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# TRANSMITTAL LETTER

PUBLICATION:

Publication 238

DATE:

6/27/2024

SUBJECT:

Bridge Safety Inspection Manual
Publication 238, 2024 Edition

INFORMATION AND SPECIAL INSTRUCTIONS:

The attached 2024 Edition of the Bridge Safety Inspection Manual represents a revised publication incorporating the Load Rating Best Practices Manual as Appendix IP 03-E, previously issued Strike-Off Letters, editorial changes, and changes from clearance transmittal comments. This transmittal is a follow-up to Clearance Transmittals B-24-001 (Load Rating Best Practices Manual) and B-24-002 (Publication 238).

This edition is effective immediately. This release will have a time neutral effect on bridge safety inspections.

Comments or questions concerning this manual may be directed to Jonathan Moses, P.E., Assistant Chief Bridge Engineer - Inspection, at 412-429-4897 or by emailing the Bridge Inspection Section at PD-BridgeInspectSection@pa.gov.

CANCEL AND DESTROY THE FOLLOWING:

This 2024 Edition supersedes the 2022 Edition.

The new edition is not available in hard copy. Electronic format is available through the Department's website under Forms & Publications as well as on the BMS2 homepage (Forms and Templates link).

ADDITIONAL COPIES ARE AVAILABLE FROM:

☒ PennDOT website - www.penndot.pa.gov
*Click on Forms, Publications & Maps*

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# BRIDGE SAFETY INSPECTION MANUAL


Pennsylvania
Department of Transportation
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PUB 238 (12-24)

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## Part IP: Policy and Procedures and Part IE: Evaluation Specifications

References to the sections, equations, figures or tables contained in the following bridge manuals are designated by the following prefixes:

| | |
|---|---|
| M | AASHTO Manual for Bridge Evaluation (MBE) |
| MC | Commentary to AASHTO Manual for Bridge Evaluation |
| IP | PennDOT Bridge Safety Inspection Manual, Pub 238, Part A "Policies and Procedures" |
| IE | PennDOT Bridge Safety Inspection Manual, Pub 238, Part B "Evaluation Specifications" |
| PP | Pub 15M - PennDOT Design Manual, Part 4, Part A "Policies and Procedures" |
| PD | Pub 15M - PennDOT Design Manual, Part 4, Part B "Design Specifications" |
| AD | AASHTO LRFD Bridge Design Specifications |
| SD | AASHTO Standard Specifications for Highway Bridges |

In Publication 238, tables and figures are numbered consecutively within their chapter using the above prefixes. Examples of this identification format are:

| | |
|---|---|
| Table IP 1.1-1 | The first table in Chapter 1, Section 1 of the Pub 238 Policies and Procedures |
| Figure IE 3.2-4 | The 4th figure in Chapter 3, Section 2 of the Pub 238 Evaluation Specifications |

Tables and figures from the other manuals above will be identified by the above prefix and the number assigned by that manual. An example is:

Figure SD 3.7.6 Standard H Trucks (from AASHTO Standard Design Specs.)

Separate Appendices were established for each Chapter. Separate documents within each Chapter are numbered sequentially using the above prefixes. Examples are:

| | |
|---|---|
| Appendix IP-02-A | 1st Appendix supporting Chapter 2 of Publication 238 Policies & Procedures |
| Appendix IP-08-C | 3rd Appendix supporting Chapter 8 of Publication 238 Policies & Procedures |

## Part IE: Evaluation Specifications

If a section is listed in the Table of Contents, it is modifying, adding to, or replacing the corresponding section of the MBE by this Part IE.

If a section or article has a suffix I, this is a new section not included in the MBE.

When Publication 238 modifies and/or adds information to an article from the MBE, the first sentence of Publication 238 shall read "*The following shall supplement Mx.x.x*."

When a Publication 238 article replaces information in an article from the MBE, the first sentence of Publication 238 shall read "*The following shall replace Mx.x.x*."

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| Strike-Off-Letter | Subject | Section Where Incorporated |
|---|---|---|
| 483-22-01 | Technical Bulletin – Bridge Safety Inspection and Bridge Maintenance Programs | Numerous |
| 483-22-02 | Publication 238 – Bridge Safety Inspection Manual Revision to Bridge Inspection Agreement Funding Categories | Section IP 1.11 |
| 483-24-01 | Bridge Inspection Training Requirements Publication 238, Bridge Safety Inspection Manual | IP Chapter 2 and IP Chapter 7 |
| 483-24-02 | Uncoated Weathering Steel Publication 238, Bridge Safety Inspection Manual | IP Chapter 2, IE Chapter 4, and Appendix IP 02-I |
| 483-24-04 | Load Rating Best Practices Manual (LRBPM) Publication 238, Bridge Safety Inspection Manual, Appendix IP 03-E | Appendix IP 03-E |

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| Section No. | Description of Revision |
|---|---|
| General | Revised cover and headers to 2024 Edition. |
| General | Revised “iForms” to “BMS3”. |
| Glossary | Revised to include applicable definitions from the NBIS and remove those no longer applicable. |
| TOC | Revised Sections and page numbers as needed, added Appendix IP 06-B. |
| IP 1.3.2 | Revised AASHTO MBE to be Third Edition, 2018, up to and including 2020 Interim Revisions in accordance with CFR 650.317. |
| IP 1.10.4.1 | Revised timeframe to update BMS2 for local bridges to 90 days to align with NBIS. |
| IP 1.12.2.3 | Added requirement to close traffic lanes below crane bucket during inspection operations. |
| IP 2.1.3.1 | Revised qualifications for Program Manager per SOL 483-24-01. |
| IP 2.1.3.2 | Revised qualifications for Bridge/Culvert Safety Inspectors per SOL 483-24-01 and added list of BMS2 certification records for Bridge Team Leader, NSTM Team Leader and Underwater Diver. |
| IP 2.1.3.3 | Added list of BMS2 certification records for Tunnel Team Leader. |
| IP 2.2.2 | Added that new rating must be approved in BMS2 within 60 days of the inspection end date. |
| IP 2.2.10 | Revised IP 2.2.10.1 to reference Uncoated Weathering Steel – Bridge Safety Inspection and Maintenance Manual and revised IP 2.2.10.2 for UWS Subject Matter Experts. |
| IP 2.3 | Revised 3rd paragraph to clarify inspections requiring a Team Leader and added list of complex bridge features. |
| IP 2.3.6.3 | Revised 3rd paragraph to state the F&F plan “must” instead of “should”. |
| IP 2.3.6.4 | Revised first NSTM inspection to occur within 3 months of the bridge opening to traffic. |
| IP 2.13.4.2 | Added that there is no acceptable tolerance for intervals of 6 months and less for the next inspection, these inspections must be completed in the month they are due. |
| IP 2.4.4 | Revised section to change “FCM” to “NSTM” where applicable and to align with Pub 100A. |
| IP 2.4.4.4 | Added reference to BMS2 Item IF09 in Pub. 100A for NSTM detail guidance. |
| IP 2.4.5.1 | Clarified NSTM hands-on policy and added fatigue category and inspection procedure requirements for F&F plans. |
| IP 2.4.5.4 | Added reference to BMS2 Item IF09 in Pub. 100A for NSTM detail guidance and revised BMS2 Item references for Required Inspection and Inspection Interval. |
| IP 2.7.1 | Added criteria for closing a bridge and revised 2nd paragraph to state “Closed bridges greater than or equal to eight feet (8’) in length owned by the Department or carrying State Routes must be inspected in conformance with those same standards. |
| IP 2.13 | Added tunnel maintenance categories (previously IP 2.13.3) to Structure maintenance Needs. |
| IP 2.13.3 | Renamed Section “Critical Findings”. Incorporated previous IP 2.13.3.1 critical findings for tunnels and added relevant bridge information. Added critical findings per CFR 650.313(q)(1)(i) and required notifications to Central Office Bridge Inspection Section and FHWA per CFR 650.313(q)(1)&(2). |
| IP 2.13.4 | Added Section 2.13.4, Uncoated Weathering Steel Bridge Maintenance. |
| IP 3.2.2.2 | Revised language related to posting of H20 vehicle. |
| IP 3.2.2.5 | Added language to contact Bridge Inspection Section for EV2 and EV3 load rating of EJ bridges, CONSPAN bridges, and similar structures if located within reasonable access to Interstate System. |
| IP 3.3.1 | Clarified application of reduction in live load intensity for lever rule. |
| IP 3.3.5 | Removed channeling devices as a temporary measure. |
| IP 3.6.1.1 | Added type of bridge, year built, and section loss location as engineering judgement considerations. |
| IP 3.9 | Added statement that sealing the load rating indicates that the rating has been reviewed and items on the Load Rating Quality Control Verification Checklist are satisfied. |
| IP 4.3.2 | Added that SLC for EV2 and EV3 vehicles should not be less than 1.0 and added guidance for choosing Superstructure SLC when the Substructure is controlling the SLC. |
| Table IP 4.3.2-1 | Added “Culvert” to table headings. |
| IP 6.1 | Added reference to MBE Section 1.4. |

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| IP 6.2.2 | Added that QC reviews are to be performed by personnel other than the individual who completed the original report or calculations. Added that ADBE Required Actions include review and acceptance of bridges with condition rating of 5 or greater by a CBSI independent of the inspection team. |
| IP Chapter 7 | Revised Training and Certification Program per SOL 483-24-01. |
| Appendix IP 01-H | Revised Underwater Bridge Inspection Diver requirements to hold a valid certification as a Bridge Safety Inspector issued by the Department and have successfully completed NHI Course #130091 – Underwater Bridge Inspection. |
| Appendix IP 01-I | Revised 6A44 and 6A45-6A48 descriptions to align with Pub 100A. |
| Appendix IP 02-B | Replaced Sample Bridge Watch Monitoring Forms with Sample Scour Monitoring Inspection Forms in Appendix C. |
| Appendix IP 02-H | Added Section 5 of the F&F Plan, the Members Detail Table. |
| Appendix IP 02-I | Added Uncoated Weathering Steel – Bridge Safety Inspection and Maintenance Manual. |
| Appendix IP 03-B | Retitled - Revised Engineering Judgement procedure, incorporating Method 1 for selected concrete bridges and Method 2 for structures such as stone masonry arch and metal arch bridges. |
| Appendix IP 03-E | Added new appendix for Load Rating Best Practices Manual. |
| Appendix IP 06-B | Added new appendix for Load Rating Quality Control Verification Checklist. |
| Appendix IP 06-C | Added new appendix for F&F Plan Quality Control Verification Checklist. |
| IE 4.3.5.6.1 | Added reference to IP 2.2.10 for guidance and requirements for inspection of uncoated weathering steel bridges. |
| IE 6B.6.1 | Revised guidelines for composite and non-composite encased I-girders. |
| **REVISIONS BELOW THIS LINE INCLUDED WITH 2024 EDITION, REVISION 1 (12-2024)** | |
| TOC | Revised headings and page numbers as needed. |
| IP 2.1.3.2 | Clarified bridge/culvert inspector qualifications. |
| IP 2.14 | Revised language for timely reporting of Priority 1/0 maintenance items and where to document POA information in BMS2. |
| IP 8.3.1 | Revised list of records to maintain in BMS2 (bridge file) and added requirements for bridge inspection reports. |
| IP 8.5.1 | Revised language for D-491 forms (no longer required to be included in inspection reports). |
| IP 8.5.2 | Revised reference for photos from SOW to IP 8.3.1. |
| Appendix IP 01-F | Revised title and language related to photographs and inspection report requirements. |

