record of the subsurface. The radar system creates a linear profile of the materials beneath the antenna pass. Refer to Figure 5.10.1-1 for an example of radar output on a typical pavement section.

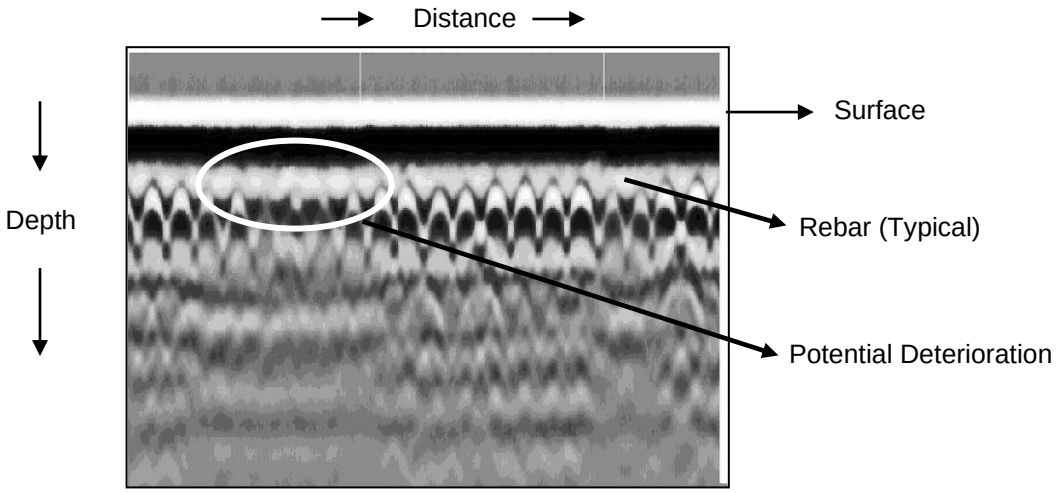

Figure 5.10.1-1: Typical Radar Output Image with Notations.

Two different types of transducers (contact or horn type) can be mounted on a data collection vehicle, or hand towed. Refer to Figure 5.10.1-2 for views of two types of vehicle-mounted transducers.

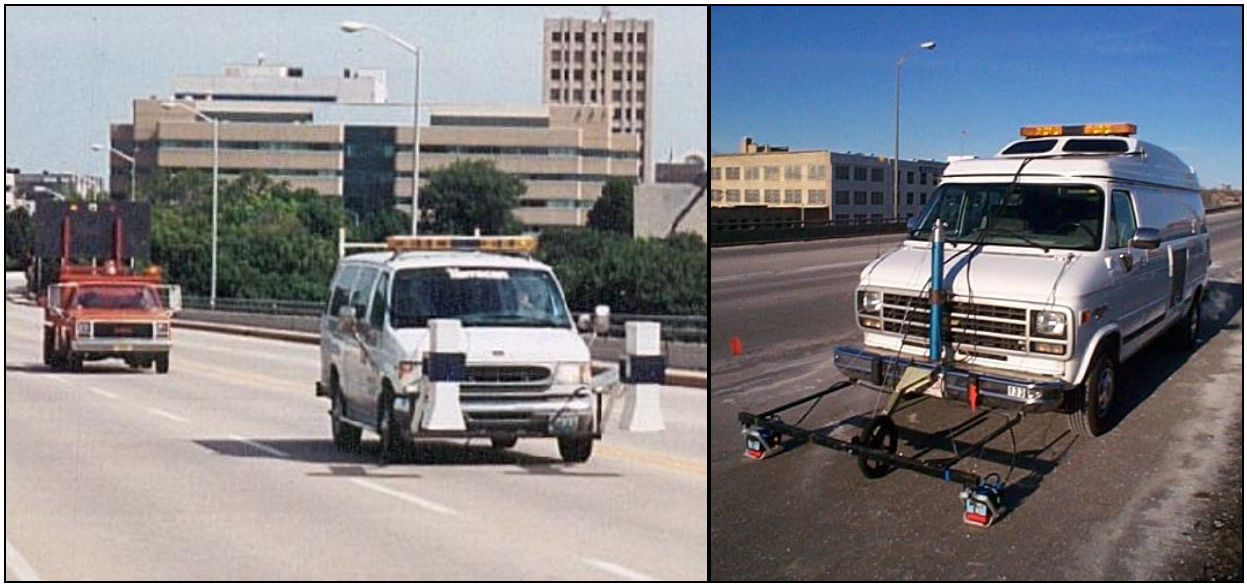

Figure 5.10.1-2: Horn Style (left) and Contact Style (right) Vehicle-Mounted Transducers.

The location of the transducers can be varied across the width of the pavement and, if additional information is required, a number of passes with the antenna in different locations can be made.

For the majority of surveys, the antennae are mounted over the wheel tracks. The data is normally collected with vehicle speed slower than normal traffic speeds. Faster speeds are attainable, but the longitudinal and vertical resolution of the system is reduced. Horizontal data positioning is accomplished by using a distance transducer connected to the drive train of the data collection vehicle.

An event mark is automatically placed on the data at user-defined intervals, allowing defects to be located accurately. Once the survey is completed, a computer processes the data and the results of the survey can then be presented in a variety of formats.

Another advantage of GPR is the fact that only one side of the element needs to be tested. Indications on the opposite side can be picked up.

GPR is covered in American Society for Testing and Materials (ASTM) D6087-97 (2001), "Standard Test Method for Evaluating Asphalt-Covered Concrete Bridge Decks Using Ground Penetrating Radar."

5.10.2 Applications

Ground Penetrating Radar (GPR) is most commonly used on concrete bridge decks with an overlay surface. This allows for an inspection of the concrete deck surface, which is hidden by the overlay surface. If the concrete deck is not covered, GPR is not often used since it is not as accurate or rapid as infrared thermography.

The GPR system provides a means of determining the following items:

1. Pavement and/or overlay thickness.

2. Locating and/or determining the depth of reinforcing steel, mesh, or pre and post tensioning strands.

3. If sufficient rigid pavement cover exists above reinforcing steel to allow pavement-grinding rehabilitation.

4. Identifying pavement or joint types.

5. Locating and determining the size of voids beneath pavements.

5.10.3 Limitations

Ground penetrating radar (GPR) identifies areas of a concrete deck with different dielectric properties or conductivities. Some concretes, such as dry low permeability concrete, affect the accuracy of GPR to detect areas of delamination. GPR is also sensitive to the presence of water and chlorides on the deck and between overlays and the base concrete, as well as the presence of debris on the deck surface. These conditions can significantly influence the accuracy of the data. Heavy or congested reinforcing layouts will also affect the detection of detects below the reinforcing mat.

GPR must also be scanned perpendicular to the top layer of reinforcing steel. Therefore, inspection of some structures will require the survey to be conducted perpendicular to the flow of traffic. This will require traffic to be restricted or stopped altogether while the survey is being conducted. Frequently, several passes must be made on the deck area and the cost may be prohibitive.

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5.11 INFRARED THERMOGRAPHY

5.11.1 Introduction

One type of deterioration encountered in bridge decks is delamination within the concrete deck. A delamination is defined as a horizontal fracture plane at or above the top layer of the reinforcing. Overlay surfaces can hide delaminations until they are well advanced and it can be difficult to distinguish between deterioration in the concrete deck slab and debonding of the overlay. Refer to Figure 5.11.1-1 for an overall view of a concrete bridge deck with an asphalt overlay.

Figure 5.11.1-1: Bridge Deck with Asphalt Overlay and Extensive Patching.

As discussed in Chapter 3 of Part 5, traditional nondestructive evaluation (NDE) methods involve chain dragging and hammer sounding. These audible methods require inspector judgment, a great deal of effort, traffic control, and traffic disruption from the lane closures. The presence of an asphalt or concrete overlay reduces the effectiveness of these traditional audible methods.

Infrared Thermography (IR) is an alternative tool for locating and mapping delaminations in bridge decks and pavements. A technique using an infrared scanner and control video camera, infrared thermography senses temperature differences between delaminated and non-delaminated areas.

A delamination in a concrete deck creates a thermal discontinuity that acts as an insulator. IR operates on the principal that when the sun warms the deck, the delaminated area heats up at a faster rate and reaches a higher temperature than the solid areas. This detection also works when the sun is setting and the ambient temperature begins to cool. The delaminated area will cool faster than the surrounding solid concrete. This method is best used on sunny or partially sunny days.

. The deck must be dry and winds must be less than 25 mph. Temperature difference is primarily related to the amount of sun, not the ambient air temperature, so inspections can be undertaken under various temperatures. From a practical standpoint, the majority of inspections are made between March and November with the use of a moving vehicle. Refer to Figure 5.11.1-2 for a schematic of a typical infrared thermography inspection.

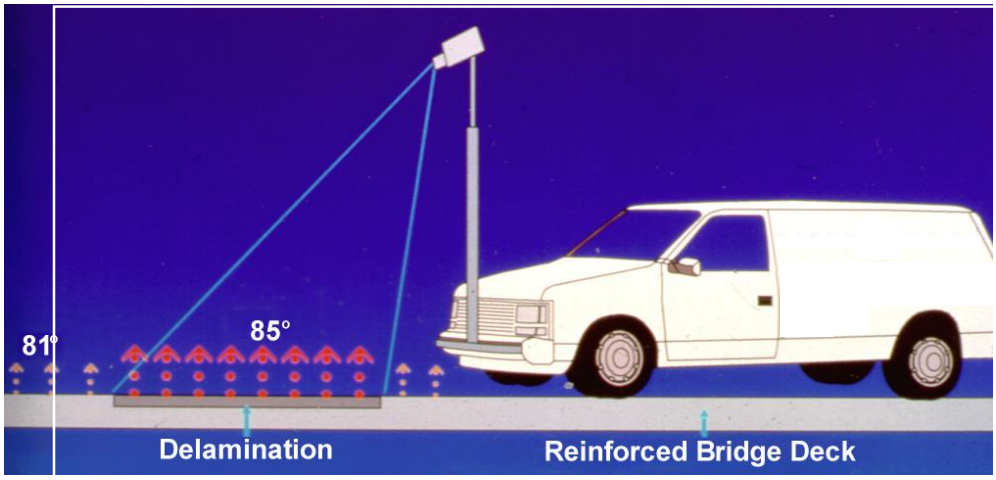

Figure 5.11.1-2: Illustration of IR of a Bridge Deck.

The procedure involves scanning the bridge deck with an infrared camera and recording the video signal on videotape for detailed analysis in the office. A single pass, with a vehicle speed of approximately five mph, is typically made for each lane and shoulder of the bridge deck. At the same time, a real-life control image of the bridge deck surface is recorded. Distance footage is superimposed onto both videotape signals to locate defects. Refer to Figure 5.11.1-3 for a view of the display images from an inspection.

Field confirmation of the infrared data consists of sounding several suspect deteriorated areas and measuring surface temperatures of both suspect and solid areas. Furthermore, select deck cores may be taken for confirmation. These proposed core locations are typically marked at the time of the inspection.

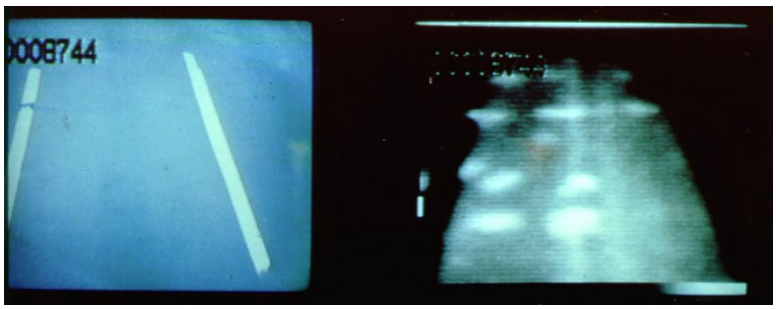

Figure 5.11.1-3: Real Life Image (left), Infrared Image (right).

Analysis of the infrared data is completed with the aid of a computer digitization program. During the analysis, the recorded temperature variations are interpreted to identify specific, delaminated areas. Each delamination is identified and plotted onto plan view drawings of the bridge deck. Square footage and percentage of delaminated deck are calculated. The

real-life control data is examined to make sure that temperature variations were not caused by concrete spalls, discoloration, patches, tar, or debris. In addition, the real life control data is used to plot patches, spalls, etc.

The use of IR for bridge decks is covered in American Society for Testing and Materials (ASTM) D4788-88; "Standard Test Method for Detecting Delaminations in Bridge Decks Using Infrared Thermography."

5.11.2 Applications

IR is most commonly used on concrete bridge decks with or without overlays; however, it can also be used on other concrete components. This is commonly accomplished through hand-held infrared camcorders that allow the inspector to move around the structure at a necessary pace. This method is proven to be accurate and easily repeatable.

IR also provides for quicker data collection, since the equipment can be vehicle mounted and driven over the bridge deck. By mounting the equipment to a vehicle, the process typically results in minimal traffic disruption. Compared to an audible inspection, infrared thermography can be used in areas with high traffic volumes or noise levels. The use of a hand-held infrared can also be used as an aid to quickly map out suspect areas for evaluation.

Data collection for infrared thermography is completed with the aid of computer logging software and the image can be digitally processed for an overall assessment of the bridge deck.

5.11.3 Limitations

Currently, IR requires a temperature differential of approximately 0.9 degrees Fahrenheit (0.5° C) between the delaminated or debonded areas and solid regions of the concrete deck. This typically requires that inspections be done on days with approximately 50 percent sunshine. Moreover, there are particular times of the day this method is applicable. As the temperature of the object cyclically warms and cools, there are periods when the delaminated area and sound concrete are at a similar temperature and therefore will not display on the infrared image. Wet areas, shadows, and debris on the deck do not allow a temperature difference to be established, and therefore, these areas cannot be inspected.

IR locates the delaminated areas in the horizontal plane but does not provide any information on the depth layer where the defect occurs. If confirmation on depth is desired, cores can be taken from the deck.

If an inspector plans to use this technology at only a few structures, or is considering setting up and operating a data collection vehicle, cost may be a prohibitive factor since the scanning equipment and data processing software are expensive. Also, the vehicle must typically be operated by at least two inspectors. However, when compared to manually sounding a large deck or several smaller decks, IR may be more cost-effective and may require less traffic control.

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5.12 IMPACT ECHO

5.12.1 Introduction

The impact echo method is a nondestructive evaluation technique used for detecting internal flaws in concrete. It has been used on a variety of members, particularly slab, beam, and wall type members. The impact echo test method produces a transient stress pulse into a member by means of a point impact. This pulse produces a surface wave as well as waves that travel into the element. These waves are reflected by internal defects and the boundaries of the element.