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| **AAR** | Association of American Railroads |
| **AASHTO** | American Association of State Highway and Transportation Officials |
| **ABAS** | Automated Bridge Analysis System |
| **AD** | AASHTO LRFD Bridge Design Specifications |
| **ADT** | Average Daily Traffic |
| **ADTT** | Average Daily Truck Traffic |
| **AMD** | Asset Management Division |
| **AORO** | Agency Open Records Officer |
| **APRAS** | Automated Permit Routing Analysis System |
| **ASD** | Allowable Stress Design |
| **ASR** | Allowable Stress Rating |
| **ATTT** | Access Roads to Tandem Trailer Truck Network |
| **BDTD** | Bridge Design and Technology Division |
| **BICT** | Bridge Inspection Crane Technician |
| **BICTS** | Bridge Inspection Crane Technician Supervisor |
| **BIRM** | Bridge Inspector's Reference Manual |
| **BIS** | Bridge Inspection Section |
| **BITM** | Bridge Inspection Training Manual |
| **BMS2** | Bridge Management System 2 |
| **BMS3** | PennDOT's web application for electronic data collection of Bridge Inspection Information |
| **BPR** | Bridge Problem Report |
| **CBSI** | Certified Bridge Safety Inspector |
| **CFR** | Code of Federal Regulations |
| **CRF** | Critical Ranking Factor |
| **CVN** | Charpy V-Notch |
| **DCNR** | Department of Conservation and Natural Resources |
| **DM4** | Publication 15M, PennDOT Design Manual Part 4, Structures |
| **ECMS** | Engineering & Construction Management System |
| **ECS** | Enterprise Content Services |

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| **EDC** | Electronic Data Collectors (Collection) |
| **EIB** | Encased I-Beam (IE-6) |
| **EV** | Emergency Vehicle |
| **F&F** | Fatigue and Fracture |
| **FAST** | Fixing America’s Surface Transportation |
| **FHWA** | Federal Highway Administration |
| **FO** | Functionally Obsolete |
| **FOIA** | Freedom of Information Act |
| **FPN** | Federal Project Number |
| **FRA** | Federal Railway Administration |
| **GIS** | Geographic Information System |
| **GRS** | Geosynthetic Reinforced Soil |
| **GVW** | Gross Vehicle Weight |
| **H&H** | Hydrologic and Hydraulic |
| **HBP** | Highway Bridge Program |
| **HOA** | Highway Occupancy Agreement |
| **HOP** | Highway Occupancy Permit |
| **IE** | PennDOT Bridge Safety Inspection Manual, Pub 238, Part B “Evaluation Specifications” |
| **IP** | PennDOT Bridge Safety Inspection Manual, Pub 238, Part A “Policies and Procedures” |
| **IR** | Inventory Rating |
| **LFD** | Load Factor Design |
| **LFR** | Load Factor Rating |
| **LOBSTORs** | Locally Owned Bridges on State Owned Roads |
| **LRFD** | Load and Resistance Factor Design |
| **LRFR** | Load and Resistance Factor Rating |
| **M / MBE** | AASHTO Manual for Bridge Evaluation |
| **MC** | Commentary to AASHTO Manual for Bridge Evaluation |
| **MPMS** | Multi-Modal Project Management System |
| **MPT** | Maintenance and Protection of Traffic |

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| **MSE** | Mechanically Stabilized Earth |
| **MUTCD** | Manual of Uniform Traffic Control Devices |
| **NARA** | National Archives and Records Administration |
| **NBI** | National Bridge Inventory |
| **NBIS** | National Bridge Inspection Standards |
| **NDT** | Non-Destructive Testing |
| **NCHRP** | National Cooperative Highway Research Program |
| **NHS** | National Highway System |
| **NSBA** | National Steel Bridge Alliance |
| **NSTM** | Nonredundant Steel Tension Member |
| **NTI** | National Tunnel Inventory |
| **NTIS** | National Tunnel Inspection Standards |
| **OCC** | Office of Chief Counsel |
| **OR** | Operating Rating |
| **OSA** | Observed Scour Assessment |
| **OSA** | Other State Agency |
| **OSAB** | Observed Scour Assessment for Bridges (IP-2) |
| **PA** | Pennsylvania |
| **PD** | Pub 15M – PennDOT Design Manual, Part 4, Part B “Design Specifications” |
| **PE** | Professional Engineer |
| **PennDOT** | Pennsylvania Department of Transportation |
| **PM** | Program Manager |
| **PP** | Pub 15M – PennDOT Design Manual, Part 4, Part A “Policies and Procedures” |
| **PSP** | Pennsylvania State Police |
| **PTC** | Pennsylvania Turnpike Commission |
| **PUC** | Public Utility Commission |
| **QA** | Quality Assurance |
| **QC** | Quality Control |
| **RAS** | Reimbursement Agreement System |

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| **RCRS** | Road Condition Reporting System |
| **RF** | Rating Factor |
| **RMS** | Roadway Management System |
| **ROW** | Right of Way |
| **RT** | Rating |
| **RTKA** | Right to Know Act |
| **RTKL** | Right to Know Law |
| **SAR** | Scour Assessment Rating |
| **SCBI** | Scour Critical Bridge Indicator |
| **SD** | AASHTO Standard Specifications for Highway Bridges |
| **SHVs** | Special Hauling Vehicles |
| **SI&A** | Structure Inventory and Appraisal |
| **SLC** | Safe Load Capacity |
| **SME** | Subject Matter Expert |
| **SNBI** | Specifications for the National Bridge Inventory |
| **SNBIBE** | Specifications for the National Bridge Inventory Bridge Elements |
| **SNTI** | Specifications for the National Tunnel Inventory |
| **SOL** | Strike-Off Letter |
| **SOW** | Scope of Work |
| **SR** | Sufficiency Rating |
| **SR ID** | State Route Identification |
| **USGS** | United States Geological Survey |
| **UWS** | Uncoated Weathering Steel |
| **UWS-SME** | Uncoated Weathering Steel – Subject Matter Expert |
| **TOMIE** | Tunnel Operations, Maintenance, Inspection, and Evaluation |
| **TTTN** | Tandem Trailer Truck Network |
| **VMS** | Variable Message Signs |
| **WBS** | Work Breakdown Structure |

**Appraisal Rating:** An evaluation of a functionality of a bridge characteristic or component in comparison to current standards for the highway system the bridge serves.

**Bridge:** A structure, including supports, erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads.

**Bridge Management System 2 (BMS2):** A system designed to optimize the use of available resources for the inspection, maintenance, rehabilitation, and replacement of bridges.

**Bridge Inspection Experience:** Active participation in bridge inspections in accordance with 23 CFR 650, Subpart C, in either a field inspection, supervisory, or management role. Some of the experience may come from relevant bridge design, bridge load rating, bridge construction, and bridge maintenance experience provided it develops the skills necessary to properly perform a bridge inspection. (23 CFR 650.305)

**Bridge Owner:** An organization or agency responsible for the inspection, load rating, and maintenance of one or more bridges.

**Condition Rating:** An evaluation of the physical condition of a bridge component in comparison to its original as-built condition.

**Complex Feature:** Bridge component(s) or member(s) with advanced or unique structural members or operational characteristics, construction methods, and/or requiring specific inspection procedures. This includes mechanical and electrical elements of moveable spans and cable-related members of suspension and cable-stayed superstructures. (23 CFR 650.305)

**Critical Finding:** A structural or safety related deficiency that requires immediate action to ensure public safety. (23 CFR 650.305)

**Culvert:** A soil interaction structure in an embankment that functions as a bridge. This structure may carry a highway, railway or pathway over a waterway, railway, highway or pathway. Culvert structure types include pipes, pipe arches, boxes and rigid frames and may be constructed of various materials.

**Damage Inspection:** An unscheduled inspection to assess structural damage resulting from environmental factors or human actions. (23 CFR 650.305)

**Element Level Bridge Inspection Data:** Quantitative condition assessment data, collected during bridge inspections, that indicates the severity and extent of defects in bridge elements. (23 CFR 650.305)

**Hands-On Inspection:** Inspection within arm's length of the member. Inspection uses visual techniques that may be supplemented by nondestructive evaluation techniques. (23 CFR 650.305)

**In-Depth Inspection:** A close-up, detailed inspection of one or more bridge members located above or below water, using visual or nondestructive evaluation techniques as required to identify any deficiencies not readily detectable using Routine Inspection procedures. Hands-on inspection may be necessary at some locations. In-Depth Inspections may occur more or less frequently than Routine Inspections, as outlined in bridge specific inspection procedures. (23 CFR 650.305)

**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. (23 CFR 650.305)

**Inspection Report:** The document which summarizes the bridge inspection findings, recommendations, and identifies the team leader responsible for the inspection and report. (23 CFR 650.305)

**Internal Redundancy:** A redundancy that exists within a primary member cross-section without load path redundancy, such that fracture of one component will not propagate through the entire member, is discoverable by the applicable inspection procedures, and will not cause a portion of or the entire bridge to collapse. (23 CFR 650.305)

**Inventory Data:** All data reported to the National Bridge Inventory (NBI) in accordance with the 23 CFR 650.315. (23 CFR 650.305)

**Legal Load:** The maximum load for each vehicle configuration, including the weight of the vehicle and its payload, permitted by law for the State in which the bridge is located. (23 CFR 650.305)

**Load Path Redundancy:** A redundancy that exists based on the number of primary load-carrying members between points of support, such that fracture of the cross section at one location of a member will not cause a portion of or the entire bridge to collapse. (23 CFR 650.305)

**Load Posting:** Regulatory signs installed in accordance with 23 CFR 655.601 and State or local law which represent the maximum vehicular live load which the bridge may safely carry. (23 CFR 650.305)

**Load Rating:** The analysis to determine the safe vehicular live load carrying capacity of a bridge using bridge plans and supplemented by measurements and other information gathered from an inspection. (23 CFR 650.305)

**National Bridge Inspection Standards (NBIS):** Federal regulations establishing national policy regarding bridge inspection organization, bridge inspection interval, inspector qualifications, inventory requirements, report formats, and inspection and rating procedures, as described in 23 CFR 650 Subpart C.

**National Bridge Inventory:** Inventory containing SI&A information for the nation's NBIS bridges.

**National Tunnel Inspection Standards (NTIS):** Federal regulations establishing national policy regarding tunnel inspection organization, tunnel inspection interval, inspector qualifications, inventory requirements, report formats, and inspection and rating procedures, as described in 23 CFR 650 Subpart E.

**National Tunnel Inventory:** Inventory containing SI&A information for the nation's NTIS tunnels.

**Nonredundant Steel Tension Member (NSTM):** A primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse. (23 CFR 650.305)

**NSTM Inspection:** A hands-on inspection of a nonredundant steel tension member. (23 CFR 650.305)

**Operating Rating:** The maximum permissible live load to which the structure may be subjected for the load configuration used in the load rating. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge. (23 CFR 650.305)

**Professional Engineer (PE):** An individual, who has fulfilled education and experience requirements and passed examinations for professional engineering and/or structural engineering license that, under State licensure laws, permits the individual to offer engineering services within areas of expertise directly to the public. (23 CFR 650.305)

**Program Manager:** The individual in charge of the program, that has been assigned the duties and responsibilities for bridge inspection, reporting, and inventory, and has the overall responsibility to ensure the program conforms with the requirements 23 CFR 650 Subpart C. The program manager provides overall leadership and is available to inspection team leaders to provide guidance.

**Quality Assurance (QA):** The use of sampling and other measures to assure the adequacy of QC procedures in order to verify or measure the quality level of a product or service.

**Quality Control (QC)**: Procedures that are intended to maintain the quality of a product or service at or above a specified level.