The testing apparatus consists of a hand held unit that generates an impact (typically a small hammer) which produces a wave and a receiving transducer which receives the reflected waves. The size of the impact device produces specific frequencies of stress waves. A computer-based system is then used to process the data and display the echo wave form data. The operator interprets the data to determine the presence and extent of defects found. Refer to Figures 5.12.1-1 and 5.12.1-2 for views of an Impact Echo Test Unit and an inspector using the unit.

Figure 5.12.1-1: Impact Echo Test Unit.

Figure 5.12.1-2: Inspector Using an Impact Echo Test Unit.

Impact Echo Testing is covered in American Society for Testing and Materials (ASTM) C1383-98a; "Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact-Echo Method."

5.12.2 Applications

The impact echo technique utilizes easily transportable equipment and can be performed by a single individual. Testing is fairly rapid and only minimal surface preparation is needed to assure proper transfer of the impact energy to the structure. Tests are often made on a grid pattern, with the size of the grid determined by the suspected damage. Experience has shown that the technique can be used to locate delamination, honeycomb, cracks, voids, etc. In the hands of a skilled technician, it may also locate voids around reinforcing steel and within grouted prestressing strands and post-tensioned tendons. Impact echo is also used solely as a means to determine member thickness.

Impact echo devices can also be vehicle or wheel mounted. These devices are comprised of a wheel with numerous transducers and solenoids (impactor) around the circumference. Typically two wheels are used in this set up. The solenoid impacts the surface as the transducer comes into contact with the surface. This is done in rapid succession. This can be a very efficient method for detecting deck and slab internal defects.

5.12.3 Limitations

The impact echo method requires interpretation of the wave form output for each test by the field technician. The testing technician must be trained and experienced in order to properly interpret the output data. Prior to testing, design plans should be carefully reviewed for embedded items or other details that may affect wave behavior and test results. The presence of reinforcing must also be properly accounted for. Varying member geometries can also affect data results due to wave reflection. The maximum element thickness for this test is approximately 6.5 feet.

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5.13 ULTRASONIC PULSE VELOCITY

5.13.1 Introduction

Ultrasonic Pulse Velocity testing (UPV) is used to evaluate the internal (volumetric) condition of materials. It differs from Ultrasonic Testing (UT) in that the stress or sound wave is not directly used to determine the internal boundaries of defects. UPV uses the velocity of the sound wave to determine the internal characteristics of the member. Specifically, it is used in concrete and timber inspection to determine voids, density changes and concrete strength properties.

Contrary to UT, UPV does not use the echo of the sound wave off internal boundaries to detect deficiencies. Rather it determines the velocity of the pulse. With the distance between the transducers known and the pulse speed determined from the material's mechanical properties, the velocity is calculated. By comparing the velocity from one location to another, the internal composition of the material can be determined. This test can be performed throughout a member during one inspection or over a period of time to detect any changes.

Refer to Chapter 5 of Part 3 for an in-depth description of how sound waves and transducers work.

UPV makes use of two transducers, similar to the "pitch-catch" method of UT. One transducer is the emitter and the other, the receiver. There are two set ups that can be utilized, direct transmission mode and semi-direct transmission mode. This allows for a greater range of inspection. Refer to Figure 5.13.1-1 for an image of direct vs. semi-direct transmission. In either case the distance between the transducers must be known for comparative results.

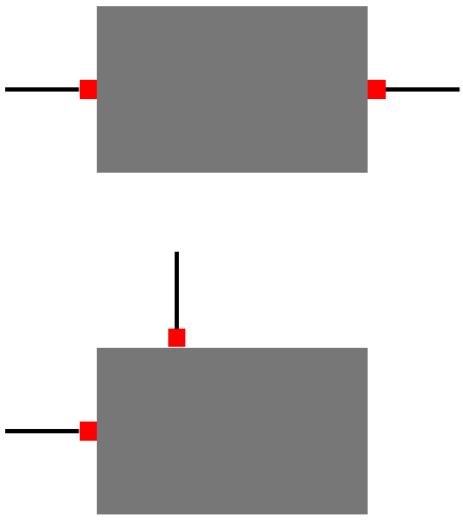

Figure 5.13.1-1: Illustration of Direct Transmission (Top) and Semi-Direct Transmission (Bottom).

The velocity of the pulse is dependent on the material's mechanical properties, specifically the elastic modulus. Speeds are higher in stiffer (high elastic modulus) materials. Moreover, when a stress wave passes through a void, honeycombed, or poor quality material, the speed is significantly reduced. With this information the inspector can compare areas to determine if a deficiency is present within the material. If a crack or debonded area is present within the test material, the emitted stress wave will reflect back or around the crack, depending on the location of the crack tip. Stress waves that are reflected back are not picked up by the emitting transducer and therefore results as no reading. Stress waves that a closer to the crack tip may manage to travel up or around the crack to the receiver transducer. This would show up as a decreased signal and longer pulse time. Refer to Figure 5.13.1-2 for an illustration of several stress wave interactions with defects.

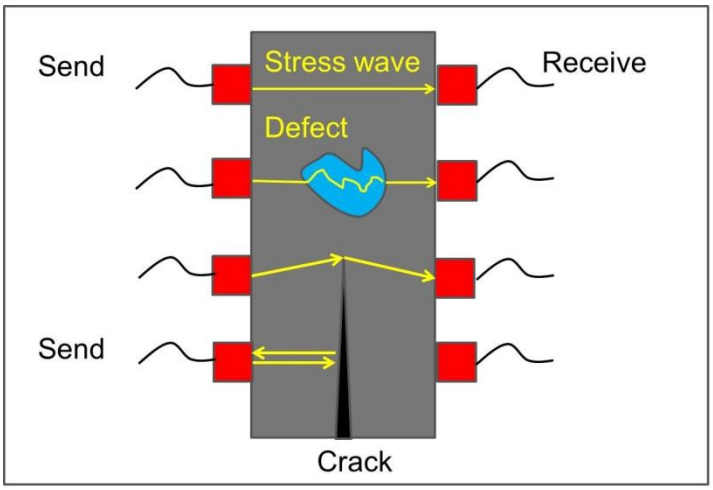

Figure 5.13.1-2: Illustration of Several Stress Wave Interactions with Defects.

5.13.2 Applications

Perhaps the most advantageous aspects of Ultrasonic Pulse Velocity (UPV) are it is highly portable and highly repeatable. As stated before, this method is most commonly used on concrete and timber materials. In concrete is offers in-situ testing of concrete strength. It is used to identify poor quality or damaged concrete. It is effective at determining if an internal deficiency, such as void, crack or poor quality is present. In some instances UPV is used to determine the effectiveness of repairs. The method can be used to determine whether a structural concrete patch is adequately bonded or if the quality is suitable to the sound material.

Similarly, UPV is used on timber members to determine any internal voids or decay that may be present. This method is also used in underwater inspections of timber piling. Refer to Figure 5.13.2-1 for an image of an inspector using UPV on a concrete pier cap.

Figure 5.13.2-1: Inspector Conducting UPV on a Concrete Pier Cap Using Semi-Direct Transmission.

5.13.3 Limitations

Ultrasonic Pulse Velocity (UPV) should not be performed on rough surfaces, on parts with complicated geometries, or highly attenuated materials. Rough surfaces may require grinding in the surface preparation. Other factors, which limit the successful application of UPV are: lack of properly trained personnel, over estimation of the accuracy of flaw locating and sizing, and poorly written testing procedures. Typically a certified Level III nondestructive evaluation (NDE) specialist should evaluate and develop written testing procedures for uncommon applications.

Unlike UT, UPV must have access to more than one side of a member being tested. This may disqualify UPV in certain applications. When UPV is used for strength determination of concrete, typically a correlation is created by using other testing methods. When testing timber, the effectiveness of UPV is compromised in certain species of wood.

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5.14 COVERMETER TESTING

5.14.1 Introduction

Covermeters or pachometers, often referred to as R-Meters, are electromagnetic devices that detect the reinforcing steel in concrete and measure its size and the depth of concrete cover. The device produces a magnetic field and locates the reinforcing steel by measuring the distortion of the magnetic field created by the presence of the steel. The signal received increases with increasing bar size and decreases with increasing cover thickness. Using more than one sensor, the covermeter can be calibrated to convert the signal into a distance, which indicates the depth of cover.

The importance of measuring depth of cover is highlighted by the relationship between cover depth and deterioration mechanisms. Inadequate cover can undermine the protection that the concrete provides to the steel reinforcement from corrosion. Carbonation, the chemical process where carbon dioxide and moisture react with cement products, begins as soon as concrete is exposed to air. The process of carbonation neutralizes the protective alkaline nature of the concrete. If the cover is too shallow, the carbonation will reach the level of the reinforcing steel and the alkaline protection will be lost, potentially leaving the steel susceptible to corrosion. The exposure of the concrete and reinforcing steel to road salts and moisture will further accelerate the corrosion process once the cover has been breached by cracks or spalls. If the cover is too deep, there is the possibility of increased crack widths and decreased effective depth, which both affect design parameters on a concrete member. Refer to Figure 5.14.1-1 for a view of a covermeter.

Figure 5.14.1-1: View of a Basic Covermeter Unit.

5.14.2 Applications

Accurately locating the reinforcing steel in concrete allows the inspector to determine if the steel is placed outside of the zone of carbonation. Covermeters, in general, can accurately measure the cover depth to within of an 0.25 inch in the range of 0 to 3 inches in lightly reinforced structural members. Covermeters can also be used to locate reinforcing steel for

the purpose of “tying-in” a new structural member to an existing structure. This process typically occurs during rehabilitation and involves drilling into the existing structure and added reinforcing steel, which spans from the old to the new components.

5.14.3 Limitations

The effectiveness of a covermeter as a nondestructive testing method is limited by several factors. A covermeter only locates the reinforcing steel and does not provide any actual information about defects or the material’s state of deterioration. Secondly, it does not distinguish if one or more bars are present at a certain location and therefore, the intensity of the signal may be misinterpreted and the cover depth can be incorrectly noted as shallower than the true depth. This problem is most pronounced in heavily reinforced structures or when large steel objects, such as scaffold, are near the test area. Also, some reports indicate that the epoxy coatings on reinforcing steel can distort the readings of an Covermeter. Likewise, the relative material properties of the concrete often must be assumed to utilize conversion charts for the readings.

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5.15 HALF-CELL TESTING

5.15.1 Introduction

Steel reinforcement is typically protected from corrosion by the alkaline nature of concrete, which creates a protective passivation layer around the steel. If the alkalinity of the concrete is compromised, corrosion on the steel will commence provided that moisture and oxygen are present. The corrosion reaction will promote anodic and cathodic activity along the reinforcing steel. The corrosion of the reinforcement produces a corrosion cell caused by these differences in electrical potential.

The Half-Cell testing method uses this process to detect whether the reinforcing steel is under active corrosion. This method utilizes a multimeter to measure the potential difference between the steel and a half-cell apparatus. The analysis of the potential difference can indicate if active corrosion is taking place on the reinforcing steel. Refer to Figure 5.15.1-1 for a schematic of a basic half-cell test.

This test is described in American Society for Testing and Materials (ASTM) C876-91, "Standard Test Method for 'Half-Cell' Potentials of Reinforcing Steel in Concrete."

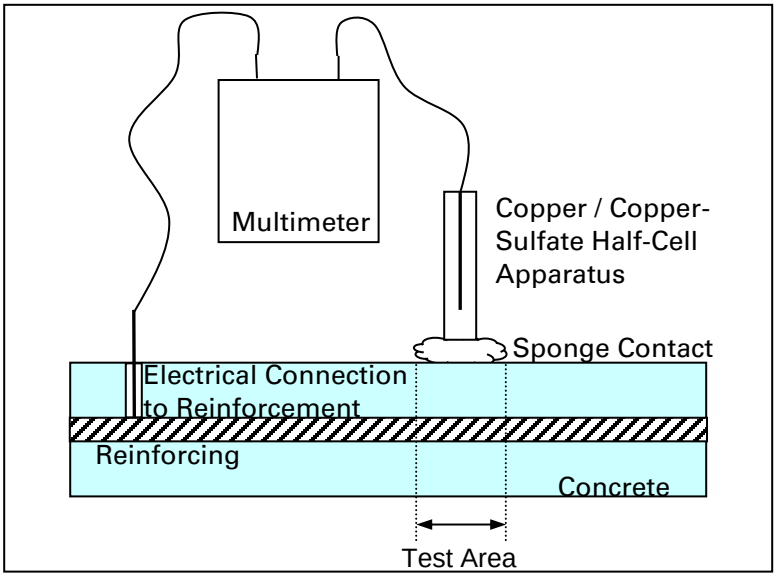

Figure 5.15.1-1: Basic Half-Cell Test Configuration.

5.15.2 Applications

Although commonly used on bridge decks, the half-cell test can be performed on any reinforced concrete component, provided a direct electrical connection can be made to the reinforcing steel. Since the test can only detect corrosion directly under the device, a systematic grid of test points should be created to map the potential readings throughout the concrete component. This map can then be analyzed to determine the probable areas of active corrosion.

It is generally agreed that the potential measurements can be interpreted as follows:

1. 0.00 to –0.20 volts indicates greater than 90% probability of no corrosion.

2. -0.20 to –0.35 volts indicates that corrosion is uncertain.

3. < -0.35 volts indicates greater than 90% probability that corrosion is occurring.

4. A positive number indicates that the moisture content of the concrete is insufficient and therefore, the test is not valid.

5.15.3 Limitations

Half-Cell testing requires specialized equipment, typically a copper/copper-sulfate half-cell apparatus and a multimeter. A connection with the reinforcing steel is required and therefore, holes may need to be drilled in the concrete to locate and connect to the steel. This test method only indicates the probability of corrosion present at the time of testing, and does not indicate the extent or rate of corrosion.

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5.16 CHLORIDE ION TESTING

5.16.1 Introduction

Chloride ions are the major cause of reinforcing steel corrosion in concrete, especially in Wisconsin. Chloride ions are most often provided from road salt, although they may also be available as contaminants in the original concrete mix. Similar to carbonation, chloride ions destroy the passivation layer surrounding the reinforcing steel which protects it from corrosion. Although present, these chloride ions are not likely to cause problems unless they exist in unusually high concentrations. Since corrosion of steel reinforcing is generally considered to begin at a chloride ion content of between 0.025 percent and 0.033 percent by weight of concrete, knowledge of chloride content can aid in determining the likelihood of the onset or presence of corrosion.

In evaluating chloride content, it is recommended that a chloride profile (chloride concentration percentage versus depth measurement below the concrete surface) be developed. This profile is important for assessing the future corrosion susceptibility of steel reinforcing and in determining the primary source of chlorides.

The chloride content in concrete is typically determined through laboratory analysis of powdered concrete samples. The powdered samples can be obtained on-site or in the laboratory. Field collected powdered samples are typically taken by drilling at different depths down to and beyond the level of the reinforcing steel. Extreme care should be exercised to avoid inadvertent contamination of the samples. Alternatively, cores can be collected and powdered samples can be obtained at different depths in the laboratory. The collection of these samples in essence actually destroys a portion of the component making this test procedure similar to Material Sampling as described in Chapter 23 of Part 5. However, since this test can be performed in the field and results obtained quickly, it has been separated from the material sampling section.