**Rehabilitation:** The major work required to restore the structural integrity of a bridge as well as work necessary to correct major safety defects. (23 CFR 650.305)

**Routine Inspection:** Regularly scheduled comprehensive inspection consisting of observations and measurements needed to determine the physical and functional condition of the bridge and identify changes from previously recorded conditions. (23 CFR 650.305)

**Safe Load Capacity:** A live load that can safely utilize a bridge repeatedly over the duration of a specified inspection interval. (23 CFR 650.305)

**Scour:** Erosion of streambed or bank material due to flowing water; often considered as being localized around piers and abutments of bridges. (23 CFR 650.305)

**Scour Assessment:** The determination of an existing bridge's vulnerability to scour which considers stream stability and scour potential. (23 CFR 650.305)

**Scour Critical Bridge:** A bridge with a foundation member that is unstable, or may become unstable, as determined by the scour appraisal. (23 CFR 650.305)

**Scour Plan of Action (Scour POA):** Procedures for bridge inspectors and engineers in managing each bridge determined to be scour critical or that has unknown foundations. (23 CFR 650.305)

**Service Inspection:** An inspection to identify major deficiencies and safety issues, performed by personnel with general knowledge of bridge maintenance or bridge inspection. (23 CFR 650.305)

**Special Inspection:** An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency, or to monitor special details or unusual characteristics of a bridge that does not necessarily have defects. (23 CFR 650.305)

**Structure Inventory and Appraisal Sheet (SI&A):** A summary sheet of bridge data required by NBIS.

**System Redundancy:** A redundancy that exists in a bridge system without load path redundancy, such that fracture of the cross section at one location of a primary member will not cause a portion of or the entire bridge to collapse. (23 CFR 650.305)

**Team Leader:** The on-site, nationally certified bridge inspector in charge of an inspection team and responsible for planning, preparing, performing, and reporting on bridge field inspections. (23 CFR 650.305)

**Temporary Bridge:** A bridge which is constructed to carry highway traffic until the permanent facility is built, repaired, rehabilitated, or replaced. (23 CFR 650.305)

**Tunnel:** An enclosed roadway for motor vehicle traffic with vehicle access limited to portals, regardless of type of structure or method of construction, that requires, based on the owner's determination, special design considerations that may include lighting, ventilation, fire protection systems, and emergency egress capacity. (23 CFR 650.505)

**Underwater Bridge Inspection Diver:** The individual performing the inspection of the underwater portion of the bridge. (23 CFR 650.305)

**Underwater Inspection:** Inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water or by wading or probing, and generally requiring diving or other appropriate techniques. (23 CFR 650.305)

**Unknown Foundations:** Foundations of bridges over waterways where complete details are unknown because either the foundation type and depth are unknown, or the foundation type is known, but its depth is unknown, and therefore cannot be appraised for scour vulnerability.

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**PART IP: POLICIES AND PROCEDURES**



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## PART IP: POLICIES AND PROCEDURES

Page

### Chapter 1 – Administrative Considerations

**Chapter 4 – Bridge Size and Weight Restrictions**

**Chapter 5 – PA's Bridge Management System 2**



**Chapter 6 – Quality Measures for Safety Inspection**

**Chapter 10 – Hauling Permits and APRAS**



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**Appendices for Chapter IP 10 Heavy Hauling Permits and APRAS**



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## 1.1 PURPOSE OF THIS MANUAL

The purpose of the Bridge Safety Inspection Manual is to compile the policies and procedures of the Pennsylvania Department of Transportation (herein referred to as the Department) as related to the Bridge Safety Inspection Program to ensure:

1. Public safety
2. Compliance with Federal and State regulations
3. Uniformity in inspection, evaluation, and load rating of Pennsylvania's public road bridges and tunnels
4. Accurate and adequate information to manage bridges and tunnels as a critical infrastructure asset.

This Manual is separated into two parts. Part IP covers the Department specific policies and procedures. Part IE follows along and directly modifies the Manual for Bridge Evaluation (MBE) for Department specific preferences as applicable. It is intended that this Manual be used by all persons involved in bridge and tunnel safety inspection activities along with the MBE and other references in IP 1.3.

This Manual is a revision of the Bridge Safety Inspection Manual, Publication 238, 2nd Edition dated March 2010.

## 1.2 SCOPE OF THIS MANUAL

The provisions of this Manual are intended for the safety inspection and management of bridges, culverts and tunnels involving public roads in Pennsylvania. This Manual also provides guidance on safety inspection of retaining walls and sign structures.

The term "bridge" as used throughout this manual shall be considered synonymous with "culvert" and "tunnel," as applicable, unless otherwise noted or implied. When NBIS or NBI is referenced, then bridge is intended to mean bridge. When NTIS or NTI is referenced, the information given only applies to tunnels. When both NBIS/NTIS or NBI/NTI are used, then bridge means bridge or tunnel.

This Manual will provide guidance on the following aspects:

1. Responsibilities of various parties for bridge safety inspections
2. Technical standards and specifications for bridge inspection, load rating and posting
3. Administrative requirements to meet State and Federal regulations regarding recording and reporting inspection information

Provisions are not included for bridges used solely for railway, rail-transit, or public utilities that are not related to public highways. For bridges not fully covered herein, the provisions of this Manual may be applied, as augmented with additional inspection and rating criteria where required.

This Manual is not intended to supplant proper training or the exercise of judgment by the Engineer, and states only the minimum requirements necessary to provide for public safety. The Owner or Engineer may require the sophistication of inspection, load rating or the testing of materials to be higher than the minimum requirements.

The Bridge Safety Inspection Program Strategic Plan (see Appendix IP 01-A) was developed to guide the Department toward successful implementation of its goals. This Manual supports the Strategic Plan mission, goals and objectives and will be updated as necessary.

The concepts of safety through redundancy and ductility and of protection against deterioration and scour are emphasized.

The Commentary located within the IE chapters of this Manual is not intended to provide a complete historical background concerning the development of this Manual, nor is it intended to provide a detailed summary of the studies and research data reviewed in formulating the provisions of this Manual. References are provided for those who wish to study the background material in depth.

The Commentary directs attention to other documents that provide suggestions for carrying out the requirements and intent of this Manual. However, those documents and this Commentary are not intended to be a part of this Manual.

PennDOT has a decentralized Bridge Safety Inspection Program that follows the guidelines and standards established by the Federal Highway Administration (FHWA) and The American Association of State Highway and Transportation Officials (AASHTO). Each District's Bridge Unit manages and administers the inspection of Department and local bridges in its area. PennDOT's Central Office, Bridge Inspection Section (BIS) in the Bureau of Bridge, is responsible for overall guidance and coordination of PA's inspection program.

Bridge safety inspection provides information on each bridge that is needed to complete and update each bridge's inventory/inspection record. This data resides in the Bridge Management System 2 (BMS2) that was implemented November 2006. This system accepts, stores, updates and reports physical and operating characteristics for all public bridges in Pennsylvania.

## 1.3 APPLICABLE SPECIFICATIONS AND STANDARDS

### 1.3.1 FHWA Requirements

#### 1.3.1.1 NATIONAL BRIDGE INSPECTION STANDARDS

The National Bridge Inspection Standards (NBIS) were developed after the 1968 Federal Highway Act became effective and were first published as a notice in the Federal Register, Volume 36, No. 81, Page 7851 on April 27, 1971. The NBIS have been amended several times by the Federal Highway Administration to include new provisions for inspection of nonredundant steel tension members (NSTMs), scour evaluations, and underwater inspections.

The NBIS are, therefore, mandated by Federal Law and are intended to ensure the proper inspection of the nation's bridges more than 20 feet in length on public roads. The National Bridge Inspection Standards are included in subpart C of Part 650 of Code of Federal Regulations (CFR), Title 23 - Highways. A copy of the current NBIS can be found on the FHWA website (https://www.fhwa.dot.gov/bridge/nbis.cfm).

The Federal Highway Administration gives policy guidance and establishes criteria and priorities for matching funds under various programs. In addition, FHWA reviews the results of those programs for compliance with the Standards through its annual compliance review.

#### 1.3.1.2 NATIONAL TUNNEL INSPECTION STANDARDS

The National Tunnel Inspection Standards (NTIS) were developed to require tunnel owners to establish an inspection program, maintain an inventory database, and report critical findings to FHWA as set forth in the Moving Ahead for Progress in the 21st Century Act (MAP-21). FHWA first solicitated public comment for the initial requirements for NTIS in 2008. The final rule establishing the NTIS is effective August 2015 and address comments from numerous State DOT's and engineering organizations among others. The NTIS are mandated by Federal Law and are included in subpart E of Part 650 of CFR, Title 23 - Highways. A copy of the current NTIS can be found on the FHWA website (https://www.fhwa.dot.gov/bridge/inspection/tunnel/).

### 1.3.2 Inspection Specifications

The latest versions of the following specifications, unless otherwise modified in this Manual, shall govern the safety inspection of bridges listed in the following order of precedence:

1) PennDOT Bridge Safety Inspection Manual, Publication 238 including all changes via Strike-off Letters (SOLs)

2) PennDOT Design Manual Part 4 Structures, Publication 15M

3) Title 23 Highways Code of Federal Regulations Part 650 Subpart C – National Bridge Inspection Standards

4) Title 23 Highways Code of Federal Regulations Part 650 Subpart E – National Tunnel Inspection Standards

5) AASHTO Manual for Bridge Evaluation, Third Edition, 2018, up to and including 2020 Interim Revisions

6) AASHTO Manual for Bridge Element Inspection

7) FHWA Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual

8) FHWA Specifications for the National Tunnel Inventory

9) AASHTO LRFD Bridge Design Specifications

10) AASHTO Standard Specifications for Highway Bridges

11) AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals

12) AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals

13) PennDOT BMS2 Coding Manual, Publication 100A

14) FHWA Manual of Uniform Traffic Control Devices

15) FHWA Guidelines for the Installation, Inspection, Maintenance and Repair of Structural Supports for Highway Signs, Luminaires, and Traffic Signals (FHWA-NHI-05-036)

16) PennDOT Traffic Engineering Manual, Publication 46

17) PennDOT Temporary Traffic Control Guidelines, Publication 213

### 1.3.3 Other Inspection Manuals and References

- FHWA Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges (FHWA-PD-96-001)
- FHWA Specifications for the National Bridge Inventory (FHWA-HIF-22-017)
- FHWA Bridge Inspector's Reference Manual (FHWA-NHI-23-024)
- FHWA Specification for the National Bridge Inventory Bridge Elements (01-21-2014)
- FHWA Inspection of Fracture Critical Bridge Members (FHWA-IP-86-26)
- FHWA Underwater Bridge Inspection Manual (FHWA-NHI-10-027)
- FHWA Underwater Bridge Repair, Rehabilitation, and Countermeasures (FHWA-NHI-10-029)
- FHWA Technical Manual for Design and Construction of Road Tunnels – Civil Elements (FHWA-NHI-10-034)
- FHWA Reference Guide for Load Rating of Tunnel Structures (FHWA-HIF-19-010)
- FHWA Hydraulic Engineering Circular No. 18, Evaluation Scour at Bridges (FHWA-HIF-12-003)
- FHWA Hydraulic Engineering Circular No. 20, Stream Stability at Highway Structures (FHWA-HIF-12-004)
- FHWA Hydraulic Engineering Circular No. 23, Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance (FHWA-NHI-09-111)
- NHI Publication 02-037: Fracture Critical Inspection Techniques for Steel Bridges
- Technical Advisory: Evaluating Scour at Bridges, T 5140.23
- Technical Advisory: Load-carrying Capacity Considerations of Gusset Plates in Non-load-path-redundant Steel Truss Bridges, T 5140.29
- Technical Advisory: Inspection of Gusset Plates Using Non-Destructive Evaluation Technologies, T 5140.31.

- Manual for Inspecting Bridges for Fatigue Damage Conditions, Research Project No. 85-02.
- AASHTO Moveable Bridge Inspection, Evaluation, and Maintenance Manual
- AASHTO LRFD Road Tunnel Design and Construction Guide Specifications
- AASHTO Guide Specifications for Internal Redundancy of Mechanically Fastened Built-Up Steel Members
- AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members
- PennDOT Handbook of Approved Signs, Publication 236
- PennDOT Design Standards for Bridge Construction, BC-700M Series, Publication 219M
- PennDOT Design Standards for Bridge Design, BD-600M Series, Publication 218M
- PennDOT Design Standards for Roadway Construction, RC 1M to 100M Series, Publication 72M

### 1.3.4 Modifications to PennDOT Inspection Publications

Revisions to Department manuals listed in IP 1.3.4.1 shall adhere to the following procedure. Whenever a user believes that modifications (including additions) would improve the present bridge safety inspection practices and procedures, the recommended modification shall be transmitted to the Assistant Chief Bridge Engineer - Inspection. Upon receiving the proposed modification request, the following steps will occur:

(a) The Assistant Chief Bridge Engineer - Inspection will review the recommended modification and determine if the change adds value and is worthwhile.
(b) If in agreement, the Assistant Chief Bridge Engineer - Inspection will oversee the changes as they are made to the documents in question and take appropriate action, including obtaining comments by means of a Clearance Transmittal Letter and securing FHWA approval if applicable.
(c) The revised or added page(s) will be distributed publicly via a Strike-Off Letter.
(d) If the proposed modification is not accepted, the Assistant Chief Bridge Engineer - Inspection will notify the originator of the reason for its rejection.