The chloride ion content of concrete is usually measured in the laboratory using wet chemical analysis. Although laboratory testing is the most accurate, it is time consuming and often takes several weeks before results are available. As a result, field test kits have been developed. The use of field test kits allows rapid determination of chloride levels to be made on-site. Although the field kits are not as accurate as the laboratory method, they do provide good correlation with laboratory tests when a correction factor is used.

The detailed procedure for chloride sampling and testing is covered in American Association of State Transportation and Highway Officials (AASHTO) T 260-84; “Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials” and in American Society for Testing and Materials (ASTM) C114-00; “Standard Test methods for Chemical Analysis of Hydraulic Cement.” However, both of these publications apply to testing in the laboratory, not in the field.

5.16.2 Applications

Chloride ion testing can be performed on any concrete component. Field kits allow inspectors to perform the test on-site and determine chloride levels immediately.

When samples are collected at different depths and plotted on a chloride profile chart, this method is a very useful tool in determining the depth of deck to be milled off prior to an overlay.

5.16.3 Limitations

Collecting samples to perform this test requires a portion of the concrete member be damaged and the tested area to be destroyed. Therefore, several samples can't be taken from a single location to validate results. This method is also a time consuming process, and requires access to the member. In the case of a bridge deck, the bridge may need to be closed to traffic during the sampling process.

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5.17 CARBONATION TESTING

5.17.1 Introduction

Concrete begins reacting with carbon dioxide the instant it is exposed to the air. Concrete itself is highly alkaline (high pH). This high alkalinity reacts with the bare steel on reinforcing steel and creates an oxide film around the steel called a passivation layer. This layer protects the steel from corrosion. The reaction of concrete with carbon dioxide reduces the pH in the concrete. This process starts at the surface and works its way into the concrete. The reaction can be expedited by cracking in the concrete surface.

The reaction of concrete with carbon dioxide does not necessarily compromise the strength of the concrete. As an inspector, the concern is if the extents of the reaction reach the reinforcing steel, the pH will drop to the point where the oxide film around the steel becomes compromised. Once this occurs the steel is vulnerable to the electrochemical process of corrosion.

Carbonation testing provides a means with which the inspector can determine the extent of carbon dioxide infiltration into the concrete. The process is similar to chloride ion testing where a sample is either removed, either by coring or drilling and the sample is tested by the application of a revealer. The revealer commonly used is phenolphthalein.

When phenolphthalein comes into contact with high pH (>10) concrete the solution shows as bright pink. When the solution comes into contact with low pH (<10) the solution shows no color change and the concrete can be considered carbonated.

Cracking in concrete allows the open air to penetrate further into the concrete surface. Cracking near reinforcing steel allows the pH in the concrete surrounding the reinforcing to be reduced that much sooner. Refer to Figure 5.17.1-1 for an image of various test results on cracked concrete samples.

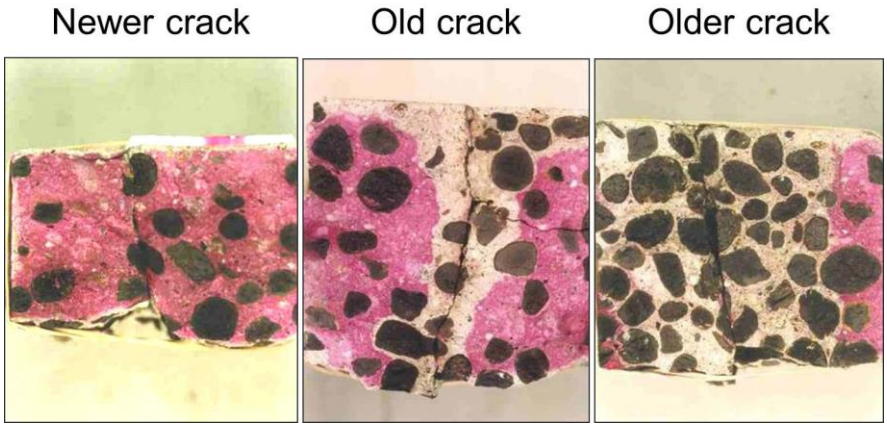

Figure 5.17.1- 1: Image of Cracked Concrete Samples After Carbonation Testing. Note the Older Crack and the Amount of Carbonation in the Concrete.

Some benefits of the Carbonation Test are that the results are immediate. There is no need to remove the sample and ship to a laboratory. All testing can be done in the field. Also the

revealer (phenolphthalein) is environmentally safe. Moreover the test can be performed by anyone. There is no need for special training. However, determination of where tests are to be taken should be discussed and planned upon beforehand.

5.17.2 Applications

Carbonation testing can be performed on any concrete component. Field kits allow inspectors to perform the test on-site and determine carbonation extents immediately.

5.17.3 Limitations

Collecting samples to perform this test requires a portion of the concrete member be damaged and the tested area to be destroyed. Therefore, several samples can't be taken from a single location to validate results. The test determines when the concrete is approximately above or below a pH of 10. Steel is protected in a pH of 12.5 or greater. Therefore the test may not indicate the carbonation extents if the pH is between 10-12.5 even through this level is compromising the steel passivation layer. In the case of a bridge deck, the bridge may need to be closed to traffic during the sampling process.

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5.18 REBOUND HAMMER (SCHMIDT HAMMER)

5.18.1 Introduction

A rebound hammer, commonly referred to as a Schmidt Hammer, is a mechanical device used to directly measure the hardness of in-place concrete and the calibrated compressive strength. The device consists of a plunger and a spring-loaded hammer. When triggered, the hammer strikes the free end of the plunger that is in contact with the concrete, which in turn causes the plunger to rebound. The extent of the rebound is measured on a linear scale attached to the device. Refer to Figure 5.18.1-1 for a view of a standard rebound hammer.

This test is covered in American Society for Testing and Materials (ASTM) C805-97, "Standard Test Method for Rebound Number of Hardened Concrete."

Figure 5.18.1-1: Standard Rebound Hammer.

5.18.2 Applications

The rebound hammer is used to assess the uniformity of in-situ concrete and to delineate zones of poor quality or deteriorated concrete. It is also useful to detect changes in concrete characteristics over time, such as hydration of cement, for the purpose of removing forms or shoring.

Advantages of the rebound hammer are that it is portable, easy to use, low cost, and can quickly be used to cover large areas and highly repeatable.

5.18.3 Limitations

The rebound hammer is valuable purely as a qualitative tool since it only measures the relative surface hardness of the concrete. Other tests, such as a compression test, must be used to determine the actual strength of the concrete. The rebound measurement is governed by several factors including the size, age, and finish of the concrete, as well as the aggregate type and the moisture content. Also, the Rebound Hammer Test should not be done over exposed aggregate or a false reading will be given.

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5.19 WINDSOR PROBE

5.19.1 Introduction

The penetration test, typically utilizing the Windsor Probe test system, consists of a device that drives a probe into the concrete using a constant amount of energy. The probe is made of a hardened steel alloy specifically designed to crack the aggregate particles and to compress the concrete being tested. Once fired, the length of the probe projecting from the concrete is measured. A test typically consists of firing three probes and averaging the projecting lengths. Refer to Figure 5.19.1-1 for a view of a Windsor Probe Test Kit.

This test is covered in American Society for Testing and Materials (ASTM) C803/C803M-97C1, "Standard Test Method for Penetration Resistance of Hardened Concrete."

Figure 5.19.1-1: Windsor Probe Test Kit.

5.19.2 Applications

Penetration tests are used to assess the uniformity of in-situ concrete and to delineate zones of poor quality or deteriorated concrete. It is also well suited for estimating compressive strength of concrete and the relative strength of concrete across the same structure. Penetration tests are commonly used to estimate early age strength of concrete for the purpose of stripping forms.

5.19.3 Limitations

The penetration test method is basically a qualitative tool, and like the rebound hammer, requires that other tests be conducted to determine the actual strength of the concrete being tested. The penetration test requires an appropriate amount of accessibility to the concrete surface being tested. The probe must be able to be oriented perpendicular to the surface. The use of the penetration method also damages the concrete at the test location. The probes must be removed and the holes patched.

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5.20 RESISTANCE MICRO-DRILL

5.20.1 Introduction

The presence of decay in timber bridges can be difficult to detect. But the extent and location of such decay can have significant effect on the engineering characteristics of timber and the load rating of timber bridges.

Visual inspection (Chapter 2) and sounding (Chapter 3) the timber elements with a hammer is the most basic type of inspection. But these can be difficult to interpret, and at best, can only give you an indication that decay is present, but cannot be used to determine the extent of the decay. It also has limited detection on large timber members, on members like piles that have surface delamination or damage near the water line and on large preservative treated timbers that may have an intact exterior, where the preservative treatment has penetrated, but has a decayed core.

In the past, increment boring tools were often used to determine the extent of the decay. But that tool is slow, takes a lot physical strength by the inspector, is difficult to interpret, and is a partially destructive testing method. Drilling into the timber with a brace and bit has also been used and the resistance can estimate the extent of decay. Boring of any type leaves large holes which are hard to seal and become a source for accelerated decay should only be used as a last resort.

Stress Wave Timbers (Chapter 21) and resistance micro-drilling can both be used to detect the extent of decay. Resistance micro-drilling pushes a small 1.5mm bore (with a flared 3mm tip) into the wood and a microprocessor measures the relative resistance. The bore does not drill into or remove wood, but forces its way into the fibers, thus it is a NDE. In most cases, the timber's preservative treatment envelope is not violated. The wood's resistance to the micro-bore is either recorded on paper or is saved to a computer file. This allows the operator to determine the amount of decayed wood and remaining cross section. The resistance of the wood depends on the species, moisture content, and whether the temperature is freezing, and the direction of the grain. Calibration of the instrument is not as important as taking a baseline reading on known sound wood and comparing that to the rest of the borings in similar wood.

This instrument can contribute to a much higher level of confidence in timber bridge inspections. Newer instruments can do a bore in about a minute and the batteries will last about 100 bores. Older machines are slower and more battery intensive.

Models available are the IML Resistograph. Older models are available for around $4,000, but the newer, faster models are recommended and cost about $7,500.

5.20.2 Applications

Before using a resistance micro-drill, the entire bridge structure should have a preliminary inspection by quickly hitting all the bridge members with a hammer and listening for a resulting hollow sound. The inspector should visually observe the overall condition of the timber, note the presence of moisture, and the age of the structure. A bridge with no discernible defects need not have further analysis with NDE. Bridges less than 30 years old normally will not need this extensive of testing. But if any of the timber is suspected of

having decay, its presence and extent must be confirmed by NDE methods. For structures where decay is suspected and detected, it is good practice to survey the entire structure. This baseline can be used to measure the progression of decay and aid in determining whether and when the structure must have repairs and/or be replaced. All data should be stored in the hard copy file and scanned in and stored in the HSI database. Future inspection can then estimate if the decay is spreading. All determinations of cross section loss should be given to a bridge load rating engineer to determine whether posting is needed. Refer to figures 5.20.2-1 and 2 for images of Resistance Micro-Drill being used on piling.

Figure 5.20.2-1: IML Resistograph, note the other old borings from a regular drill

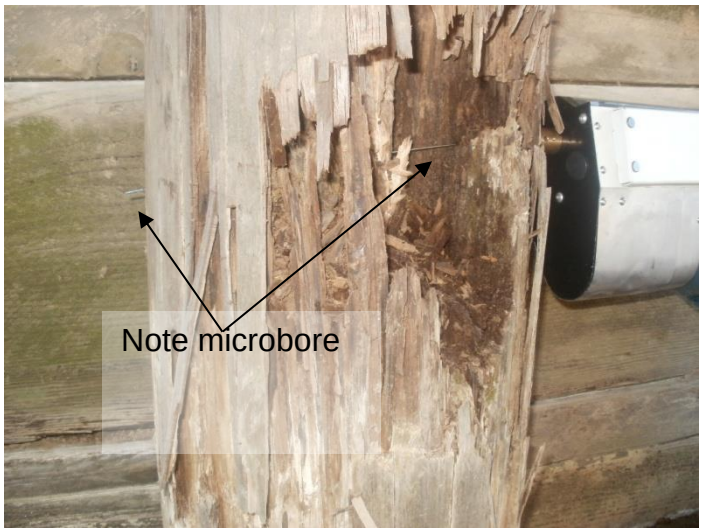

Figure 5.20.2-2: Photograph of Resistance Micro-Drill boring through severely decayed pile.

Resistance micro-drills can be used on timber bridge beams, decks, abutments, piles, and railings. Forms should be used to facilitate the tracking of the data from year to year, and to accurately note where the testing was done and the extent of the suspected rot and its

increase over time. The forms documenting the data must be saved in the hard copy file and scanned and placed in the HSI database. Refer to figures 5.20.2-2 thru 5.20.2-4 for images taken from a resistance micro-drill.

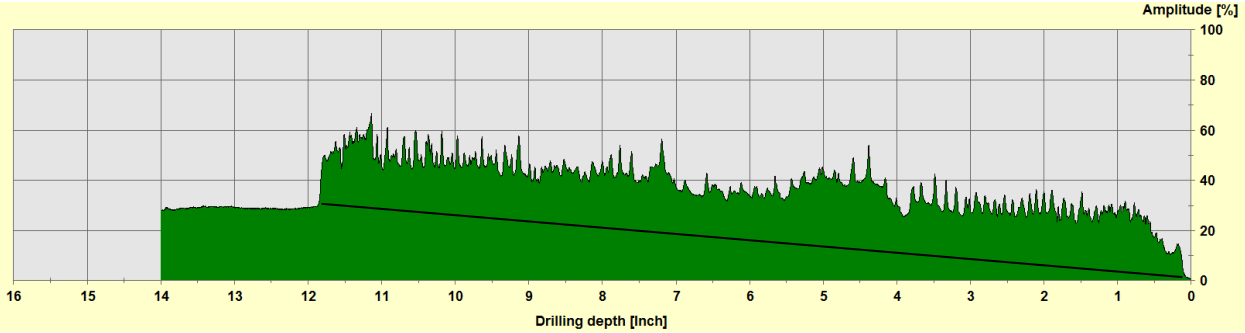

Figure 5.20.2-3: Micro-drill output of good, sound piling

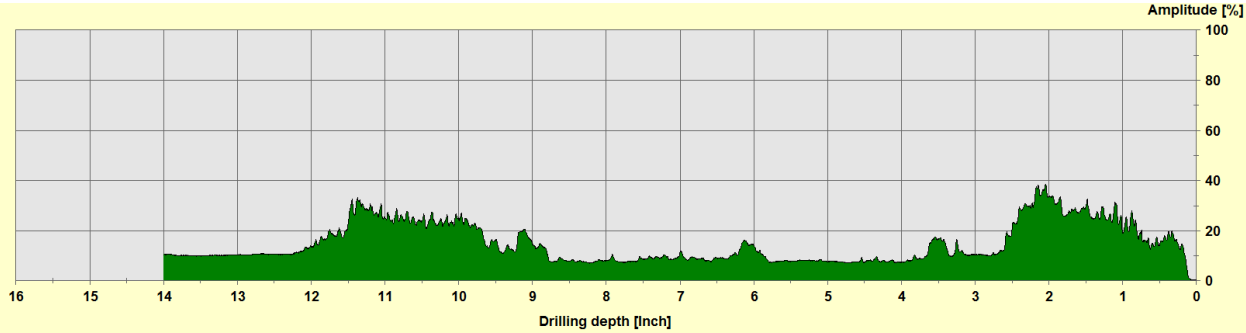

Figure 5.20.2-4: Micro-drill output a decayed pile showing 7” hollow.

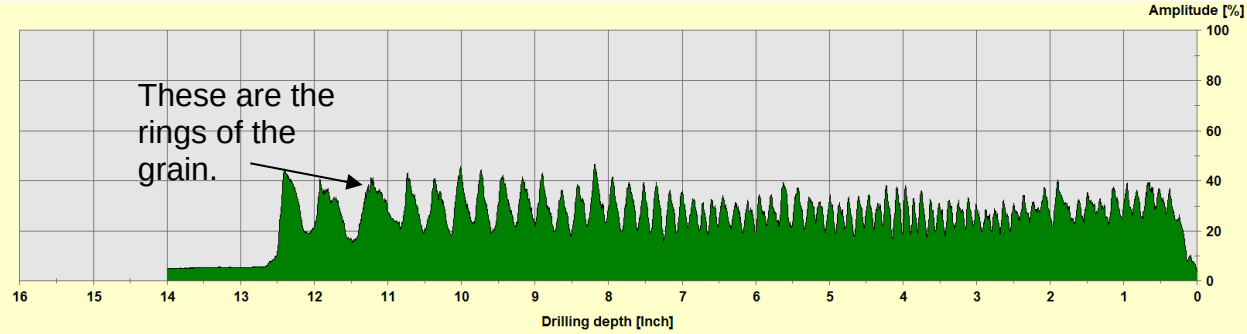

Figure 5.20.2-5: Micro-drill output of good, sound beam.

5.20.3 Limitations

Limited access caused by closely spaced member may limit areas where the micro-drill can be used.

The micro-drill cannot make absolute distinction between sound wood and incipient decay. Comparisons to bores in known sound wood must be used and experience must be used to accurately judge the decay. The original grading of the timber takes into account naturally occurring defects in the wood. As wood is decaying it is difficult to judge to when the decay is impacting the engineering characteristic of the wood and how the decay advances over time.

The soundness of the wood is only measured at the bore's penetration, multiple bores can be made and all areas of suspect wood can all be bored in different direction to estimate the extent of the decay. Judgment must still be used in determining the areas to bored and it is very possible to miss a decayed area. Voids are easily detected and usually one can tell if this is a natural defect (not a structural issue as the grading takes those into account) or advanced decay.

A way to charge the batteries off a vehicle's electrical system must be carried as part of the inspector's tools. A laptop can also be taken to download files.

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5.21 STRESS WAVE TIMERS

5.21.1 Introduction

The presence of decay in timber bridges can be difficult to detect. But the extent and location of such decay can have significant effect on the engineering characteristics of timber and the load rating of timber bridges.

Sounding the timber elements with a hammer is the most basic type of inspection (Chapter 3). But this can be difficult to interpret, and at best, can only give you an indication that decay is present, but cannot be used to determine the extent of the decay. It also has limited detection on large timber members, on members like piles that have surface delamination near the water line (giving a false positive), and on large preservative treated timbers that may have an intact exterior, where the preservative treatment has penetrated, but has a decayed core.

In the past, incremental boring tools were used to better judge the extent of the decay. But that tool is slow, takes a lot physical strength by the inspector, is difficult to interpret, and is a destructive testing methods. Increment boring should only be used as a last resort.

Stress Wave Timbers (SWT) and resistance micro-drilling (Chapter 20) can both be used to detect the extent of decay. SWT measures the time that a pulse travels through a timber. The pulse travels much faster in sound, dense wood than in decayed wood, so decay can be detected and its extent can be quickly estimated. There is no penetration of the preservative envelope on the timber. Though judgment must still be used, this can contribute to higher level of confidence in timber bridge inspections. There are several brands of stress wave timers with each fast to operate. Pulses are either generated with a hammer hit, or the instrument generates its own pulse. The time for the pulse to travel in the wood is registered by the instrument and is noted by the inspector and can be saved to a computer file.

SWT for timber is similar to Ultrasonic Testing (Chapter 5) and Ultrasonic Pulse Velocity (Chapter 13).

5.21.2 Applications

Before using SWT, the entire bridge structure should have a preliminary inspection by quickly hitting all the bridge members with a hammer and listening for a resulting hollow sound. The inspector should visually observe the overall condition of the timber, note the presence of moisture, and the age of the structure. A bridge with no discernible defects need not have further analysis with NDE. But if any of the timber is suspected of having decay, its presence and extent must be confirmed by NDE testing.

For structures where decay is suspected and detected, it is good practice to survey the entire structure. This baseline can be used to measure the progression of decay and aid in determine whether and when the structure must have repairs and/or replaced.

SWT can be used on timber bridge beams, decks, abutments, piles, and railings. Forms should be used to facilitate the track of the data from year to year, and to accurately note where the testing was done and the extent of the suspected rot. Diagrams must be made

and used to record the location of the testing, and it must be saved in a location to assure future inspectors have access to the information.

The inspector must analyze the results, summarize the finding and include this in the inspection folder and the HSI database.

SWT does not measure an absolute value of soundness of wood. The speed of the pulse is determined by decay, but also the species of the wood, and the presence of other defects such as splits. Therefore, a baseline should be used on a known sound piece of wood on the bridge or other similar sized and treated timber.

On a creosote treated Douglas Fir 12x12 pile cap, the typical time for a pulse to travel through the wood is between 400 and 600 microseconds. But in a decayed pilecap, the time will be 1500 or more microseconds.

SWT is fast and more than a hundred readings can be taken in a day.

Models available are the Fakopp Microsecond Timer, Metriguard Stress Wave Timer, and the Silvatest Duo. Prices range from $2,000 to $4,000.

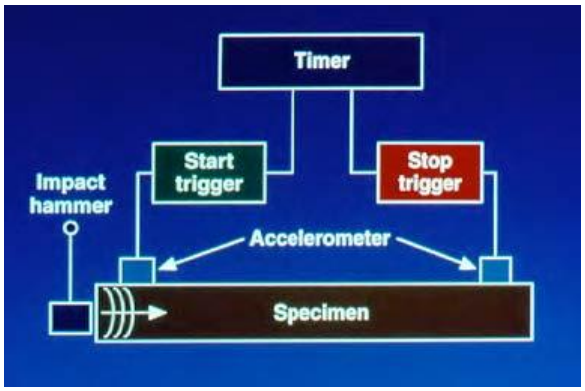

Figure 5.21.2-1: Schematic of a Stress Waver Timer

Figure 5.21.2-2: Stress Wave Timer using impact hammer

Figure 5.21.2-3: Stress Wave Timer using automated pulses.

5.21.3 Limitations

This method will only work on timber with access to two sides. Decks usually will not have two sided access, and some beams may be too close together to use SWT. Proper interpretation requires comparing to a timber that is known to be sound. False positive can be caused by internal split in the wood that does not impact the structural characteristics, and by delamination of the rings on the surface of piles at the waterline. The latter case is most likely caused by decay occurring and its limitation often means that the SWT cannot be used.

Experience is necessary to estimate the extent of the detected rot.

The exact location of the decay cannot be determined, only that there is decay. In most cases, a resistance microbore is a better choice (Chapter 20).

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5.22 MONITORING SYSTEMS

5.22.1 Introduction

In contrast to using other nondestructive evaluation (NDE) methods where the inspector conducts an inspection over a finite time interval, Monitoring Systems provide continuous data over an indefinite time interval since the monitoring devices are typically mounted to the structure, connected to a data collection device, and left by the inspector to monitor the structure. These systems can be limited to monitor a specific component or section of the structure, or be designed to encompass the entire structure. The scope of monitoring is dependent on the desired data, the identified potential problem areas of the structural components, and/or potential areas of structure movement.

A monitoring system can be comprised of a variety of sensors and a data collection device (computer). In the case of a remote system, a communication device, which transmits the data to the monitoring station for analysis would be used. Refer to Figure 5.22.1-1 for a view of a remote system computer and communication device.

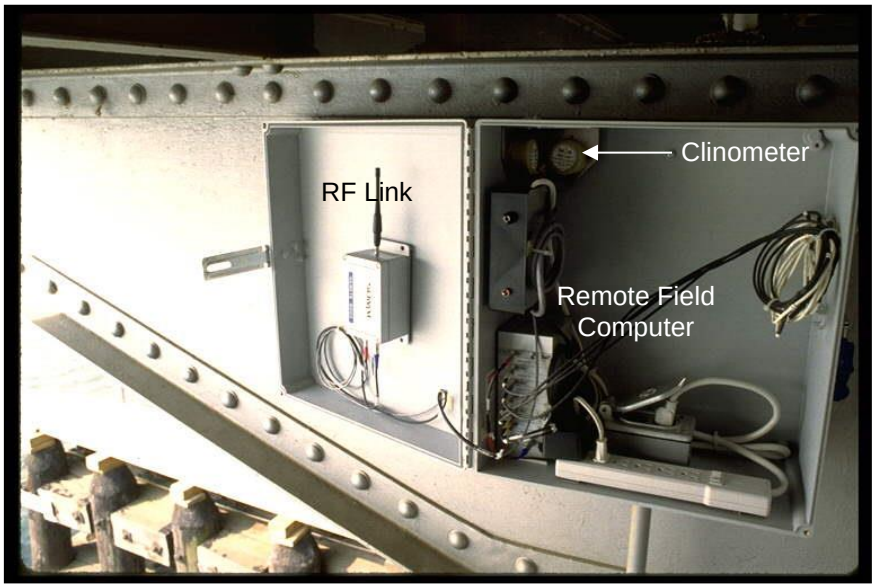

Figure 5.22.1-1: Remote Field Computer, Clinometers, and RF Link Mounted in Protective Enclosure.

Sensor types include strain gauges, clinometers (tilt meters), accelerometers, thermocouples, as well as various other devices.

1. **Strain Gauges:** The term “strain gauge” typically covers a wide range of devices that are used, as their name implies, to measure the strains of structural members under load.

2. **Clinometer:** Measures inclination or tilt of a structure or structural component.

3. **Accelerometer:** Measures structure dynamics under conditions such as high winds, seismic activity, and/or vehicular traffic.

4. **Thermocouple:** Measures the temperature of a structure or its environment.

5. **Transducers:** In Acoustic Emission (AE), these devices detect stress waves emanating from actively growing defects within a bridge structure. Refer to Part 5 Chapter 6 for more information on AE.

Strain gauges are the predominant sensor in use today. A variety of strain gauges are available, and their selection should be made based on the location of their use, as well as the specific data type and amount to be collected. A strain gauge will have an established initial set length, the "gauge length," with a corresponding electrical resistance, which is used as a datum. Strain is a measure of deformation, specifically the change in length of fibers within an object relative to their original length due to an applied stress. Strain gauges measure this change via change in electrical resistance due to the lengthening or shortening of the resistive element within the gauge that is bonded to the component. This change in resistance is proportional to the strain within the member, related through a set of equations. This calculation is typically performed automatically by the data acquisition system. Strain however, is often a very small value, typically measured in microstrain (strain x 10^6) and can be affected by such factors as temperature and lead wire resistance, so it is important to have a good understanding of the technology for proper application and obtaining accurate results. Strain gauges can also be used to measure rotational strain. Strain gauges can be placed either externally, or in certain circumstances, within the object being examined. Groups of gauges are usually installed in patterns determined by the type of data desired. Strain gauges are typically small and flat, and do not interfere with the use of the structure. The strain gauges are connected to a data acquisition system which records the strain data. Under real-time loading situations, the acquisition system can automatically collect data at a user-defined time interval. To obtain a comprehensive strain-time history, data is acquired rapidly, measured in recordings per second or hertz (Hz). This rate of acquisition is required to capture specific loading events (i.e. truck impact). Other applications may only require occasional readings.

In the area of transportation structures, strain gauges may be installed at carefully selected locations on a structure to measure strains under live load conditions. These strains may be due to traffic, wind, temperature, or specifically applied test loads. This strain data is then evaluated directly, or more often converted to stresses that can be compared to calculated design stresses. Thus, strain data allows the real performance of a structure to be compared to the theoretical design, and can provide data for an analytical model to more accurately predict actual performance. Strain gauges are also often employed to study the performance of a local area or detail for which theoretical analysis may be difficult.

In field situations, strain gauges may also be used as monitoring systems at locations of great concern, either of movement, or of changes in stresses. Refer to Figures 5.22.1-2 and 5.22.1-3 for views of strain gauges attached to steel bridge components. This work may be required to verify the safety in areas of uncertain stability or strength. In a system of enough sophistication, movement beyond a certain range may cause alarms to sound at the structure or at an off-site monitoring station. Therefore, strain gauges may be used at locations where it is difficult or prohibitively expensive to replace an existing structure of uncertain strength, while still allowing safe use of the structure.

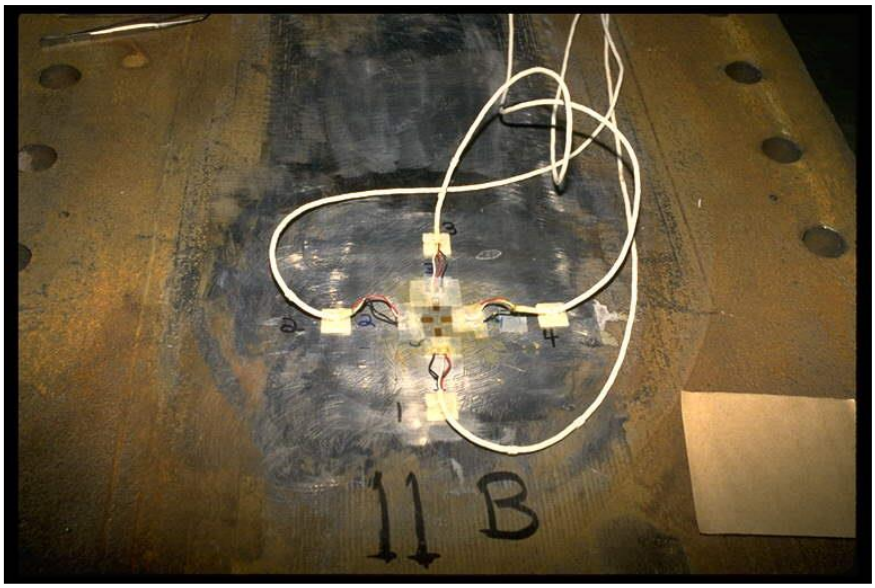

Figure 5.22.1-2: Strain Gauges Attached to the Steel Track Casting of a Movable Bridge.