#### **1.3.4.1** APPLICABLE MANUALS

This modification procedure applies to the following Department inspection manuals:

- Bridge Safety Inspection Manual, Policies and Procedures, Publication 238
- Bridge Safety Inspection QA Manual, Publication 240
- Bridge Management System 2 Coding Manual, Publication 100A

## 1.4 INSPECTION PROGRAM ACTIVITIES

In addition to the field inspection and bridge rating, many activities by various parties are needed for a successful Statewide bridge safety inspection and management program. While specific responsibilities are detailed in the various sections of this Manual, an outline of the bridge inspection activities is as follows:

<u>FHWA</u>

- Annual Report to Congress on the condition of the nation's bridges
- Establishment of criteria for NBI/NTI data
- Collection and compilation of NBI/NTI data for all States
- Verification of NBIS/NTIS compliance for all States
- Provision of federal monies for bridge inspection
- Inspection of Federal Lands bridges in PA

<u>PennDOT Bridge Inspection Section (BIS)</u>

- Development of policies and procedures for the bridge inspection and management
- Collection and compilation of all bridge inventory and inspection data for all public roads in PA
- Development and analysis of bridge information for Statewide planning needs
- Verification and assurance of PA's compliance with NBIS/NTIS
- Reporting of PA's NBI/NTI data to FHWA
- Maintenance and operation of PA's BMS2
- Management of Statewide QA program for safety inspection

- QC of State Bridge Postings
- Maintenance of a Training and Certification program for bridge and tunnel safety inspectors
- Coordination of Statewide Scour Assessment program
- Maintenance of Statewide Federal Aid agreements for bridge inspection
- Maintenance of Statewide open-end engineering agreements for the underwater inspection of State and local bridges
- Maintenance of Statewide open-end engineering agreements for the NBIS inspection of State and local bridges
- Coordination of Statewide bridge inspection crane utilization
- Maintenance of software for electronic data collection (BMS3) and uploading to BMS2 of bridge inspection information

<u>PennDOT Engineering Districts, DCNR, and PTC</u>

- NBIS/NTIS compliance for all of the bridges for their respective jurisdictions
- Maintenance of an adequate and qualified in-house bridge inspection staff
- QC of their bridge inventory and inspection data/reports
- QC of local bridge inspection reports (Districts only)
- QC of bridge restrictions on local bridges
- Data entry into BMS2
- Review and approval of bridge posting recommendation for selected routes
- Establishment of engineering and/or reimbursement agreements for inspection of their bridges, as needed
- Establishment of engineering and/or reimbursement agreements for inspection of non-State bridges in their jurisdiction, as needed (Districts only)
- Operation and maintenance of PennDOT's bridge inspection crane fleet (Home Districts only)

<u>Other Bridge Owners</u>

- Inspection and rating of all bridges (by in-house staff or by consultant)
- NBIS/NTIS compliance
- Development of bridge posting recommendations for approval
- Establishment and maintenance of proper bridge postings
- Reporting of BMS2 data to Districts for input into BMS2

## 1.5 GENERAL INSPECTION PROGRAM TERMINOLOGY

For a comprehensive list of terminology, see the Glossary at the beginning of this Manual. Definitions for terminology commonly used in the Bridge Safety Inspection Program are as follows:

### 1.5.1 Bridge/Tunnel

A bridge is a structure, including supports, erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads.

A tunnel as defined by NTIS is an enclosed roadway for motor vehicle traffic with vehicle access limited to portals, regardless of type of structure or method of construction, that requires, based on the owner's determination, special design considerations that may include lighting, ventilation, fire protection systems, and emergency egress capacity.

### 1.5.2 Miscellaneous Structures Not Covered by This Manual

Other miscellaneous structures such as utility bridges, conveyor belts, pipelines, and traffic signal structures are not considered to be bridges. Provisions solely for inspections of such miscellaneous structures are not included herein. For a list of structures covered by the provisions of this manual see IP 1.2.

### 1.5.3 Highways (Public Roads)

A highway is a publicly maintained roadway open to the public for the purposes of motor vehicle traffic. A highway is publicly ordained as such through State statute or local ordinance. The term "public road" may be used interchangeably with the term "highways."

Interpretations of this definition include:

- The following are considered highways or public roads:
  - All State Routes
  - Local roads and streets on the inventory of routes receiving Liquid Fuels Tax allocations
  - All highways open to public vehicular traffic in State parks, forests, etc., even if public access is seasonal
  - Portions of the PA Turnpike system open to public vehicular traffic
- The following are not considered public roads:
  - Privately-owned driveways open to public vehicular traffic
  - Service roads in State parks, etc., not open to public vehicular traffic
  - Ramps and roadways open to the public but not ordained through statute or local ordinance
  - Routes limited to pedestrians, bicycles, snowmobiles, maintenance vehicles and/or emergency vehicles and not open to public vehicular traffic

### 1.5.4 Public Road Bridges

A public road bridge is defined as a bridge carrying a highway. Bridges carrying roads open only to public transit busses also qualify as public road bridges.

In cases where there is conflict or question, the Assistant Chief Bridge Engineer - Inspection shall make the determination whether or not a bridge route meets the definition of a public road. The District will prepare background information for this determination.

### 1.5.5 Highway Bridges

Highway bridges are those that carry highways or public roads.

### 1.5.6 Non-Highway Bridges and Structures

Non-highway bridges are bridges that are maintained on non-highways for the purpose of carrying motor vehicles (e.g., haul road vehicles, private drives) and/or non-motor vehicle traffic (e.g., pedestrians, bicycles, snowmobiles). Non-highway bridges also include:

- Bridges carrying non-publicly owned roads open to motor vehicle traffic
- Bridges that are normally restricted to pedestrian/trail use, but may carry occasional motor vehicles only for maintenance purposes or for emergency access (e.g., fire trucks, ambulances)
- Bridges that carry railroads over highways

Non-highway structures are highway-related structures (not bridges, or culverts or tunnels) such as retaining walls, noise walls, high mast lighting, etc. and include those that carry facilities (e.g., pipelines, sign structures) over highways or other features. (Note: Structures such as pipe lines, sign structures, and conveyor systems, even if located over a highway, are not considered to be bridges since they do not carry moving loads (vehicles or trains)).

### 1.5.7 Bridge Categories by Length

The general definition of a bridge in IP 1.5.1 does not depend upon its length. However, the bridge length defines many legal inspection requirements, program applicability, and funding eligibility.

#### **1.5.7.1** STRUCTURE LENGTH AND METHOD OF MEASUREMENT

The Structure Length, as defined by the FHWA Structure Inventory and Appraisal (SI&A) Coding Guide and as recorded in BMS2 Item 5B18, is the overall length measured along the centerline of roadway from paving notch to paving notch or back-to-back of backwalls of abutments, if present. Otherwise, the Structure Length is measured from end-to-end of the bridge deck, but in no case less than the bridge opening length.

Measure culverts, arches and pipes between the inside faces of walls along the centerline of the roadway regardless of their depth below grade.

- If there are multiple culverts or pipes and the clear distance between multiple openings is less than half that of the smaller contiguous opening, the measurement is to be between the inside faces of the exterior culverts or pipes.
- For pipes and arches where the measurement is to be taken between the inside faces, measure the length at the height of the springline.
- If the culvert or arch is at grade, measure the length from paving notch to paving notch if any, or back-to-back of exterior walls along the centerline of roadway.

If the structure is a highway tunnel, measure the Structural Length of the tunnel along the centerline of the roadway. For examples of measuring lengths of bridges, see the sketches in the instructions for BMS2 item 5B18 Structure Length in the BMS2 Coding Manual, Pub. 100A.

**1.5.7.2** BRIDGE OPENING AND METHOD OF MEASUREMENT

The Bridge Opening, as defined by the FHWA SI&A Coding Guide, is the length measured along the centerline of roadway between front faces of abutment stems, between inside faces of culverts and springlines of arches and pipes.

- If there are multiple culverts or pipes and the clear distance between multiple openings is less than half that of the smaller contiguous opening, the measurement is to be between the inside faces of the exterior culverts or pipes.
- If there are multiple arches, the measurement is to be between the inside faces of the exterior arches measured at the springline.

When underpinning or facing has been added to the bridge substructure, it should not be considered in the opening length measurement unless it extends the full height of the abutment.

For additional examples of measuring bridge openings, see the sketches in the instructions for BMS2 item 5E01 NBIS Bridge Length in the BMS2 Coding Manual, Publication 100A.

**1.5.7.3** NBIS BRIDGES

In order to be under the jurisdiction of the NBIS, and part of the NBI, the bridge must be a highway bridge and its bridge opening must be greater than 20’, measured along the centerline of roadway. The BMS2 Item 5E01 NBIS Bridge Length is provided to record whether the bridge meets the minimum opening specified for NBIS as per IP 1.5.7.2. NBIS bridges must be coded: BMS2 Item 5E01 = Y (for Yes)

**1.5.7.4** OTHER BRIDGES (8’-20’ LENGTH)

Bridges with openings less than the NBIS length are not governed by the NBIS and therefore there are no statutory requirements for their inventory and inspection.

Bridges in the 8’-20’ range may behave in a similar manner to those meeting the NBIS length definition and can present significant risks to public safety. Moreover, these bridges may represent a large portion of the infrastructure that owners have to maintain.

Best Business Practice:

- Inventory and inspect 8’ – 20’ bridges at the same level of scrutiny as NBIS-length bridges.
- Maintain these bridge records in BMS2 for further analysis and as the base information for asset management.

The Department requires that all PennDOT-owned bridges in the 8’ to 20’ range be inventoried in BMS2 and inspected in conformance with the criteria outlined in Table IP 2.3.2.4-2.

**1.5.7.5** MINOR BRIDGES (LENGTH < 8’)

Minor bridges (or culverts) less than 8' in length are a lesser concern because the risks to public safety are generally significantly lower than for larger bridges. However, minor non-culvert structures with separate bridge superstructures can be very sensitive to heavy axles and should be load rated for posting and safety.

Owners may elect to use BMS2 to maintain inventory and inspection data for these structures. The Department prefers that minor structures less than 6' in length be inventoried elsewhere.

**1.5.7.6** NTIS TUNNELS

All structures defined as highway tunnels (see IP 1.5.1 for definition) on all public roads, on and off Federal-Aid highways shall be included under the jurisdiction of NTIS and shall be inspected in accordance with the TOMIE Manual.

### 1.5.8 Inspection

For the purposes of this Manual, the term "inspection" encompasses all activities needed to determine and record the required inventory, condition, and appraisal items, including the structural analysis and rating. The term "field inspection" will be used when relating to the investigations at the site only.

## 1.6 PERTINENT STATUTES REGARDING BRIDGE INSPECTION

While bridge inspection is necessary for assurance of public safety, there are several statutes governing this activity. Some of the more critical statutes, but not all, are referenced in this section.

### 1.6.1 Federal Regulations

**1.6.1.1** NATIONAL BRIDGE INSPECTION STANDARDS (NBIS)

Initially adopted in 1970 following the infamous collapse of the Silver Bridge in West Virginia, the NBIS is the backbone of the inspection-related statutes. The 2022 NBIS mandates that inspections be performed and sets the applicable inspection and data reporting standards. The 2022 NBIS is contained in the CFR Title 23 Highways – Part 650, subpart C.

The NBIS addresses the following:

- Purpose. (§650.301)
- Applicability. (§650.303)
- Definitions. (§650.305)
- Bridge inspection organization responsibilities. (§650.307)
- Qualifications of personnel. (§650.309)
- Inspection interval. (§650.311)
- Inspection procedures. (§650.313)
- Inventory. (§650.315)
- Incorporation by Reference. (§650.317)

The requirements of the NBIS have been incorporated into this Manual. A copy of the current NBIS can be found on the FHWA website (https://www.fhwa.dot.gov/bridge/nbis.cfm).

**1.6.1.2** NATIONAL TUNNEL INSPECTION STANDARDS (NTIS)

The NTIS were established in 2015 and are contained in the CFR Title 23 Highways – Part 650, subpart E.