Figure 5.22.1-3: Strain Gauges Attached to the Tie Girders of a Tied Arch Bridge.

5.22.2 Applications

Monitoring systems have many applications in the inspection of structure components. The sensors are very versatile and can be applied to many materials. They typically are small and can be attached in tight fitting areas. Furthermore, many of the sensors have a high level of accuracy and can be applied in both static and dynamic situations. Once installed, the sensors can provide data for an indefinite period of time.

The ability to continuously monitor structures or specific components, allows the owner to record and clearly observe the performance and detect deterioration. These systems work well from a preventative maintenance stance.

5.22.3 Limitations

Monitoring systems; however, do have some limitations. Sensors take measurements at specific locations, which is useful in many applications, however to obtain a global response of the structure would require a great multitude of gauges which may be cost prohibitive or impractical.

The sensors, although typically inexpensive, are often one-time use devices. Once mounted to a particular structure they cannot be removed and reused for another application. The sensors also typically require a high level of expertise to install.

The data collection and transmission devices, on the other hand, can be expensive and require specialized individuals to maintain and process the data. The gathered data must also be analyzed and manipulated to provide useable information.

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5.23 MATERIAL SAMPLING

5.23.1 Introduction

To fully determine the condition of a structure, it may sometimes be necessary to extract material samples from the structure so that laboratory tests may be run to better determine the condition of the structure's materials or states of deterioration or damage. Typical laboratory tests may include compressive tests and petrographic examination of concrete; tension tests, Charpy tests, or crack surface investigations of steel; or even simple integrity examination of timber.

Prior to obtaining any samples, the extent and purpose of the sampling must be determined. The sample size is often stipulated in the specific test methods to be used. In most cases, particularly where deterioration is present, it is advisable to take samples from both good and bad areas so that a comparison can be made. Once the number and location of samples is determined, they should be plotted on a drawing of the structure both to aid in fieldwork and serve as a record for the evaluation of the test results.

5.23.2 Applications

All materials can be sampled and tested either in the field or in a laboratory to provide useful information as to the strength, extent of deterioration, and material characteristics. Specimens should come from representative areas of the structure and typically three samples are required.

All material samples should be collected and tests conducted in accordance with applicable American Society for Testing and Materials (ASTM) and American Association of State Transportation and Highway Officials (AASHTO) methods for the respective materials.

5.23.3 Limitations

The removal of material from the structure should only be conducted when a specific piece of information is required and the information attained provides useful information in the evaluation of the structure.

The extracting of samples will leave holes or voids in the tested component and therefore, repairs are required. Concrete and timber repairs are relatively easy, but steel repairs may be more complex. Welding requires the use of experienced personnel and care should be taken to minimize any residual stresses or fatigue prone details associated with the repair.

5.23.4 Concrete Testing

Concrete material sampling most often consists of drilled cores, though sections may also be obtained by sawing or breaking off a portion of the component. The core size should be determined by the tests to be run; however, in most cases a four-inch diameter core is extracted. Core holes are normally filled with grout; other sample areas should also be repaired with a suitable mortar material. When feasible, steel reinforcement should typically be avoided unless sampling specifically requires it to be part of the core.

Samples should be marked for location and orientation, and packed to prevent damage during transport. As part of the sampling operation, reinforcing steel is typically located and marked to avoid cutting it during the sample extraction. In some instances, it may be desirable to include reinforcing steel as part of the sample. In these cases, it is necessary to confirm that the cut reinforcing steel will not jeopardize the structure's integrity. Refer to Figure 5.23.4-1 for a view of a concrete coring machine and core sample.

Some of the concrete tests that require samples are the following:

1. Carbonation
2. Chloride Ion Content
3. Permeability
4. Cement Content
5. Percent Air Content
6. Moisture Content
7. Steel Reinforcing Yield Strength
8. Concrete Compressive Strength
9. Modulus of Elasticity (static & dynamic)
10. Concrete Splitting Tensile Strength

Figure 5.23.4-1: Concrete Coring Machine (Left) and Core Sample (Right).

5.23.5 Steel Testing

Material coupons for steel members are usually obtained by sawing, coring, or by collecting drill shavings. Coupons may be flame cut; however, the heat induced by the cutting

operation alters the material's properties in the vicinity of the cut both in the sample and remaining base material. These heat-affected areas must then be removed by grinding prior to testing. Repair to the base material is also often required. For these reasons, flame cutting should typically be avoided. In selecting coupon locations to test material properties such as yield strength or toughness, it must be remembered that the properties of steel members vary over the cross-section as a result of varying rates of heat loss due to fabrication techniques and rolling/production practices. Note that the orientation of the steel samples should be recorded prior to removal.

Some of the steel tests that require samples are the following:

1. Brinell Hardness Test
2. Charpy Impact Test.
3. Chemical Analysis
4. Tensile Strength Test

Refer to Figure 5.23.5-1 for a view of the Charpy Impact Testing Machine.

Figure 5.23.5-1: Charpy Impact Testing Machine.

5.23.6 Timber Testing

Historically Timber sampling often consisted of the use of incremental borer to extract cores, though sections may also be obtained by sawing off a portion of the component. Core holes should be plugged with a treated hardwood dowel.

Some of the timber tests that require samples were the following:

1. Extent of Rot
2. Location of Voids
3. Preservative Penetration

. Refer to Figure 5.23.6-1 for a view a several incremental borer core samples.

These cores are assessed to determine if bacterial or fungal decay is present, the extent of interior rot, and to determine the species of timber, if required. However, these methods typically do not produce a global sample specimen; however, several local specimens from random locations can be effective. Any holes should be plugged with a treated hardwood dowel.

Moisture content and rot can also be assessed on specimens using electrical devices, such as the Shigometer. These devices require electrodes to be driven into the timber or that small holes be drilled to insert probes into the timber. These detect the presence of timber rot; however, drilling or coring should be conducted to determine the extent of the rot.

Figure 5.23.6-1: Typical Incremental Borer Core Samples.

The presence of decay in timber bridges can be difficult to detect. But the extent and location of such decay can have significant effect on the engineering characteristics of timber and the load rating of timber bridges.

Sounding the timber elements with a hammer is the most basic type of inspection (Chapter 3). But this can be difficult to interpret, and at best, can only give you an indication that decay is present, but cannot be used to determine the extent of the decay. It also has limited detection on large timber members, on members like piles that have surface delamination near the water line (giving a false positive), and on large preservative treated timbers that may have an intact exterior, where the preservative treatment has penetrated, but has a decayed core.

In the past, incremental boring tools were used to better judge the extent of the decay. But that tool is slow, takes a lot physical strength by the inspector, is difficult to interpret, and is a destructive testing methods. Increment boring should only be used as a last resort.

1. Non-Destructive Testing (NDT) should now be used on timber structures. There are two methods used: Stress Wave Timbers (SWT) (Chapter 21) can both be used to detect the extent of decay. SWT measures the time that a pulse travels through a timber. The pulse travels much faster in sound, dense wood than in decayed wood, so decay can be detected and its extent can be estimated.

2. Resistance micro-drilling (Chapter 20) pushes a small 1.5 mm bore (with a flared 3mm tip) into the wood and a microprocessor measures the relative resistance. The bore does not drill into or remove wood, but forces its way into the fibers, thus it is a NDT. In most cases, the timber's preservative treatment envelope is not violated. The wood's resistance to the micro-bore is either recorded on paper or is saved to a computer file.

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5.24 UNKNOWN FOUNDATION INVESTIGATION

5.24.1 Introduction

It was estimated that approximately one-fifth of the bridges on the National Bridge Inventory (NBI) have unknown foundations in terms of the type and/or depth. A large number of older non-federal-aid and to a lesser extent federal-aid bridges do not have any design or as-built plans on file to document the type, depth, geometry, or materials incorporated into their foundations. These bridges with unknown foundations pose a potential problem from a scour safety perspective. Since the undermining of bridge foundations poses a risk to the public safety, it is crucial to evaluate all bridges over or near water and determine their susceptibility to scour. In addition to scour concerns, unknown foundations are also a concern when a bridge is considered for improvements.

The evaluation of unknown foundations can be conducted either by conventional methods, such as physically disruptive excavation, coring, or boring methods, as well as less invasive nondestructive evaluation (NDE) methods. Conventional methods are typically considered to be expensive, destructive, and limited in their application. Therefore, research emphasis has recently been placed on NDE methods that can inexpensively and reliably determine the foundation properties.

The items important in the evaluation of these unknown foundations are:

1. Foundation Depth – the bottom elevation of the footing, piles, or combined system.
2. Foundation Type – shallow (footings), deep (piles or shafts), or a combined system.
3. Foundation Geometry – buried substructure dimensions, pile locations, etc.
4. Foundation Materials – steel, timber, concrete, masonry, etc.
5. Foundation Integrity – condition state of foundation materials.

The foundation depth and type, if unknown, are considered to be the most critical pieces of information in a scour evaluation. The foundation geometry, materials, and integrity are frequently desired when improvements are being considered near or to a structure.

In determining which NDE methods might be useful, the ability of the method to detect and to delineate the foundation components from the surrounding environment is often the deciding factor. The subsurface environment typically consists of a mixture of air, water, slope protection materials, soil, and/or rock. Thus, the method needs to consider the wide range of substructure, geological, and hydrological conditions at a particular structure site.

The NDE methods used for unknown foundation investigation can be categorized into either Surface Methods or Borehole Methods. Surface methods are generally less invasive since they do not require soil disruption like borehole methods. Although the following list provides a brief sample of applicable methods, the inspector should be aware that other methods are also currently being researched and implemented.

Currently used Surface Methods include:

1. **Sonic Echo/Impulse Response (SE/IR) Test:** The source and receiver are placed on the top and/or sides of the exposed pile or columnar shaped substructure. The depth of the reflector is calculated using the identified echo time(s) for SE tests, or resonant peaks for IR tests.

2. **Bending Wave Test:** Two horizontal receivers are mounted a few feet apart on one side of an exposed pile and then the pile is impacted horizontally on the opposite side a few feet above the topmost receiver. This method is based on the dispersion characteristics and echoes of bending waves traveling along very slender members like piles.

3. **Ultra-seismic Test (UST):** An exposed substructure is impacted with an impulse hammer to generate and record the travel of compression or flexural waves down and up the substructure at multiple receiver locations.

4. **Spectral Analysis of Surface Waves (SASW) Test:** This involves determining the variation of surface wave velocity verses depth in layered systems. The bottom depths of exposed substructures or footings are indicated by slower velocities of surface wave travel in underlying soils.

5. **Ground Penetrating Radar (GPR):** This method uses a radio frequency signal that is transmitted into the subsurface and records the reflection echoes from the concrete/soil interface to determine the unknown depth.

Currently used Borehole Methods include:

1. **Parallel Seismic Test:** An exposed foundation substructure is impacted either vertically or horizontally with an impulse hammer to generate compression or flexural waves that travel down the foundation and are reflected by the surrounding soil. The reflected compression wave arrival is tracked at regular intervals by either a hydrophone receiver or geophone receiver.

2. **Borehole Radar Test:** A transmitter/receiver radar antenna is used to measure the reflection of radar echoes from the side of the substructure foundation.

3. **Induction Field:** A magnetic field is induced around the steel of the pile or reinforced concrete foundation. The field strength will decrease significantly below the bottoms of the foundation.

5.24.2 Applications

Applications vary depending on the chosen nondestructive evaluation (NDE) method. Refer to Figure 5.24.2-1 for a comparison of the NDE applications.

Method		Applications	Advantages
Surface	Sonic Echo/Impulse Response	Most useful for columnar or tabular structures. Best penetration attained in loose soils. Good for determining thickness and geometry.	Lower cost equipment and inexpensive testing. Data interpretation may be able to be automated. Theoretical modeling should be used to plan field tests.
	Bending Wave	Most useful for a purely columnar substructure. Best penetration attained in loose soils.	Lower cost equipment and inexpensive testing. Theoretical modeling should be used to plan field tests. The horizontal impacts are easy to apply.
	Ultraseismic	Best penetration attained in loose soils. Good for determining thickness and geometry.	Lower cost equipment and inexpensive testing. Can identify the bottom depth of foundation inexpensively for a large class of bridges. Combines compressional and flexural wave reflection tests for complex substructures.
	Spectral Analysis of Surface Wave	Good for determining thickness and geometry.	Lower cost equipment and inexpensive testing. Shows variation of bridge material and subsurface velocities verses depth and thickness of accessible elements.
	Surface Ground Penetrating Radar	Can indicate geometry of inaccessible elements and bedrock depths.	Lower testing costs. Fast testing times.
Borehole	Parallel Seismic	Accurate for determining foundation bottom depths for a large range of structures. Under certain conditions can indicate foundation orientation.	Lower cost equipment and inexpensive testing. Can detect foundation depths for largest class of bridges and subsurface conditions.
	Borehole Radar	Good at determining foundation parameters. Sensitive to detecting steel or steel reinforced members.	Relatively easy to identify reflections from the foundation; however, imaging requires careful processing.
	Induction Field	Highly sensitive to detecting steel or steel reinforced members that are electrically connected to the surface.	Lower cost equipment Easy to test. Compliments Parallel Seismic in determination of pile type.

Figure 5.24.2-1: Comparison of NDE Methods

5.24.3 Limitations

Limitations vary dependent on the chosen nondestructive evaluation (NDE) method. Refer to Figure 5.24.3-1 for a comparison of the NDE limitations.

Method		Limitations
Surface	Sonic Echo/Impulse Response	Response complicated by bridge superstructure elements. Can only detect large defects. Stiff soils and rock limit penetration.
	Bending Wave	Response complicated by various bridge superstructure elements. Response complicated by stiff soils that may show only depth to the stiff soil layer.
	Ultraseismic	Cannot image piles below cap. Difficult to obtain foundation bottom reflections in stiff soils.
	Spectral Analysis of Surface Wave	Cannot image piles below cap. Use restricted to bridges with flat, longer access for testing.
	Surface Ground Penetrating Radar	Higher cost equipment. Signal quality is highly controlled by environmental factors. Adjacent substructure reflections complicate data analysis.
Borehole	Parallel Seismic	Difficult to transmit large amount of seismic energy from pile caps to smaller (area) piles.
	Borehole Radar	Radar response is highly site dependent (very limited response in conductive, clayey, salt-water saturated soils).
	Induction Field	The reinforcement in the columns is required to be electrically connected to the piles underneath the footing. Only applicable to steel or reinforced structures.