The NTIS addresses the following:

- Purpose. (§650.501)
- Applicability. (§650.503)
- Definitions. (§650.505)
- Tunnel inspection organization responsibilities. (§650.507)

- Qualifications of personnel. (§650.509)
- Inspection interval. (§650.511)
- Inspection procedures. (§650.513)
- Inventory. (§650.515)
- Incorporation by reference. (§650.517)

The requirements of the NTIS have been incorporated into this Manual. A copy of the current NTIS can be found on the FHWA website (https://www.fhwa.dot.gov/bridge/inspection/tunnel/).

### 1.6.2 Pennsylvania Consolidated Statutes TITLE 75 VEHICLES (Vehicle Code)

Title 75 contains the statutes for the use of PA State and local highways by vehicles. Most pertinent to bridge safety and an inspection is:

- Chapter 49 Size, Weight and Load
  The scope of this Chapter is to regulate vehicle size/weight and authorize restrictions to be placed on roads and bridges (i.e., weight postings).

  No vehicle, combination or load which has size or weight exceeding the limitations provided in this chapter and no vehicle, combination or load which is not so constructed or equipped as required in this title or the regulations of the Department shall be operated or moved upon any highway of this Commonwealth, unless permitted as provided in this title by the Department or local authority with respect to highways and bridges under their respective jurisdictions.

### 1.6.3 Pennsylvania Statute ACT 44 of 1988

Act 44 of 1988 (71 P.S. §512(a)(19)) gives the Department the power and duty to compile, maintain and forward to FHWA the required information for public road bridges greater than 20’ in length [NBIS bridges]. Act 44 imposes additional powers and duties upon the Department of Transportation relating to the inspection of bridges without regard to ownership. Act 44 empowers the Department to inspect those bridges owned by counties and municipalities that have not been inspected in accordance with NBIS.

According to the Department’s interpretation, Act 44 of 1988 directs the Department to inspect the municipality or county owned bridges if the owner:

- Refuses/is unable to inspect their bridges
- Refuses/is unable to inspect the bridges in a timely manner
- Elects to have the Department inspect the bridge on their behalf

Act 44 of 1988 also authorizes and directs the Department to post the bridges with the required information (e.g., weight restrictions, closures) in a similar fashion as the inspections.

Act 44 of 1988 also authorizes (in 71 P.S. §511.5) the Department to collect from the municipality’s Liquid Fuels allocation that share of the cost of the Department-directed inspections and postings that were not reimbursed by the FHWA.

### 1.6.4 PA Code Statutes TITLE 66 PUBLIC UTILITIES

Sections 2702 and 2704 of Title 66 establish the exclusive jurisdiction of the PA Public Utility Commission (PUC) over all parties (i.e., the Department, railroads, municipalities, utilities) at public crossings. This overarching authority balances and resolves any potential conflicting public interests within our highway right-of-ways where a highway and a railroad intersect at-grade or cross over or under. Highway-railroad grade separation bridges are considered to be rail-highway crossings subject to the PUC.

Through its official and legally enforceable Orders/Secretarial Letters, the PUC assigns various maintenance responsibilities to the involved parties at each individual bridge. These responsibilities may include performing and/or payment for: structure repair/rehabilitation/replacement, bridge inspection, weight/vertical clearance postings, track maintenance, highway maintenance, etc.

The District Grade Crossing Unit is the responsible organization for the liaison between the PUC, railroads, and other interested parties at any rail-highway crossing structure involving State Routes. The Right of Way, Utilities and Grade Crossing Division in the Bureau of Design and Delivery assists the Districts in District and Statewide PUC issues.

### 1.6.5 PA Code TITLE 67 TRANSPORTATION

TITLE 67 TRANSPORTATION sets forth the Department regulations governing transportation issues. The following sections are most critical to bridge inspection and management:

- <u>Chapter 179 Oversize and Overweight Loads and Vehicles</u>
  This Chapter regulates the use of State highways for the purpose of moving mobile homes, oversize or overweight vehicles, and combinations of vehicles, including the loads carried thereon, in order to preserve the safety of the users of Commonwealth highways; to facilitate the movement of mobile homes, oversize or overweight vehicles, and combinations of vehicles, as well as the movement of traffic, generally; to protect the structural integrity of the highway and bridge system; and to encourage the economic growth of commerce and industry in the Commonwealth without the necessity of constant supervision by Department employees, police and local officials. Nothing contained in this chapter is intended to relax existing safety requirements.

- <u>Chapter 185 Axle Weight Table</u>
  This chapter provides a table applying the formula in TITLE 75 VEHICLES § 4943(b)(1)(also known as the “Bridge Formula”) to the various numbers, weights and spacings of axles found on combinations registered in PA that weigh in excess of 73,280 pounds.

- <u>Chapter 191 Authorization to Use Bridges Posted Due to Condition of Bridge</u>
  This chapter regulates the use of bridges posted under 75 Pa. C.S. § 4902(a) (relating to restrictions on use of highways and bridges) by vehicles or combinations having a gross weight in excess of the posted weight limit or a physical dimension in excess of the posted size restriction.

- <u>Chapter 212 Official Traffic-Control Devices</u>
  The purpose of this chapter is to establish required study procedures and warrants for the establishment, revision and removal of restrictions on all public highways within this Commonwealth, whether by the Department on State-designated highways or by local authorities on any highway within their physical boundaries.

Further attention is drawn to critical subsections of this Chapter:

- § 212.117 (a) Weight Restriction based on condition of bridge
- § 212.117 (b) Weight Restriction based on condition of highway
- § 212.117 (c) Size Restriction based on condition of bridge or highway
- § 212.117 (d) Weight and Size Restrictions based on traffic conditions

## 1.7 RESPONSIBILITIES FOR BRIDGE SAFETY INSPECTION

### 1.7.1 General Inspection Responsibilities

#### 1.7.1.1 BRIDGE OWNER RESPONSIBILITIES

In this context, the term “bridge owner” applies to the party with overall maintenance responsibility for the bridge or structure. Thus, bridge owners may include the Department, counties, municipalities other than counties, other State, local and federal agencies, multi-party owners, parties assigned maintenance responsibilities by PUC, private parties/companies, etc.

The bridge owner has an overall obligation to ensure that its structure does not present an unacceptable safety risk to the public. In PA, the acceptable level of safety is defined by Department standards as presented or referenced in this Manual. Owner must perform restoration or repair activities or take other actions (i.e., closing or removal) to ensure public safety. In order to satisfactorily demonstrate that a structure is safe, safety inspections by the owner are best practice and, in some cases, prescribed by law and Department regulations.

The responsibilities of the bridge owner are further delineated for each type of bridge in the following sections.

**1.7.1.2** DEPARTMENT RESPONSIBILITIES

In addition to its responsibilities as a bridge owner for its many bridges and structures, the Department has federal and State statutory responsibilities for the safety and inspections of public road bridges in PA owned by others. Some of the more critical of these responsibilities include assurance of NBIS/NTIS compliance, proper bridge restrictions for vehicle size and weight, administration of federal monies for NBIS/NTIS inspection and the reporting of NBI/NTI bridge data to FHWA. In addition to public road bridges, the safety of non-highway bridges and structures over State Routes is a Department responsibility.

The responsibilities of the Department are further delineated for each type of bridge in the following sections.

**1.7.2 Highway Bridge & Tunnel Inspection Responsibilities**

**1.7.2.1** OWNER RESPONSIBILITIES FOR HIGHWAY BRIDGES

For NBIS/NTIS highway bridges, the owner responsibilities include:

- Inspection of the bridge in accordance with the NBIS/NTIS and Department standards
- Reporting of bridge inventory and condition information, as well as critical findings and follow-up actions to the Department in accordance with Department standards and in a timely manner
- Installation and maintenance of proper bridge restriction signing for vehicle weight and size, including barricades for closed bridges
- Maintenance of bridge inventory and inspection records
- Payment for any portion of the bridge inspection costs not included in the federal reimbursement program as administered by the Department
- Other responsibilities as outlined in the tunnel inspection agreement (between PennDOT and tunnel owners only)

DEPARTMENT HIGHWAY BRIDGES: The Districts are responsible to perform and manage the safety inspections of the Department bridges in their jurisdiction, including Department owned non-NBIS length bridges (8’-20’ length). The District may elect to inspect their bridges in house or hire a consultant to perform the work. If performing the inspections in house, the District shall have sufficient staff with proper training and available equipment.

LOCALLY OWNED HIGHWAY BRIDGES: The local bridge owner is responsible to perform and manage the safety inspections of all NBIS/NTIS bridges in their jurisdiction. The inspection teams may be from their in-house staff or from consultants, but they must meet NBIS or NTIS qualifications as required, see IP 2.1.3. Locals may also use consultants provided by the Department’s open-end agreements or may use an umbrella agency (e.g., a County-wide contract for township bridges).

TURNPIKE HIGHWAY BRIDGES: The Pennsylvania Turnpike Commission (PTC) is responsible for the inventory and inspection of the NBIS/NTIS bridges under their jurisdiction. The PTC will maintain the inventory and inspection data in the Department’s BMS2 in a timely manner through their remote terminals. The Department and BIS will provide technical guidance and assistance for all activities related to the inspection program.

DCNR HIGHWAY BRIDGES: The Department of Conservation and Natural Resources (DCNR) is responsible for the inventory and inspection of all NBIS/NTIS bridges in their jurisdiction. The DCNR will maintain the inventory and inspection data in the Department’s BMS2 in a timely manner through their remote terminals. The Department and BIS will provide assistance for all activities related to the inspection program.

OTHER STATE AGENCY HIGHWAY BRIDGES: Other State agencies that own NBIS/NTIS public road bridges are required to inspect their bridges or to contract with another agency to do so. The Department and BIS will provide assistance and coordinate all activities related to the inspection program

FEDERAL LAND HIGHWAY BRIDGES: For Federal Land bridges in PA, the Eastern Federal Lands Highway Division of FHWA is responsible for their inspection and NBI/NTI reporting. Send any questions about those bridges to BIS and they will be forwarded to the FHWA.

HIGHWAY BRIDGES UNDER PUC JURISDICTION: Highway bridges (>20' in length) under PUC jurisdiction carrying public roads over a railroad must meet the same inventory and inspection requirements of NBIS and the Department standards as other highway bridges. Non-NBIS highways less than 20' length carrying State Routes over a railroad are to be inspected to Department standards also.

For highway bridges over railroads under the jurisdiction of the PUC, the term "owner" is not entirely applicable because the PUC has the sole authority to assign "maintenance responsibilities" (including inspection responsibilities) for all or portions of the bridge through a PUC Order that is legally enforceable on all involved parties. The District Grade Crossing Engineer/Administrator should seek to have the PUC Orders clearly identify the party responsible for performing and reporting inspections and for the authority to approve the use of the bridge by hauling permit vehicles (see IP 10.3). For the PUC proceedings concerning determination of inspection, maintenance, and permit approval responsibilities, the Department would normally make the following arguments:

- The public agency that has administrative jurisdiction over the Public Road involved has a responsibility to the users of its highway to ensure their safety and should take the lead in resolving and inspecting the bridge.
- The party assigned maintenance responsibilities should also be responsible for the inspection duties.
- The party assigned maintenance and inspection responsibilities should be given the authority to approve hauling permits.
- For bridges not involving State Routes, the Department's participation should be limited to reporting required NBIS data to the FHWA.

For highway bridges over railroads under PUC jurisdiction, the District Grade Crossing Engineer/Administrator (E/A) must work closely with the District Bridge Engineer to ensure that all technical, safety, and administrative issues are resolved. The Grade Crossing E/A is to do the coordination and administrative tasks, while the District Bridge Engineer is to provide technical support for the bridge related issues. This technical support includes producing written documents (e.g., inspection reports, summary letters), attending meetings and providing testimony at hearings and other legal proceedings. A qualified staff engineer may be substituted for the District Bridge Engineer, as appropriate. See other portions of IP 1.7 and IP 2.8 for further instructions on highway bridges over railroads under PUC jurisdiction. Also refer to the Department's Grade Crossing Manual, Publication 371.

HIGHWAY BRIDGE OWNERSHIP IN DISPUTE: Where the ownership of a highway bridge on a State Route is in question, the District is to submit information to enable the Office of Chief Counsel to make a determination of the Department's legal position. If the bridge is an NBIS length bridge and other parties are not fulfilling the NBIS responsibilities, the Department shall inspect the bridge through its Districts until the issue is resolved.