Figure 5.24.3-1: Comparison of NDE Limitations.

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5.25 HYDROGRAPHIC SURVEY

5.25.1 Introduction

Hydrographic survey is often known as underwater or sub-bottom profiling, as well as referred to as water depth soundings. This process is used to obtain underwater surface elevation data for evaluating the channel bottom surrounding a structure and the waterway in general. Similar to a topographic land survey, a hydrographic survey consists of a series of elevation measurements over a particular area in a waterway. Refer to Section 1.3.8 in Part 1 for discussions on scour inspections with hydrographic surveys. The level of accuracy in hydrographic surveying varies greatly based on the equipment and methods used. Although the U.S. Army Corps of Engineers have specifications and general requirements for hydrographic surveying and there is even a process to become a certified hydrographic surveyor, scour inspections typically only require the use of more simplified methods performed by an individual familiar with the applications and limitations of water depth measurements near a structure.

The inspector determines channel bottom elevations, and then compares the values to previous data. It is necessary to measure the water level at the time of inspection against a benchmark of known elevation on a pier or other part of the substructure. This location of known elevation should be documented in the report for future use, and may even be permanently marked on the structure. Such benchmarks may be chiseled into the substructure for use in future inspections. A “crow’s foot” is a common marking.

5.25.2 Applications

Applications for hydrographic surveys are primarily for evaluating channel bottom movement. The methods include: lead line, sounding pole, fathometer, and more complex systems.

5.25.2.1 Lead Line

A Lead Line is a simple device typically consisting of a standard surveyor’s tape with a weight attached to the end. The inspector simply lowers the Lead Line until the weight comes to rest on the channel bottom. The inspector then pulls the line taut and records the reading from the channel bottom to the waterline or top of deck. An inspector working from the top of the deck most often obtains Lead Line readings.

5.25.2.2 Sounding Pole

A Sounding Pole is another simple device that typically consists of an extendable, graduated rod. An inspector in the water or in a boat will typically place the pole vertically on the channel bottom and record the measurement at the waterline. The inspector then records the distance from the waterline elevation to a known elevation on the structure. Refer to Figure 5.25.2.2-1 for a view of an inspector using a sounding pole.

Figure 5.25.2.2-1: Inspector Using a Sounding Pole.

5.25.2.3 Fathometer

The most commonly used electronic sounding device is the 200 kHz Fathometer, commonly referred to as a "fish finder." This device uses a transducer just below the waterline and repeatedly transmits sound energy through the water column. The time interval between the transmission of the sound pulse and the returning echo from the channel bottom is used to automatically calculate a depth measurement that is recorded onto the device. Refer to Figure 5.25.2.3-1 for a view of an example of a black and white Fathometer reading.

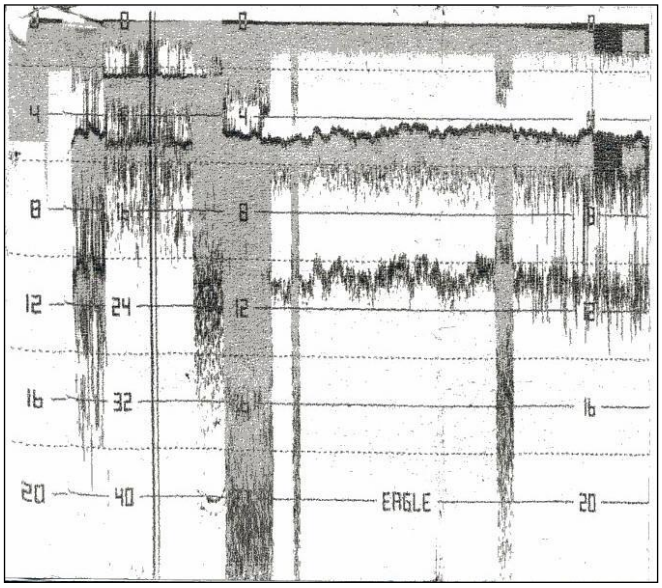

Figure 5.25.2.3-1: Example of a Black and White Fathometer reading.

Fathometers may transmit at a lower frequency (less than 10 kHz) to penetrate up to 10 feet into the channel bottom. The display will indicate materials of different densities as different reflections. This feature can be useful in determining if infilling of previous scour holes by silt or timber debris has occurred. Colors may be assigned to the differing amplitudes of the reflected signals. Fathometers can now color step at 4 dB, meaning small changes in the signal can be observed and displayed on the Fathometer.

5.25.2.4 Complex Systems

Continuous Seismic-reflection Profiling (CSP) systems utilize low frequency sonar that transmit seismic energy from a transducer through the water column and into the channel bottom. They are either fixed-frequency or swept-frequency. The fixed–frequency systems typically use a 3.5-, 7-, or 14-kHz signal, where as the swept-frequency systems typically use a signal that sweeps from 2- to 16-kHz. The swept-frequency CSP system can be used in water as shallow as 1 foot deep, can penetrate up to 200 feet in some silts and clays, and may be able to detect layers as thin as 3 inches. Exposed pier footings, scour depression geometry, and scour depression infill thickness can often be detected with this device.

Ground-Penetrating Radar (GPR) systems radiate short pulses of electromagnetic energy from a broad-bandwidth antenna. These systems typically use a signal of 80-, 100-, or 300-MHz. Dependant on the GPR system used, penetration of up to 40 feet into resistive granular material can be attained and layers as thin as 2 feet can be detected. However, GPR systems will not work in soils or waters that are highly conductive due to chlorides or pollution. Scour depression geometry, scour depression infill thickness, and riverbed deposition can often be detected using this technique.

5.25.3 Limitations

The lead line and sounding pole is limited by the softness of the channel bottom; the swiftness of the current which can introduce horizontal drift into the line or cause a lightly weighted tape to drift downstream; and the time involved in lowering and raising the tape for each new measurement position. This method also may be more prone to inaccuracies based on the experience of the inspector.

Depending on the complexity of the device, fathometers are limited by their ability to detect refilled scour holes; false readings from heavy drift or heavy turbulence; distorted scale on the readout due to varying boat speed; and their inability to provide information about the sub-bottom.

The data collected by a Continuous Seismic-reflection Profiling (CSP) system can be affected by side echoes and by multiple reflections. Side echoes are echoes from the shoreline or piers, and will interfere with the true echo from the channel bottom. Water-bottom multiple reflections occur when the echo is bounced back and forth between the channel bottom and the surface creating multiple readings. These are most evident when the water-bottom reflection coefficient is large, such as in a river with a hard bottom.

Ground-Penetrating Radar (GPR) is typically only useful in shallow (less than 20 feet) fresh water with granular bottom and sub-bottom sediments.

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6.1 INSPECTION PROGRAM OVERVIEW

6.1.1 Introduction

It is important for the safety of the driving public that qualified personnel inspect Wisconsin's Tunnels. The Tunnel Team Leader and other tunnel personnel are required to render judgment on a regular basis pertaining to the safety and integrity of the structures inspected.

The individuals involved in the Wisconsin Tunnel Inspection Program (WisTIP) have critical input on many issues, including the allocation of scarce and limited rehabilitation funds and the decision to close tunnels. It is important that the Tunnel Team Leader and Inspectors are highly trained and proficient; he/she must understand the mechanics, behavior trends, and economics of a wide range of Tunnel types.

The mission of the program is as noted below:

1. Ensure public safety.
2. Provide for the efficient use of resources in maintaining the serviceability of Wisconsin's tunnels.
3. Comply with all federal and state laws, rules, and policies.
4. The State is given the responsibility to accurately inventory, inspect or cause to inspect all highway tunnels on public roads.
5. If a tunnel is not assigned to be inspected by the state inspectors, then the failure of a county or city to perform these responsibilities may cause a loss of Federal funding. The State shall have the authority to take the appropriate action to assure tunnel safety. These assurances will include that the tunnel has been inspected at the proper frequency, that if necessary, the tunnel is posted, and that the posting is done in a timely manner. The State shall have the authority to close unsafe tunnels.

6.1.2 Inspection Program

The Wisconsin Tunnel Inspection Program (WisTIP) is federally mandated and was formalized in 2021. The program policies are based on the National Tunnel Inspection Standards (NTIS), the Specifications for National Tunnel Inventory (SNTI), and the Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual. Tunnel inspection reports and records are stored in the Tunnel Inventory Database.

The National Tunnel Inventory XML file is developed and forwarded to the FHWA on an annual basis.

NTIS define a tunnel as:

> *"an enclosed roadway for motor vehicular traffic with vehicle access limited to portals, regardless of type of structure or method of construction. Tunnels do not include bridges or culverts inspected under the National Bridge Inspection Standards (23 CFR 650- Subpart C- National Bridge Inspection Standards). Tunnels are structures that require, based on owner's determination, special design considerations that may include lighting, ventilation, fire protection systems, and emergency egress capacity."*

This definition is consistent with the definition used by the American Association of State Highway and Transportation Officials (AASHTO). This manual will address all highway tunnels that meet this definition.

6.1.3 **Organization**

The State Program Manager (SPM) is charged with administrating the WisTIP. , Regional Program Manager (RPM), Tunnel Team Leaders (TTL), and Tunnel Team Members (TTM) report to the SPM. The SPM, RPM and TTL are required to be certified Tunnel Inspectors.

The organization of the Tunnel Program is shown in Figure 1.1 and described in detail later in this section.

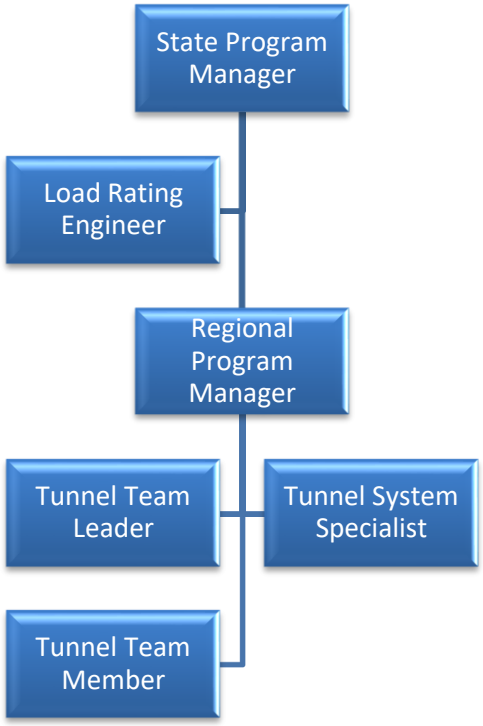

Figure 1.1 Organizational Structure

6.1.4 **Qualifications and Responsibilities**

Below are listed the qualifications required for the tunnel inspection positions and categories. The Qualifications Record Form for Structural Inspection Program (DT2001) must be submitted to the SPM for each of the tunnel inspection positions. Qualifications have been determined for the following tunnel positions:

1) State Program Manager (SPM)
2) Regional Program Manager (RPM)
3) Tunnel Team Leader (TTL)

4) Tunnel Team Member (TTM)
5) Load Rating Engineer (LRE)
6) Tunnel Discipline Specialists (TDS)

For more information, refer to Section 4.4 of the TOMIE manual.

All certified Tunnel Inspectors in the state of Wisconsin must have their credentials on file with the Wisconsin Department of Transportations' Bureau of Structures.

6.1.4.1 State Program Manager (SPM)

The SPM is responsible for setting all tunnel inspection policies and procedures, and for all tunnel inspections and related reporting in the state.

SPM Minimum Qualifications

The SPM must meet the following minimum qualifications:

1. Capable of overseeing the Tunnel Team Leaders (TTL) in Wisconsin, and other assigned Tunnel personnel.
2. Sound background in tunnel inspection and/ or bridge inspection.
3. Specialized knowledge and skills in bridge or tunnel design, construction, soils, construction materials, or emergency repair techniques.
4. Successful completion of Tunnel Safety Inspection (FHWA-NHI-130110) course.
5. Registered Professional Engineer (PE) in the state of Wisconsin.

   OR

   Has a minimum of 10 years of experience in either bridge and/or tunnel inspections.
6. Complete a cumulative of 18 hours of approved tunnel inspection refresher training over each 60 months period.
7. Be a nationally certified tunnel inspector (NCTI).

SPM Responsibilities

As a part of the responsibilities of this position, the SPM shall:

1. Establish statewide policies for NTIS tunnels.
2. Oversee the WisDOT or consultant Tunnel Team Leaders (TTLs).
3. Manage the statewide tunnel inspection and inventory programs.
4. Ensure all tunnels in the state are inspected at a frequency and by a method consistent with the NTIS.
5. Ensure that tunnel inspection data is uploaded to the NTI within mandated time frames.

6. Ensure load ratings are completed in accordance with all federal requirements under the direct supervision of a registered professional engineer.
7. Oversee quality assurance and quality control of all tunnel inspection programs.
8. Coordinate with federal, state, county, and local governmental agencies.
9. Formulate and monitor unique or special features requiring additional attention during inspection to assure the safety of such structures.
10. Notify FHWA of all critical findings based on requirements within this manual within 24 hours of finding.
11. Ensure proper signage is in place for tunnels that require load posting or other restrictions, and ensuring load posting signage is in place no later than 30 days after completion of a valid load rating or notification of missing or improper signage following an inspection.
12. Formulate and administer programs and policies.
13. Develop, implement, and evaluate inspection and preservation policies, standards, procedures, and programs.
14. Lead prompt, decisive, and effective responses to emergencies such as floods, earthquakes, and major tunnel damage.
15. Train tunnel inspection personnel when needed.
16. Develop, monitor, and update training programs for state and consultant inspectors.
17. Arrange or conduct inspection training programs and refresher programs as necessary.
18. Provide training on proper access, equipment operation, and safety procedures.
19. Review and approve all qualifications for RPM, TDS, TTL, LRE, and TTM.
20. Maintain a list of all qualified Tunnel Team Leaders. The list will identify training required to keep the qualifications up to date.
21. Evaluate Tunnel Team Leaders and other tunnel personnel that require additional training as necessary.
22. Maintain their own documentation for national certification.
23. Be able to determine when Tunnel Team Leaders must meet additional requirements for complex or other tunnels.
24. Provide advice on technical issues concerning problems or deficiencies discovered during inspections.
25. Act as a Tunnel Team Leader, as needed.
26. Monitor inspections and develop a good, general knowledge of all tunnels in the state and their inspection records.
27. Manage state tunnel inspection personnel and consultants to meet the needs of the State Tunnel Inspection Program.
28. Manage state-owned tunnel access equipment to assist in the inspection of tunnels statewide.

6.1.4.2 Regional Program Manager (RPM)

The RPM is responsible for assisting the SPM and to help ensure that National Tunnel Inventory (NTI) Data is being collected in a timely manner and stored in the Tunnel Inspection Database properly and reported to the FHWA as required.