NON-NBIS HIGHWAY BRIDGES: For non-NBIS highway bridges (primarily those with lengths 8'-20'), the owner may elect to inspect and report the inventory and condition information to the Department for inclusion in the BMS2. Department owned bridges 8'-20' in length must be inspected and data maintained in BMS2.

**1.7.2.2** DEPARTMENT RESPONSIBILITIES FOR HIGHWAY BRIDGES

The Department responsibilities include:

- Maintain an inventory of highway bridges in PA and their condition in BMS2, including:
  - All NBIS/NTIS highway bridges including those owned by locals and other agencies
  - All other highway bridges (8'-20' length) on State Routes or owned by the Department
  - All other highway bridges (8'-20' length) owned by others when requested by the owner

- Ensure compliance with NBIS/NTIS for all PA highway bridges.
- Ensure that vehicle size and weight restrictions on all bridges are in accordance with Department standards.
- Reporting NBIS/NTIS-required bridge inventory and inspection information to FHWA.
- QC/QA of bridge safety inspections.

**1.7.2.3** DEPARTMENT RESPONSIBILITIES FOR PUC JURISDICTION HIGHWAY BRIDGES ON STATE ROUTES

The Department is to ensure that all highway bridges carrying State Routes under the PUC jurisdiction are inspected in accordance with NBIS and Department standards.

PUC ORDERS FOR SAFETY INSPECTION INFORMATION: In the past, the PUC has issued Orders or Secretarial Letters requiring the Department to perform new structure safety inspections to provide the needed bridge condition information to serve as the basis for its Orders regarding maintenance of the bridge for public safety. The PUC has also requested that the Department provide copies of the inspection reports to all parties in the proceeding. Please note the following:

- Because the safety inspections may identify maintenance needs for portions of the bridge for which other parties are assigned maintenance responsibility, it is appropriate that the Department share the inspection reports with those parties. These other responsible parties may include the railroad (or its successor), the local municipality, etc.
- Providing the inspection information to the PUC may be problematic if the Department's inspection information is not identified as "Not for Public Disclosure" and handled accordingly. The bridge inspection record might mistakenly become part of the public record of the PUC proceeding and then subject to improper discovery and dissemination.

In order to provide the needed information to the various parties and to ensure that the information is kept from becoming public record, the following procedures should be followed:

1. The District, through the Grade Crossing E/A, must consult with the Office of Chief Counsel (OCC) if the PUC requests such inspection data and before any inspection information is disseminated.
2. The District is to provide copies of the inspection reports to the other parties with maintenance responsibilities at the crossing. Each recipient is to be advised that the records are not for public disclosure and instructed not to allow further dissemination or copying or inclusion in the PUC staff files. Each copy distributed by the District must have the "Not for Public Disclosure" stamp affixed as per IP 1.8.3.
3. If requested by the PUC, the District is to provide a copy of the inspection report to the PUC Bureau of Transportation, Rail Safety Division on a temporary basis to allow the PUC engineers to fully understand the bridge conditions and the Department's recommendations for the structure. The cover letter to the PUC should advise the PUC that the records are not for public disclosure and instructions not to allow further dissemination or copying or inclusion in the PUC staff files. Each copy distributed must have the "Not for Public Disclosure" stamp affixed as per IP 1.8.3. The District's cover letter should specify when the report is to be returned to the Department (minimum of 30 days, maximum 90 days). In addition, counsel will seek a protective order from the judge providing that: (a) The report is kept in a sealed envelope in the hearing file clearly marked as "Not for Public Disclosure" and (b) all parties are instructed that following the use of the report in the proceeding the report is to be returned to the Department, or destroyed with an affidavit submitted to the Department that the report has been destroyed. The District must ensure the report is returned to the Department.
4. If requested by the PUC and/or approved by OCC, the District may provide a Summary Letter of the bridge conditions (as outlined in IP 1.8.2.5) to the PUC. The Department will allow this Summary Letter to be entered into the public records of the PUC proceedings. The contents of this letter should be reviewed by OCC before its release to the PUC.

PUC BRIDGES REQUIRING POSTING OR CLOSING: For those PUC jurisdiction bridges carrying State Routes over a railroad, should the condition and appraisal reveal serious deterioration that warrants posting a new bridge load limit, changing an existing posted load limit or closing the structure, the District, acting on behalf of the Department, shall immediately take appropriate action to mitigate the problem. For a bridge restriction on a route that would normally require Highway Administration approval, submit a posting authorization request as per IP

1.7.2.4 and IP 4.6.3. If need be, the District is to take any emergency actions to safeguard the public.

Concurrent notification of impending or completed action is to be sent to the PUC requesting concurrence. Such notification to the Commission will be made by the District's Grade Crossing E/A with a copy of all correspondence to the Right of Way, Utilities and Grade Crossing Division of the Bureau of Design and Delivery. The District is to submit a letter to the PUC Bureau of Transportation and Safety, Railroad Safety Division, stating in general terms the need for establishing the bridge restriction and requesting concurrence from the PUC. The notification letter to the PUC should contain the BMS2 number (Item 5A01 SR ID) and the DOT Number (Item FR05 formerly titled AAR Number). The following public record information is to be attached to the PUC notification letter:

- Summary Letter prepared by the District Bridge Engineer stating need for the posting (see IP 1.8.2.5)

Additional inspection information is needed on a temporary basis by the PUC Rail Safety Division for their consideration of the District's posting recommendation. Because of the confidential nature of this inspection information, each of the following inspection documents must be stamped "Confidential" and noted in the letter for return within 30 days:

- Posting approval letter with completed bridge posting request form
- A copy of the inspection report
- Structural analysis of critical elements

NOTE: The Grade Crossing Unit shall be notified immediately of the date the posting or closing of the structure takes effect.

**1.7.2.4** DEPARTMENT RESPONSIBILITIES FOR LOCALLY OWNED NBIS/NTIS HIGHWAY BRIDGES

For locally owned NBIS/NTIS highway bridges, the Department has additional responsibilities under the NBIS/NTIS and PA Act 44 of 1988 to ensure that bridges are properly inspected and posted for weight restrictions as needed. Those responsibilities include:

- The Districts shall notify, in writing, the local government bodies that have responsibility for bridges on the local highway system and apprise them of the NBIS/NTIS requirements.
  - The letter shall state the Department's intent to inspect bridges on behalf of the local government in the event that NBIS/NTIS inspections become past due.
  - The letter should also inform the local bridge owner that the non-reimbursable portion of the inspection will be deducted from their liquid fuels allocation.
  - The letter shall be sent annually (in January/February) to the local government bodies with an attached list of bridges with inspection due dates in the forthcoming year beginning in April.
  - The letter shall also provide an attachment of a list of bridges requiring Scour Plans of Action.
  - The letter needs to be sent to the local bridge owner with a copy to their consulting engineer, when applicable. In cases when a county-wide agreement is utilized, the umbrella planning agency should also receive a copy. Provide a copy of all letters to The District Municipal Services Unit.
  - As stated in this section under the subheading NON-COMPLIANCE WITH NBIS/NTIS (see below), the letter must notify the local government(s) sixty (60) days before the bridge inspections could become past due.
  - A sample letter is located in Appendix IP 01-D, Local NBIS Inspection Notification Letter.
- The Districts should review local inventory and inspection procedures at least once every two years. The primary objective of such reviews should be to apprise the local governments of new developments and to provide necessary guidance.
- The Districts are to monitor local compliance with NBIS/NTIS on a monthly basis.
- The Districts are to perform QC measures on local bridges to ensure that restrictions needed for public safety are in place.
- The Districts should enter into agreements with the local governments to reimburse them for NBIS/NTIS- eligible inspection costs.
  - The Districts should review the status of the agreements, paper or electronic, in conjunction with sending the Local NBIS Inspection Notification Letter to the local governments.
  - The District must initiate the process to draft, review, and execute successive agreements at least one year prior to the expiration of an existing agreement to ensure continuity between consecutive

agreements.

- Under emergency conditions, the District should provide recommendations and engineering assistance to local governments upon request. Example: Furnish the services of the "Board of Inquiry - Bridge Collapse Investigation."
- The District should not furnish services which are normally available from private sources, such as engineers or inspectors, when there is ample time to contract for the same and public safety is not in jeopardy.
- The Districts are to maintain bridge inspection files for local bridges. At a minimum, the file for each individual bridge should contain the most recent inspection report with the current analysis/rating/posting evaluation, and other important safety related pertinent correspondence (e.g., critical deficiencies, posting letters). The owner is responsible to maintain the more complete bridge file (see IP 8).
- The District should provide liaison services relating to the above matters and should make an extra effort to explain recommendations and findings to the local officials and or employees to the extent needed for them to understand the findings and recommendations and the consequences of action or inaction.

NON-COMPLIANCE WITH NBIS/NTIS: When the local governments responsible for bridge inspection cannot or will not inspect their bridges in a timely manner in accordance with NBIS/NTIS and Department standards, Act 44 of 1988 requires the Department to perform the necessary inspections. Act 44 also authorizes the Department to deduct the owner's portion of the inspection costs from that municipality's Liquid Fuel Allocation. If the bridge Owner has not taken appropriate actions (e.g., engage an engineer) in a timely manner to have a bridge inspected to meet NBIS/NTIS requirements, the District is to follow the following procedure:

1. Notify the Owner in writing of the Department's intent to have the bridge inspected and to deduct the owner's share of the inspection costs from the Owner's Liquid Fuels Allocation. Appendix IP 01-D, Local NBIS Inspection Notification Letter, should be sent each year that the owner has bridge inspections due. The letter must be received sixty (60) days before the bridge inspection past due date.
2. Contract for consultant engineering services through a District or Statewide inspection agreement.
3. Have the consultant perform the inspection within thirty (30) days from the past due date and report findings to the bridge owner.
4. Deduct the owner's share of the inspection cost from its Liquid Fuels Allocation.

DEPARTMENT RESPONSIBILITIES FOR PUC JURISDICTION HIGHWAY BRIDGES ON LOCAL ROUTES: The Department's responsibilities for these NBIS bridges are similar to other bridges owned by the counties and municipalities, as listed previously in this section. The party to whom the PUC assigns responsibility for the maintenance of the bridge should also be tasked with the safety inspection. The Department can work with that party to ensure compliance with NBIS. If the Department's consultant prepares an inspection report on behalf of one of the parties, the District should not disseminate copies to other parties.

HIGHWAY BRIDGES OF UNKNOWN OR DISPUTED OWNERSHIP ON LOCAL ROADS: When the ownership of an NBIS highway bridge is unknown or in dispute, the Districts should act as mediators to assist in reaching agreement for the inspection responsibilities. The public agency that has administrative jurisdiction over the Public Road carried has perhaps the largest responsibility to the users of its road to ensure their safety and should take the lead in resolving which party will inspect the bridge. If the bridge is an NBIS length bridge and other parties are not fulfilling the NBIS responsibilities, the Department shall inspect the bridge through its Districts until the issue is resolved to ensure public safety.

HIGHWAY BRIDGES REQUIRING POSTING OR CLOSING: When it has been determined that an NBIS/NTIS bridge is not safe to remain open at the currently posted weight restriction and the local owner cannot or will not take the necessary actions to ensure public safety, Act 44 of 1988 authorizes and directs the Department to place required restrictions on that bridge. The Districts must take a proactive stance to resolve unsafe local bridges expeditiously. The Districts are to use the following procedure:

1. The District is to inform the local owner immediately of the concerns for the safety of the bridge. The initial contact may be by telephone or in person, but must be immediately followed by a letter, signed by a staff Professional Engineer (preferably the District Executive or the District Bridge Engineer) informing the owner of the bridge condition and its responsibility for public safety. Inclusion of inspection reports or reference thereto will be more compelling. Immediate delivery of the letter

(facsimile is acceptable) is recommended. Forward a copy of that letter and other supporting documentation to the Assistant Chief Bridge Engineer - Inspection.