RPM Minimum Qualifications

The RPM must meet the following minimum qualifications:

1. Capable of overseeing the Tunnel Team Leaders (TTL) in Wisconsin, and other assigned Tunnel personnel.
2. Sound background in tunnel inspection and/ or bridge inspection.
3. Specialized knowledge and skills in bridge or tunnel design, construction, soils, construction materials, or emergency repair techniques.
4. Successful completion of Tunnel Safety Inspection (FHWA-NHI-130110) course.
5. Registered Professional Engineer (PE) in the state of Wisconsin.
6. Has a minimum of 5 years of experience in either bridge and/or tunnel inspections.
7. Complete a cumulative of 18 hours of approved tunnel inspection refresher training over each 60 months period.
8. Be a nationally certified tunnel inspector (NCTI).

RPM Responsibilities

The RPM serves as the assistant to the SPM. He/she has functions delegated by the SPM and is responsible for carrying out those functions.

6.1.4.3 Tunnel Team Leader (TTL)

The TTL is responsible for the tunnel inspection and reporting for all assigned tunnels. In addition, TTL is responsible for ensuring the tunnel inspection work is filed properly and has ensured that RPM has received such documentation. Consultant TTL may inspect any local or county tunnels when assigned by local or county government.

Tunnel Team Leader (TTL) Minimum Qualifications

The TTL must meet the following qualifications:

1. Successful completion of Tunnel Safety Inspection (FHWA-NHI-130110) course.
2. Be a nationally certified tunnel inspector (NCTI).
3. Complete a cumulative total of 18 hours of FHWA-approved tunnel inspection refresher training over each 60-month period.

4. Capable of overseeing Tunnel Team Members (TTM).
5. Demonstrate a strong background in such areas as bridge or tunnel structural engineering, structural behavior trends and bridge or tunnel rehabilitation techniques.
6. Demonstrate thorough familiarity with NTIS, this manual and applicable WisDOT guidelines.
7. In addition to the above qualifications, all TTL's must meet qualifications in 23 CFR 650, Part E.
8. Good eyesight and the ability to walk and climb over uneven surfaces. In addition, be comfortable working at heights, near water, in confined spaces and close to live traffic.

Tunnel Team Leader (TTL) Responsibilities

As a part of the responsibilities of this position, the TTL shall:

1. Coordinate inspections to ensure that all inspections are completed in compliance with this manual and the TOMIE, including team leader being on-site during the inspection process.
2. Oversee TTMs.
3. Ensure that all tunnel inspection results are approved and sent to the SPM and RPM within 60 days of the date of the inspection and within seven days for all closures and emergency inspections.
4. Supply a copy of the completed inspection report to the Load Rating Engineer (LRE) for review. All defects, deficiencies or inventory changes that could affect the load rating will be communicated to the LRE for review of the effect on the load rating.
5. Notify the SPM of all tunnel critical findings in accordance with this manual.
6. Maintain their own documentation for national certification.

6.1.4.4 Tunnel Team Member (TTM)

The Tunnel Team Member (TTM) is the person responsible for assisting the Tunnel Team Leader (TTL). Preferably, the tunnel inspection team should consist of at least two persons: a TTL and an TTM or two TTL(s). When deemed appropriate by the SPM, or RPM other tunnel specialty personnel may be required.

Tunnel Team Member (TTM) Minimum Qualifications

The TTM must meet the following qualifications:

1. Be responsible in assisting in the field work and be on site during the inspection.
2. Demonstrate a strong background in such areas as bridge or tunnel structural engineering, structural behavior trends, and bridge or tunnel rehabilitation techniques.
3. Good eyesight, the ability to walk and climb over uneven surfaces, and the ability to work at heights, near water, in confined spaces, and close to live traffic.

4. Recommended that TTM are registered and nationally certified tunnel Inspectors; however, this is not a mandatory requirement.

Tunnel Team Member Responsibilities

The Tunnel Team Member is responsible for the following:

1. Following all Tunnel Team Leader (TTL) instructions in a safe and appropriate manner.
2. Assisting the TTL in the field.
3. Documenting his/her participation and experience.

6.1.4.5 Load Rating Engineer (LRE)

Routine load ratings of state-owned tunnels are generally performed and maintained by WISDOT's Load Rating Engineer in the Bureau of Structures – Development Section.

Locally owned structures shall provide their own load rating engineer, when necessary, to perform calculations related to capacity.

LRE Minimum Qualifications

The LRE must meet the qualifications listed below:

1. Have experience calculating load ratings for bridge structures and tunnels and knowledge of load capacity rating computer programs.
2. Registered PE licensed in the state of Wisconsin, qualified to oversee, review and certify all load capacity ratings performed under his/her supervision.

LRE Responsibilities

The LRE must:

1. Provide engineering judgment to those performing the load ratings.
2. Be actively involved in reviewing the quality and accuracy of all tunnel load ratings.

6.1.4.6 Tunnel Discipline Specialists/Specialty Contractor (TDS)

Tunnels may include several functional systems (mechanical, electrical, lighting, fire detection, etc.) that require personnel with specialized skills to inspect, test and maintain.

The specific qualifications required for tunnel system specialists will vary based upon the type of functional system and should be detailed in the tunnel-specific inspection procedures documentation. It should be noted that these personnel require very specific certifications to perform their inspection work: For more information, refer to Sections 4.4.3 and 4.4.4 of the TOMIE manual. These personnel are not required to be nationally certified tunnel inspectors.

TDS Minimum Qualifications

When complex civil/structural, mechanical, or electrical systems need to be inspected, the tunnel team leader should assign tunnel discipline specialists/specialty contrator (TDS) with suitable training, qualifications, and experience to help conduct these inspections and these criteria should be addressed in the tunnel specific procedures.

TDS are beneficial when the regular inspection staff lacks the specialized skills and experience necessary to inspect sophisticated equipment or complex systems such as power distribution systems, fire protection and detection systems, and security systems. It is advisable to use qualified specialty contractors when inspecting complex units that pose elevated risks to safety such as boiler units, electrical systems, or energized equipment like transformers. This may help to minimize health and safety risks to the inspection crew and prevent damage to very expensive equipment.

TDS Responsibilities

The TDS are responsible for the following:

1. Following all Tunnel Team Leader (TTL) instructions in a safe and appropriate manner.
2. Assisting the TTL in the field.
3. Documenting his/her participation and experience.
4. Adhere to the assigned TDS specific duties in Sections 4.4.3 and 4.4.4 of the TOMIE manual.

6.1.5 Tunnel Inventory Database

As the Lead Agency in interactions with FHWA, it is WISDOT's responsibility to maintain an inventory of all tunnels within Wisconsin. All tunnel data shall be electronically stored on the Department online file storage system in a location designated by the SPM.

Data Included in Database:

1. Inventory Information required by the NTIS and SNTI (incorporated by reference)
2. Field inspection information including sketches and photographs showing typical and deteriorated conditions, entered within 3 months after completion of work.
3. Data for tunnel modifications and new tunnels entered within 3 months after completion of work.
4. Data for tunnel load restrictions and closures entered within 3 months after completion of work.
5. Tunnel Files.
6. Critical Findings in accordance with this manual
7. Tunnel Specific Inspection Procedures

8. Load Rating Information
9. Posting Documentation
10. Inspection reports including conditions of each structural and functional system component
11. Maintenance and repair records
12. Photos, diagrams, and sketches.
13. Functional System Testing Logs.
14. Plans and as-builts.

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6.1.1 Introduction

There are numerous types of inspections, each designed to obtain specific information. For example, an Initial Inspection is performed after a tunnel is constructed to document the as-built conditions, whereas Routine Inspections are used to monitor the condition of a tunnel at regular intervals. Damage Inspections are used to assess damage resulting from events such as deterioration, impacts, fires, or floods. These inspections help create a complete picture of a tunnel's condition and are described in detail in this chapter.

Visual inspection is the primary examination method for all inspections. Nondestructive testing (NDT) techniques may be required to identify internal flaws or hard-to-see external defects in critical members.

6.1.2 Inspection Procedures

Section 650.513(a) of the NTIS requires that tunnel structural elements and functional systems be inspected in accordance with the inspection guidance provided in the TOMIE manual.

Section 650.513(c) of the NTIS requires that tunnel-specific inspection procedures be prepared and documented for each NTIS tunnel. These inspection procedures shall consider the design assumptions and the tunnel complexity. Tunnel-specific inspection procedures must be reviewed and updated by the inspection team leader upon completion of each inspection report. Tunnel-specific inspection procedures shall be developed for each tunnel and stored in the WisDOT NTI file system.

Pre-inspection coordination is required for all tunnel inspection types. This coordination will be led by the inspection team leader. The pre-inspection coordination will be detailed in the tunnel-specific inspection procedures. Pre-inspection coordination must begin well in advance of the inspection and should be defined in the tunnel-specific inspection procedures. At a minimum, the pre-inspection coordination will include the following:

- Coordination with stakeholders regarding anticipate inspection or system testing conflicts with operations (ie tunnel, airport or court house operations). Review operations that may interfere with inspection or testing. Determine solutions for resolving identified conflicts.
- Contact and coordinate with fire department and maintenance department for required testing or request test results. Test results should be available to the inspection team prior to or during the inspection.
- Required traffic control approvals and permits with facility owners for vehicular and pedestrian traffic.
- Coordination with facility owners for required access to mechanical rooms, electrical panels, area above the tunnel, etc.
- Contacts and notification to police, fire, EMS, schools, or others significantly affected by inspection operations.

6.1.3 Inspection Types

The NTIS defines five types of tunnel inspections:

1) Initial Inspection
2) Routine Inspection
3) Damage Inspection
4) In-Depth Inspection
5) Special Inspection

All inventory and inspection data shall be entered into the Tunnel Inventory Database within 90 days of the completion of the inspection (or construction of new tunnels) per NTIS requirements.

6.1.3.1 Initial Inspections

6.1.3.1.1 Purpose

An Initial Inspection is the baseline inspection that should be completed on every new tunnel, after a major rehabilitation, or when the configuration or geometry of a tunnel changes significantly (e.g., when a tunnel is widened). An Initial Inspection is a fully documented inspection using the plans to determine basic data which will be stored in the Tunnel Inventory Database. Initial Inspections are also used when a tunnel is discovered that has not been previously inventoried. In this case, the tunnel plans may not be available. As part of the Initial Inspection, inspectors evaluate the tunnel and decide what other foreseeable inspections will be required throughout its life, which include Special, and In-depth Inspection.

As a part of the Initial Inspection, Inspectors must review the “Tunnel File” and research and look for all missing documents and have any that are found scanned and uploaded into the Tunnel Inventory Database.

6.1.3.1.2 Precision

The Initial Inspection should be a fully documented investigation. Inspectors must be able to identify any deficiencies and verify the geometric data. All observed deficiencies, cracks, construction errors, and alignment problems should be documented. Hands-on inspection of components shall be used, when possible.

At least one NTIS Team Leader must be present during an initial inspection.

An Initial Inspection should be performed in accordance with the inspection guidance provided in the TOMIE manual and should include:

1. Verify and record all tunnel inventory data required by federal and state regulations.

2. Gather relevant information required to maintain an accurate Tunnel file, scan and uploaded in the Tunnel Inventory Database.

3. Ensure that all tunnel systems are noted and evaluated in accordance with the TOMIE which may include the following:

    - Emergency generator system
    - Miscellaneous mechanical systems
    - Electrical distribution systems
    - Electrical and lighting systems
    - Fire and life safety, emergency systems
    - Emergency communication systems
    - Tunnel security and operation systems
    - Tunnel ventilation systems
    - Other tunnel systems that need special consideration

4. Determine and evaluate the baseline structural condition, as well as systems.

5. Identify the location and condition of any details that may require a Special Inspection.

6. Verify that all clearances and geometric dimensions are noted in the Tunnel Inventory Database.

7. Identify any critical findings and notify the appropriate individuals and agencies identified in this manual.

All inspection results should be fully documented and filed in the Tunnel Inventory Database.

6.1.3.1.3 Frequency

The Initial Inspection shall be completed before the new construction or rehabilitation construction contract is finalized and the tunnel is open to traffic.

6.1.3.2 Routine Inspection

6.1.3.2.1 Purpose

Routine Inspections are regularly scheduled inspections consisting of observations and/or measurements needed to determine the physical and functional condition of the tunnel, and to identify any changes from previously recorded conditions. The Routine Inspection also ensures that the tunnel continues to satisfy present service requirements.

The NTIS requires that routine tunnel inspection be performed in accordance with the TOMIE Manual. Tunnel specific inspection procedures are required for each NTIS tunnel inspection. At least one NTIS Team Leader must be present during a routine inspection.

6.1.3.2.2 Precision

Routine Inspections will follow the inspection procedures document for the structure, as documented in the Tunnel Inventory Database. A specific inspection procedure for each tunnel must be documented in the database.

Routine Inspections are generally conducted from the portal to portal or from permanent work platforms and walkways, if present. A complete visual inspection of all components of the structure and adjacent roadway is required. A complete visual inspection of all tunnel systems is required, such as:

A Routine Inspection should include the following system inspections in accordance with the TOMIE Manual, if present:

- Emergency generator system
- Miscellaneous mechanical systems
- Electrical distribution systems
- Electrical and lighting systems
- Fire and life safety, emergency systems
- Emergency communication systems
- Tunnel security and operation systems
- Tunnel ventilation systems
- Other tunnel systems that need special consideration, including inspection of structural supports/anchorages of any systems directly over traffic.

1. Complete an inspection and evaluation of all required data identified in accordance with this manual.
2. Verify tunnel inventory data.
3. Gather other relevant information required to maintain an accurate tunnel file.
4. Note any existing problems or components.
5. Identify the location and condition of details that may require a Special Inspection.
6. Required Photos:
    - Alignment photos from both ends of the tunnel.
    - Photos looking at the portal entrance (north or east entrance) and exit (south or west entrance).
    - Closing, posting, and/or restriction signs should be visible and legible in the photos.
    - Elevation photos, preferably of both ends of the tunnel, (as a minimum on one side of the tunnel).
    - Photos of all tunnel elements with a condition state of 3 or 4.
    - Photos under each superstructure type, clearly showing details.
    - Photo of any detail that requires a Special Inspection.
    - Photos of significant deterioration and collision damage.
7. Note if a new load rating is warranted.
8. Identify any Critical Findings and notify the appropriate individuals and agencies identified in this manual.
9. Provide maintenance recommendations to tunnel owner.

6.1.3.2.3 Inspection Frequency

The NTIS requires routine tunnel inspections to be conducted on a 24-month interval. Certain deterioration may require more frequent inspections; If only a portion of a tunnel needs more frequent inspections, a Special Inspection is required.

For more information, refer to Chapters 4 and 5 of the TOMIE manual and the SNTI.