2. If the owner indicates its unwillingness or inability to take appropriate actions, the District should immediately inform the Assistant Chief Bridge Engineer - Inspection of the situation and prepare plans for the Department to take necessary actions. These plans may include posting or closing the bridge until further studies can determine its safety or until repairs by the owner can be implemented. The Assistant Chief Bridge Engineer - Inspection will coordinate approval of this plan with the Chief Bridge Engineer and the Deputy Secretary for Highway Administration.
3. The Deputy Secretary for Highway Administration will authorize the District to proceed with its plans to restrict the bridge.
4. The District shall maintain records of associated costs for initial, revised and maintenance of existing postings and 100% of those costs will be deducted from the Liquid Fuels Allocation for the bridge owner.

DEPARTMENT ENGINEERING CONSULTANT AGREEMENTS FOR LOCAL BRIDGE INSPECTION: In order to have resources available to locals to ensure NBIS/NTIS compliance for all PA bridges or to perform emergency inspections, the Department will maintain open-end engineering consultant agreements for bridge inspection services and make them available to local owners (see IP 1.10.3). BIS will maintain a Statewide agreement available to all Districts. The Districts should consider having similar District-wide agreements if the projected need is present.

### 1.7.3 Inspection of Non-Highway Bridges and Miscellaneous Structures Over State Routes

#### **1.7.3.1** GENERAL DISCUSSION

Where a non-highway facility (a bike/pedestrian pathway, utilities, sign structure, etc.) exists over a Public Road, the bridge/structure needs to be inventoried only to be in compliance with NBIS. The NBIS does not require a structural safety inspection as it does for highway bridges. However, the Public Road owner may elect to require the bridge owner to perform a structural inspection as part of the occupancy agreement that permits the bridge or structure to be built and/or maintained over its Right-of-Way.

As part of its responsibilities as steward of its facilities, the Department is to verify and ensure public safety of non-highway bridges over State Routes by requiring the bridges to be inspected by the owner through formal legal agreements such as PUC Orders, Highway Occupancy Permits (HOP), or Highway Occupancy Agreements (HOA). The scope of such non-highway bridge inspections may be tailored to the subject bridge. Federal bridge inspection monies are available only for the inventory, not inspection of these bridges over Public Roads.

There are three basic situations where non-highway bridges or structures cross over State Routes, each with a different method of specifying bridge safety inspection requirements:

1. Rail bridges over State Routes:
   - Title 66 and the Public Utility Commission Orders (PUC) govern the crossing.
   - Inspection requirements: See IP 1.7.3.4 entitled Responsibilities for Railroad Bridges Over State Routes.
2. Non-highway utility structures over State Routes:
   - Example: standalone utility pipeline structures
   - Inspection requirements: To be included in Highway Occupancy Permit (HOP)
3. For other, non-utility, non-highway bridges/structures:
   - Examples: pedestrian/trail bridges, building passageways, sign structures, privately-owned driveway bridges
   - Inspection requirements: The Highway Occupancy Agreement (HOA) for the site must include the specific requirements for the inspection of the overhead bridge/structure. The General Scope of Work for Safety Inspection of State and Local Bridges shall be followed for inspection of these structures (see Appendix IP 01-F). Since these structures are not reported to the NBI, they require less inventory information to be collected (see Appendix IP 01-I).

#### **1.7.3.2** DEPARTMENT RESPONSIBILITIES FOR NON-HIGHWAY BRIDGES/STRUCTURES OVER STATE ROUTES

The Department is to ensure that non-railroad bridges or structures maintained over State Routes do not pose an unacceptable risk to public safety. In order to do so, all such structures must be inventoried and inspected. The District's responsibilities for the inventory and inspection of bridges over State Routes are as follows:

1. <u>Inventory of Bridge Information</u>
    a. Maintain required inventory data in BMS2.
    b. Maintain a permanent file of inventory and inspection information on the bridge.
    c. Maintain all information and APRAS data needed for permit routing on the State Route. Measure or verify all vertical and horizontal clearances associated with the State Route on an interval not to exceed 24 months.
    d. Provide a copy of specific maintenance needs and other relevant findings to the bridge owner after each inspection of a bridge site and/or update of BMS2 data.

2. <u>Inspection of Bridge Site</u>
    a. Inspect the highway environs portion of the bridge site on a 24-month interval for deficiencies. Record highway-related maintenance needs such as: Signing, drainage, pavement, guide rail, etc.
    b. Observe the overall condition of the bridge from the highway. Inform the bridge owner in writing of any deficiencies noted that present safety and/or maintenance problems.
    c. Maintain a permanent file of information on the bridge. Maintain all information and APRAS data needed for hauling permit review and approval.
    d. Inspect any Department-owned signing and supports for deficiencies.
    e. In BMS2, set BMS2 Item 7A03 Type of Inspection = H (for Highway Environs only) for inventory update/bridge site inspection to differentiate it from the safety inspection performed by the owner.

3. <u>Review and Acceptance of Owner's Bridge/Structure Inspection Report</u>
    a. Ensure that there is in place a legal agreement (i.e., PUC Order, HOA, or HOP, etc.) between the Department and the bridge owner concerning the requirements for safety inspection.
    b. Ensure compliance of owner with inspection scope and interval.
    c. Provide Quality Control (QC) review of the Bridge Inspection Report, including load ratings.
    d. Maintain inspection data in BMS2. Provide the bridge owner with a copy of updated BMS2 printouts.
    e. In BMS2, set BMS2 Item 7A03 Type of Inspection = O for safety inspection by owner.

NON-HIGHWAY BRIDGES OVER STATE ROUTES WITHOUT AN HOA OR HOP: There may be existing bridges or structures over State Routes that were previously allowed under a now non-standard agreement or without any formal agreement. New HOPs or HOAs shall include inspection requirements which are to be implemented at each site to bring uniformity to these non-highway bridges over State Routes. It is recommended that existing HOPs and HOAs with no inspection requirements be amended to include this requirement when possible.

BRIDGE DEFICIENCIES: Bridge or structure deficiencies are the responsibility of the owner. When critical deficiencies are found, the District must take a proactive stance to inform the owner of the Department's concerns and to ensure that the owner takes proper action to ensure public safety. The initial contact may be by telephone or in person, but must be immediately followed by a letter, signed by a staff Professional Engineer (preferably the District Executive or the District Bridge Engineer) informing the owner of the bridge condition and the owner's responsibility for public safety. This notification letter may include field views or inspection reports produced by the Department or consultants (or portions thereof), but the Districts must take care to protect the confidential nature of such information. See IP 1.8 for discussion of the procedures to be used when such sensitive information is to be shared. Immediate delivery of the letter (facsimile is acceptable) is recommended. Forward a copy of the bridge deficiency notification letter, and other supporting documentation, to the Assistant Chief Bridge Engineer - Inspection.

#### **1.7.3.3** OWNER RESPONSIBILITIES FOR NON-HIGHWAY BRIDGES/STRUCTURES OVER STATE ROUTES

The owner is responsible to maintain the bridge or structure in a condition as to not pose a threat to the public safety. The owner also has an obligation to demonstrate the safety of that structure to the public whose property the structure crosses. Accordingly, the owner must inventory and inspect the bridge and report the findings in a manner as directed by the legal agreement governing the crossing.

The Department policy for bridges and structures over State Routes is to assign the following inventory and inspection responsibilities to the owner of the overhead facility through the HOA, HOP, or PUC process:

1. Inventory
   a. Maintain permanent file of bridge/structure records as described in IP 8.
   b. Provide one copy of structure plans to the District for their records.
2. Inspection
   a. Inspect the bridge, load rate if required, and prepare a bridge inspection report in accordance with Department standards and requirements of the HOA/HOP, including:
      (1) Report all maintenance needs as per the BMS2 IM item screens
      (2) Report all completed maintenance or repairs.
      (3) Submit a copy of the Report to the District Bridge Engineer for review and acceptance.

For bridges under the jurisdiction of the PUC, the term "owner" is not applicable because the PUC has the sole authority to assign or re-assign "maintenance responsibilities" for all or portions of the bridge in an Order which is legally binding to all involved parties. The responsibility for bridge inspection should be outlined in the PUC Order. See IP 1.7.3.4 for further instructions on railroad bridges under PUC jurisdiction.

**1.7.3.4** RESPONSIBILITIES FOR RAILROAD BRIDGES OVER STATE ROUTES

This policy applies at all railroad bridge crossings over State Routes under the jurisdiction of the PUC, regardless of the status of the railroad. This may include operating railroads, former railroad company no longer operating as a railroad but possessing abandoned railroad facilities, an abandoned railroad owned by any individual or corporation purchased from a former railroad company, or a railroad being subsidized by the Commonwealth and/or Federal Government.

When a bridge carries a railroad over a highway, the PUC has jurisdiction over the rail highway crossing until the crossing is officially abolished through a formal PUC proceeding. Because of bankruptcy and reorganization of many railroad companies throughout the Commonwealth, the responsibility for inspection and/or maintenance for many of these railroad structures crossing highways is in question. Without ongoing surveillance, public safety may be in jeopardy. Where such structures are situated on right-of-way of non-operating railroads, the PUC retains jurisdiction over the highway-railroad crossing until it is formally abolished by an Order/ Secretarial Letter of the Commission, even though the Commission's jurisdiction over the non-operating railroads or their successor companies is not clear. Highway-railroad crossing structures with ownership, inspection and/or maintenance responsibilities in question do not serve the best interest of public safety and convenience.

The Department must be proactive to ensure all railroad bridges over State Routes are properly inspected. However, for rail-highway crossings under the jurisdiction of the PUC, the Department cannot unilaterally impose new bridge inspection requirements on the railroad or any other party.

For railroad bridges over highways, the PUC would normally assign maintenance responsibilities to the railroad. Because a good maintenance program would normally begin with a review of the infrastructure conditions, bridge inspections should have already been completed for that purpose. The CFR Title 49, Part 237, dated October 1, 2019, also provides requirements and guidance for railroad bridge inspection. Since it is reasonable to assume the railroads inspect their bridges for safety and/or maintenance, providing the Department with a portion of that inspection information should not be overly burdensome to them.

The District Grade Crossing E/A, with the assistance of the District Bridge Engineer, is to first ask the railroads to provide this safety inspection information through the Statewide cooperation agreements between the Department and railroads. If the railroads are not responsive, petition the PUC to require the railroads to inspect those bridges over State Routes and report the findings to BIS on a regular basis.

PUC REQUESTS FOR NEW SAFETY INSPECTIONS: In the past, the PUC has issued Orders or Secretarial Letters for the Department to perform new structure safety inspections and to provide copies of the inspection reports to all parties in the proceeding. These PUC requests pose many of the same issues raised in IP 1.7.2.3. The District, through the Grade Crossing E/A, must consult with the OCC if the PUC requests such inspection data.

In order to ensure the needed information is provided to the various parties and to ensure that the records are not for public disclosure, the following procedures should be followed:

1. The District, through the Grade Crossing E/A, must consult with the OCC if the PUC requests such inspection data and before inspections are performed and any information is disseminated.
2. SAFETY INSPECTION
   - The Department should agree to perform the inspections (by its own forces or by consultant) for the highway environs only of structures carrying railroads over State Routes, as required in IP 1.7.3.2.
   - It is the Department's opinion that the structure portion of the non-highway bridge should be inspected by the primary user on a regular basis similar to the provisions of IP 1.7.3.2. The District is to request that the Department receive any inspections made by other parties.
   - For highway-railroad crossing structures at locally owned roads, the Department should recommend that PUC assign the safety inspection of the structure and of the highway environs to the more appropriate parties involved at the crossing. The Department's role should be limited.
3. The District is to provide copies of the inspection reports to the other parties with maintenance responsibilities at the crossing. Each recipient is to be advised that the records are not for public disclosure and instructed not to allow further dissemination or copying or inclusion in the PUC staff files. Each copy distributed by the District must have the "Not for Public Disclosure" stamp affixed as per IP 1.8.3.
4. If requested by the PUC, the District is to provide a copy of the inspection report to the PUC Bureau of Transportation, Rail Safety Division on a temporary basis to allow the PUC engineers to fully understand the bridge conditions and the Department's recommendations for the structure. The cover letter to the PUC should advise the PUC that the records are not for public disclosure and instructions not to allow further dissemination or copying or inclusion in the PUC staff files. Each copy distributed must have the "Not for Public Disclosure" stamp affixed as per IP 1.8.3. The District's cover letter should specify when the report is to be returned to the Department (minimum of 30 days, maximum 90 days). In addition, counsel will seek a protective order from the judge providing that: (a) The report is kept in a sealed envelope in the hearing file clearly marked as "Not for Public Disclosure" and (b) all parties are instructed that following the use of the report in the proceeding the report is to be returned to the Department or destroyed with an affidavit submitted to the Department that the report has been destroyed. The District must ensure the report is returned to the Department.
5. If requested by the PUC and/or approved by OCC, the District may provide a Summary Letter of the bridge conditions (as outlined in IP 1.8.2.5) of the portions it inspected to the PUC. The Department will allow this Summary Letter to be entered into the public records of the PUC proceedings. The contents of this letter should be reviewed by OCC before its release to the PUC.