6.1.3.3 Special Inspections

6.1.3.3.1 Purpose

As defined by the NTIS, a "special inspection" is an inspection scheduled at the discretion of the tunnel owner. A special inspection is typically performed after an initial, routine, damage or in-depth inspection when significant deficiencies have been discovered and need to be monitored. Special inspections might continue until the deficiency is repaired or further inspections are no longer deemed necessary. The scope, frequency, and duration of special inspections are up to the discretion of the tunnel owner. The qualifications of personnel performing a special inspection is at the discretion of the tunnel owner.

6.1.3.3.2 Precision

Special Inspections may include a special inspection procedure, if required. The qualifications of personnel performing a special inspection is at the discretion of the tunnel owner.

6.1.3.3.3 Frequency

The scope, frequency, and duration of special inspections are up to the discretion of the tunnel owner. The qualifications of personnel performing a special inspection is at the discretion of the tunnel owner.

6.1.3.4 In-Depth Inspection

6.1.3.4.1 Purpose

An "in-depth inspection" is a close-up inspection of one, several, or all tunnel structural elements or functional systems to identify any deficiencies not readily detectable using routine inspection procedures. In-depth inspections may use non-destructive testing or other advanced methods. In-depth inspections may involve testing of tunnel system, components, and materials. Access equipment may be necessary to facilitate hands-on inspection at some locations.

6.1.3.4.2 Precision

The scope of an In-Depth Inspection should be dependent on the purpose of the inspection and shall be coordinated by the tunnel owner, TTL, RPM, and the SPM. This is a relatively infrequent inspection scheduled for structures that typically do not require a scheduled inspection beyond the routine inspection. This inspection will give the inspector the opportunity to make sure that all the components of the structure are performing as intended.

At least one NTIS Team Leader must be present during an in-depth inspection.

6.1.3.4.3 Frequency

In-depth inspections are scheduled based on the needs of the tunnel facility, inspection findings, and established written procedures. In-depth inspections may occur more frequently or less frequently than routine inspections, as outlined in the tunnel-specific inspection procedures.

6.1.3.5 Damage Inspections

6.1.3.5.1 Purpose

A Damage Inspection is an unscheduled inspection to assess structural damage resulting from environmental factors or human actions. Flood damage, fire damage and vehicle impact are examples of events that may call for a Damage Inspection.

6.1.3.5.2 Precision

The scope of a Damage Inspection should be sufficient to determine whether there is a need for emergency load restriction, or closure of part or all of the tunnel to traffic. The amount of effort expended on this type of inspection may vary significantly and depends on the extent of the damage. If major damage has occurred, the inspector shall document the damage, including measuring section loss or misalignment, and any loss of foundation support.

This inspection may be supplemented by a timely Special Inspection to more fully document the extent of damage and the urgency and scope of repairs. A more refined analysis, to establish or adjust interim load restrictions, may also be required as follow-up for a Damage Inspection. A structural engineer may need to be consulted for the inspection or analysis. If the inspection identifies a Critical Finding, the inspector must follow the notification procedures outlined in Chapter 4 of this manual.

A damage inspection is required for all tunnels in which the event has left permanent physical evidence. The damage inspection data and pictures shall be entered into the Tunnel Inventory Database as soon as possible and no more than seven (7) days after the inspection.

The Inspector of state-owned or state-assigned tunnels should gather data on the vehicles and drivers involved and any police report after a crash. This information will be used to bill the appropriate insurance company for damages.

A damage inspection of an NTIS tunnel should be performed by an NTIS Team Leader, NBIS Team Leader, or a Professional Engineer.

6.1.3.5.3 Frequency

A Damage Inspection is an unscheduled inspection that is performed to determine if significant damage has been done to the tunnel. Based on the findings of the damage inspection, the inspector will determine if the damage warrants placing the structure on a special inspection. Pictures of any damage will be uploaded into Tunnel Inventory Database with a complete description of the event. Generally, a law enforcement officer on the site of an accident involving a tunnel will notify the owner who will request a Damage Inspection be performed to determine if the tunnel should be closed.

6.1.4 Tunnels Closed To Traffic

If a tunnel is closed to all traffic for construction when an inspection is due, the inspection team shall:

1. Document the tunnel is properly closed with photos.
2. Note that the inspection date was changed in the Tunnel Inventory Database.
3. Verify the estimated date of completion of the construction.
4. Schedule a new Initial Inspection and all other required inspections for the estimated completion date. The initial inspection shall be completed prior to opening to traffic.
5. All rescheduled Routine inspections should be completed ± two months of the original inspection date. Routine Inspections should remain in the month that it had been prior to construction. If the construction schedule will not allow for this, the initial inspection shall be a full Routine inspection.
6. Leave other NTI data items unchanged, until the initial inspection is conducted.

If a tunnel is only partially closed to traffic during construction, then a tunnel inspection is required. On WisDOT tunnels, the contractor is required to provide a time and access for inspectors to conduct all needed inspections.

If a tunnel has been closed permanently when inspection is due, the inspection team shall:

1. Document the tunnel is properly closed with photos. No other inspection work is required. If the tunnel is not properly closed, a critical finding must be immediately submitted.
2. A memorandum will be published stating that the tunnel is permanently closed. The memorandum will be electronically stored on the Tunnel Inventory Database.
3. Code the appropriate NTI Date Item(s) with the date the inspectors were at the tunnel.
4. Note that the inspection dates were changed in the Tunnel Inventory Database.
5. Leave other NTI data items unchanged.
6. Recommend the removal of the tunnel be scheduled as soon as possible.

Permanently closed tunnels are generally not eligible to use federal tunnel inspection funds to conduct inspection activities.

6.1.5 Complex Tunnels

The NTIS defines a “complex tunnel" as a tunnel characterized by advanced or unique structural elements or functional systems.

WisDOT has defined a “complex tunnel” as follows:

> *A “complex tunnel" is a tunnel with advanced or unique structural elements or functional systems. Specific examples include tunnels passing below waterways, tunnels with multiple traffic levels, tunnels with ventilation systems that must operate continuously to ventilate exhaust, or any tunnel designated as “complex” by the Wisconsin Program Manager.*

Based upon this definition, WisDOT does not currently consider any of the NTIS tunnels to be “complex”.

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6.3.1 Tunnel File

The Tunnel file is the collective term for all documents necessary to provide a comprehensive history of each Tunnel Asset. At a minimum, the Tunnel file is to contain the following documents prior to being identified as complete. Each item is annotated with the required repository.

- Tunnel Inspection Reports
- Original Plans (Either Approved Design signed by PE or As-Built Record)
- All Tunnel Rehabilitation Plans (Either Approved Design signed by PE or As-Built Record)
- Significant Correspondence. Significant correspondence includes agreements regarding inspection responsibility, ownership, operations, or other issues that have an impact on timely inspections.
- Memoranda of Agreement (including Maintenance Agreements), where applicable
- Load Rating Reports and Load Rating Calculations/Models
- Functional System Testing Logs
- Maintenance Reports
- Inspection Procedures
- Posted Speed Limits within the Tunnels (summarized in an Excel file)

The required report sections in a Tunnel inspection report include the following:

1. Report Cover and Location Map
2. Executive Summary. The executive summary is to include a general statement of condition of the Tunnel and a statement of areas of concern.
3. National Tunnel Inventory and Miscellaneous Asset Data, including inspection procedures per TOMIE.
4. Field Inspection Information. Field inspection information is to include sketches and photographs showing typical and deteriorated conditions. A brief narrative is required to justify a change in condition rating. All pictures in the inspection report are to be labeled.
5. Critical Findings. Critical finding documentation is to be in accordance with this Tunnel Inspection Manual.
6. Load Rating and Posting Documentation.
7. Photos and Sketches. All pictures and sketches in the inspection report are to be labeled.
8. Inspection procedures utilized for the inspection.

9. Maintenance recommendations for the tunnel owner.

The WisDOT Tunnel file is electronically stored on the Department online file storage system in a location designated by the SPM

6.3.2 Tunnel Numbers

The state uses an alpha-numeric numbering system to identify the Wisconsin Department of Transportation (WISDOT) Tunnel identification number. Since Wisconsin has very few tunnels compared to bridges and culverts, the numbering system is based on:

a. Structure Type: T for Tunnel

b. County number: 40 for Milwaukee

c. Structure number: 4-digit number after county

d. Unit number: Used if multiple units exist on a structure

The following five tunnels are currently in Wisconsin:

- T400292 South Kilbourn Tunnel
  - Unit 1 State Owned (West Section)
  - Unit 2 City Owned (East Section)
- T400293 North Kilbourn Tunnel
  - Unit 1 State Owned (West Section)
  - Unit 2 City Owned (East Section)
- T400744 College Avenue Tunnel Airport (County) Owned
- T400282 Howell Avenue Tunnel Airport (County) Owned
- T400001 9th Street Tunnel City Owned

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6.4.1 Introduction

A Critical Finding (CF) is defined as a structural or safety related deficiency that requires immediate action to ensure public safety.

6.1.1 6.4.2 Critical Findings

A CF is a structural or safety related deficiency that requires immediate follow-up inspection or action.

Any of the following will be considered a CF:

- Structural conditions that, if not promptly corrected, could result in a collapse or partial collapse (or partial failure) of the tunnel. This also includes any structural condition that warrants immediate closure of the tunnel.
- Non-structural safety hazards that pose an immediate threat to the traveling public, including but not limited to deteriorated non-structural elements located above the roadway.

6.4.3 Procedures for Tunnel Team Leaders (TTL)

When a CF is identified, the safety of the traveling public must be the initial focus.

The immediate actions taken by the TTL will vary with the circumstance. Based on the conditions present, a conservative decision to limit traffic or close the tunnel should be considered until further analysis can be performed.

The CF must be submitted to the State Program Manager (SPM) and FHWA within 24 hours of discovery. If notification is via phone or verbal, then inspector shall follow up with an email notification. That notification shall be sent to the SPM, RPM, and ASPM. The email shall include all the pertinent data including pictures, sketches, and other related files in addition to the DT2026 Critical Findings Report.

The TTL will make a preliminary determination to identify the CF as Urgent or Severe per the definitions below:

An **Urgent** CF is where a deficiency is discovered which threatens the integrity of the structure. The Tunnel is closed immediately until the deficiency can be investigated. The deficiency may require tunnel replacement or major rehabilitation to put the structure back in service.

A **Severe** CF is one where either a structural deficiency or a non-structural safety hazard is discovered that necessitates immediate lane restrictions or partial closures of the structure until repairs are performed.

The SPM will record the CF for tracking in WisDOT's CF Log and will notify the FHWA within 24 hours of the incident. The owner and SPM will ensure that immediate action is taken on the closures.

Once closures are in place, a plan of action (POA) to address the CF will be developed by the owner and submitted to the SPM no later than seven (7) days after the discovery of the finding.

The SPM will review the POA and request updates, as appropriate, from the owner until the POA is completed and the CF is closed out.

6.4.4 Documentation

The Critical Finding will become a permanent record in the Tunnel file. The Critical Findings Report (DT2026) shall be used to document the incident. This form shall be included in the Tunnel file, along with all pertinent photographs, sketches, calculations, and other documents necessary.

For quarterly and annual reporting requirements to FHWA, the tunnel critical findings will be logged in the same spreadsheet as is used by WisDOT for bridge critical findings. This spreadsheet will be forwarded to FHWA on request, or at specified intervals.

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6.5.1 Introduction

Quality in the inspection and the resulting documentation is an important aspect that must be considered to ensure the safety of the traveling public in tunnels. The NTIS definitions for QC and QA are as follows:

> **Quality control (QC).** The term "quality control (QC)" means the procedures that are intended to maintain the quality of a tunnel inspection and load rating at or above a specified level.
>
> **Quality assurance (QA).** The term "quality assurance (QA)" means the use of sampling and other measures to ensure the adequacy of quality control procedures in order to verify or measure the quality of the entire tunnel inspection and load rating program.

Both QC and QA must be performed to ensure that the inspections conducted, and deliverables produced meet the tunnel owner's quality requirements, while ultimately ensuring the public's safety.

6.5.2 Quality Control

Consultants or local inspectors performing inspections on highway tunnels in Wisconsin are required to submit a quality control plan to the RPM for review.

The responsibilities include but are not limited to the review of the inspection reports and review of the inspection methods.

At least one inspection will be reviewed by a qualified PM (SPM, ASPM, or RPM) each cycle. All tunnels in the inventory will have an inspection reviewed at least once every 10-years.

The PM review of inspection reports will consist of the following:

1. Overall review of the inspection report to ensure that the report format per Part 6 Chapter 3 has been used, that the correct tunnel is identified and that all required information per the NTIS and Part 6 of the WisDOT Structures Inspection Manual has been entered.
2. Review that all information has been correctly entered in accordance with the SNTI and this manual. This review will include but not be limited to a check that proper coding conventions, format, significant digits and correct units have been used.
3. Check that the overall quantity of condition states is consistent with the condition states of the individual element deficiencies.
4. Check that there is adequate documentation for elements that have deficiencies in condition states 2, 3 & 4.

5. Check that all sketches, charts and/or photographs have been properly cross referenced in the inspection report.
6. Check that there is consistency of information between the current inspection report and previous inspection reports and/or rating report, if applicable.
7. Review functional system testing logs, as necessary.
8. Check that proper documentation was incorporated into the inspection report for any changes that may have occurred from the previous inspection report.
9. Check that inspection procedures were appropriate and were utilized by the inspection team
10. Review maintenance recommendations
11. Review of all items after data entry to check that they have been properly and correctly entered.
12. For every Initial Inspection, a check of the inventory data against the construction plans to ensure that the data is consistent.
13. For every Initial Inspection, a set of inventory photos has been taken and included in the report and saved in the Tunnel Inventory Database.
14. Load ratings, if applicable, will be reviewed by LRE

6.5.3 Quality Assurance

The review will include checking that the inspection data complies with the Federal and WISDOT requirements.

The QA review is responsible for ensuring that the defined quality control procedures are enforced throughout the state. A review includes all aspects of functions to ensure adherence to Federal and State inspection criteria, laws, codes, standards, and regulatory requirements.

The SPM, ASPM, and RPM are responsible for performing quality assurance reviews for NTIS tunnels. The QA review will be performed one of the PM's who was not part of the inspection team.

At the direction of the SPM, one of the PM's will field review at least one NTIS inspection report each inspection cycle, checking for the accuracy of element condition ratings and documentation. This review will be documented on the Structures Inspection Quality Assurance Form (DT2003).

The tunnel-specific inspection procedures and functional system testing procedures will also be reviewed for accuracy and to what level the TL followed the procedures. Also, the QA should determine if functional system tests are being adequately documented.

All tunnels in the inventory are required to be reviewed at least once in a 10-year period.

6.5.4 Inspector Disqualification/Requalification Procedures

Inspectors that fail to uphold the requirements for a quality inspector as established by WisDOT may be subject to the disqualification/requalification process.

Procedures are discussed in Part 1 Chapter 2 of the SIM.