Because the bridge carrying a railroad over the highway is not an NBIS bridge, Federal reimbursement of inspection costs is not available. For non-NBIS structures, all of the inspection costs should be borne by the structure owner or party responsible for maintenance of the structure. PennDOT will cover the cost of the highway environs inspection of these structures.

BMS2 INVENTORY and INSPECTION DATA: To provide reasonable assurance that public safety is not jeopardized, a proper inventory and inspection would require the BMS2 information outlined in Table IP 1.7.3.4-1.

DEPARTMENT RESPONSE TO DANGEROUS BRIDGE CONDITIONS: For those structures carrying railroads over State Designated Highways, should any dangerous conditions (which would jeopardize highway or railroad traffic) be noted, the District Grade Crossing Unit shall immediately notify the railroad, the Right of Way, Utilities and Grade Crossing Division of the Bureau of Design and Delivery and the PUC. If the situation is urgent, the District should first take proactive steps to ensure public safety and contact the PUC as time permits.

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**Table IP 1.7.3.4-1 Inventory and Inspection Data for Railroad Bridges over State Routes**

| BMS2 Input Pages | Inventory and Inspection Information Required |
|---|---|
| D-491 5A/6A/VM | Items 5A01, 6A06, VM05, 5A15, 5A21, VM02, VM03, VM04, 5A17 and 5A18 |
| D-491 4A/5A/5B/5C/6C/FR | Items 4A19, 4A20, 5A07, 5A19, 5B09, 5C01, 5C03-5C06, 5C10-5C11, 5C14, 5C22, 5C28-5C29, 5C33, 6C05, 6C18-6C23, 6C25-6C26, 6C34, FR03-FR06, FR07, FR10-FR15 |
| D-491 5B/5E/6A/VD | Items 5E04, 6A26-6A29, 5E01, 5B11, 5B14, 5B17, 5B18, and VD19<br>Note: 5E01 = N |
| D-491 4A | Item 4A01 |
| D-491 1A//4A/4B/6B/7A/VA | Items 7A14, 7A01, 7A03, 7A05, 1A01-1A06, 6B36-6B40, 6B48 4A02, 4A08-4A11, 4B03 and VA02 |
| D-491 IM | Items IM03-IM06, IM09, IM10, IM15A for highway related activities |

NOTES:
- Items 7A01, 7A03, 7A05 and 7A14 are required for each inspection (by Department or owner)
- Items 4A01, 7A14, 7A01, 7A03, 7A05, 1A01-1A06, 6B36-6B40, 4A02, 4A08-4A11, 4B03 and VA02 are required by BMS2 editing functions.
- When Department (or highway owner) inspection is limited to highway environs (7A03 = H), the value “N” should be entered into items not rated/evaluated. Indicate on the field inspection report narrative fields if an item is not applicable or if it was not inspected. The highway owner may elect to evaluate certain items such as 4A10 Deck Geometry or 4A08 Safety Features for features related to the State Route only for their information and further evaluation.
- When Department (or highway owner) inspection is limited to highway environs (7A03 = H), the highway owner should record maintenance items for highway related activities.

## 1.8 RELEASE OF INVENTORY AND INSPECTION INFORMATION

### 1.8.1 General Discussion

#### 1.8.1.1 APPLICATION

The policies and procedures in this section shall apply to all information collected or retained by the Department regarding the inventory and safety inspection of structures in PA including bridges, culverts, walls, sign structures and other miscellaneous types of highway-related structures. This information shall include, but not be limited to all information, documentation, drawings, computer software, ideas, and concepts in any tangible format.

#### 1.8.1.2 RESPONSIBLE PARTIES FOR STRUCTURE INFORMATION

All requests for structure information should be referred to the responsible party:

- Statewide Structure Data:
  The Assistant Chief Bridge Engineer - Inspection is responsible for the dissemination of Statewide structure statistics and information.
- Individual Structure or District-wide Data:
  The District Bridge Engineer is responsible for the dissemination of statistics and information regarding Department and locally owned structures, on an individual or District-wide basis.
- Other State Agency Bridges:
  The lead individual in the appropriate agency is responsible for the dissemination of statistics regarding that agency’s structures.

#### 1.8.1.3 GENERAL TYPES OF STRUCTURE INFORMATION

**Inventory Type Information**

Inventory type information pertains to a structure’s characteristics that change only when the structure is altered in some way. General categories of inventory information include:

- General – Identification, location, owner information, detour length, etc.
- Features Intersected – Feature carried or intersected, vertical and horizontal bridge clearances, highway

network data, traffic data, railroad information, etc.

- Structure – Length, width, type of structure, span, configuration, material, foundation, structure details, etc.
- Stream and Navigation – Stream information, navigation controls, etc.
- Bridge Posting – Weight limits and associated dates, Blanket Permit Bridge Restrictions, etc.
- Utility – Occupancy information
- Proposed improvements – Information regarding planned or scheduled projects may be released if there is no conflict with other regulations.

**Inspection Type Information**

Inspection type information pertains to the assessment of the condition of the structure, the appraisal of its functionality and recommendations for maintenance or improvements. Load capacity analysis/ratings and assessment of scour potential are considered to be inspection information.

**1.8.1.4** PERTINENT FEDERAL AND PA STATUTES

A number of statutes and regulations have been enacted regarding the release of bridge inspection records. The following list with its comments is provided for information only and may not be a complete or up-to-date listing.

**Federal statutes**

- Freedom of Information Act (FOIA)
  Provides for the release of public documents but applies only to federal agencies
- Code of Federal Regulations (CFR) Title 23, United States Code (U.S.C.) § 409 [ 23 U.S.C. §409]
  Restricts the discovery and admission of certain documents compiled or collected for the purpose of identifying and planning highway safety enhancements.

**Pennsylvania statutes**

- PA Right to Know Law (RTKL) [ 65 P.S. §67.101 et seq.]
  Provides for the release of "public records" of a State or local agency
- PA Vehicle Code, Title 75 [75 Pa. C.S. §3754(b)]
  Provides that the use of in-depth accident investigations and other safety study records and information is prohibited in any legal action or proceeding, and that information is protected from disclosure in the discovery process. Bridge inspections are considered to be one of the types of engineering and safety studies protected from disclosure.

**1.8.1.5** RESPONSIBILITY FOR LEGAL INTERPRETATION

The Office of Chief Counsel (OCC) is responsible for interpretations of all pertinent statutes and regulations regarding the Right to Know Law (RTKL). Contact the Assistant Chief Counsel for the General Law Division when questions arise.

### 1.8.2 Requests for Release of Bridge Records

**1.8.2.1** INFORMATION FOR DEPARTMENT STRUCTURES

The Department is to follow the direction of the Agency Open Records Officer (AORO) in the handling of a request under the RTKL, which accounts for then applicable policies. This direction will help shape the agency's course of action with respect to other requests for bridge inspection information, including without limitation requests from the press. policies set forth by the RTKL and Management Directive 205.36. Unredacted bridge inspection reports are not subject to access under the RTKL or public information. The Department, after weighing all applicable law and policy, may release redacted bridge inspection reports where appropriate. In most cases, bridge inspection information shall be considered exempt from the RTKL. However, in some circumstances, this information may be deemed nonexempt. Figure IP 1.8.2.1-1 shows the general process for a RTKL request for those cases deemed nonexempt.

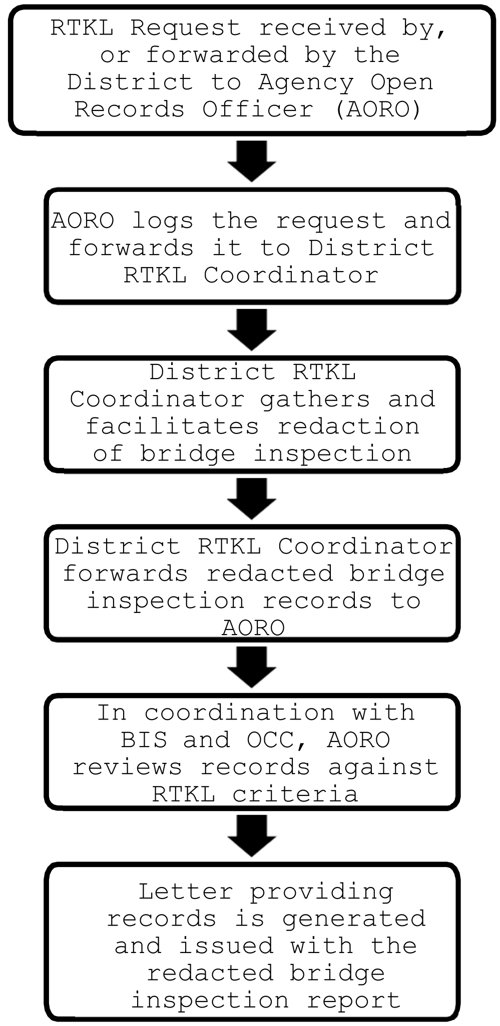


**Figure IP 1.8.2.1-1 – Right –to – Know Process**

**1.8.2.2** INFORMATION FOR NON-DEPARTMENT STRUCTURES

Information for other PA structures may be subject to disclosure. The procedure in Figure IP 1.8.2.1-1 applies to the release of that information.

**1.8.2.3** REQUESTS FOR INSPECTION TYPE INFORMATION FROM COURTS OR ADMINISTRATIVE AGENCIES

For PUC requests for inspection type information, see IP 1.7.2.3 and IP 1.7.3.4.

This section applies to a request or demand for inspection related information issued by a State or Federal Court or Administrative Agency via an appropriate order or subpoena. This section does not address information requests governed under the RTKL or FOIA.

If a subpoena or other written court order is issued requesting bridge inspection information, the District is to forward the subpoena or other written court order requesting the relevant bridge information with a cover letter via email with a hard copy to:

Deputy Chief Counsel
General Law Division
400 North Street, 9th Floor
Harrisburg, PA 17105

The District is to send a copy of the information request to the Assistant Chief Bridge Engineer - Inspection. If the District is aware that a specific attorney from either PennDOT Office of Chief Counsel or the Office of the Attorney General is assigned to the case referenced in the subpoena or other written court order, send a copy to that lawyer also.

**1.8.2.4** RECOUPMENT OF COST OF INFORMATION

The full costs of gathering and release of inventory information (including research, printing, handling and mailing) should be paid by the requestor.

**1.8.2.5** SUMMARY LETTERS FOR REPORTING STRUCTURAL CONDITIONS

There are situations where the Department is warranted to provide a synopsis of pertinent structure conditions to justify an action or recommendation, including:

- Establishment of bridge weight restrictions or bridge closure
- Justification for emergency repairs
- To inform the public of the need for bridge repairs, reconstruction, or replacement
- In lieu of releasing the actual inspection report to PUC.
- Claims by the Department to recoup costs due to damage by others

<u>Do not release the structure safety inspection report</u>. A Summary Letter Report with a synopsis of the condition(s) of most concern should suffice for most situations.

If there is a question about the need or advisability of such a Summary Letter Report and/or its content, contact the BIS.

**<u>Guidelines for Preparation of a Summary Letter Report</u>**

The following guidance is offered for the preparation of a Summary Letter Report:

- Do not reference a specific inspection report. State that this is the current condition of the structure.
- Provide only excerpts pertinent to the information request. For example, a reply concerning a parapet collision would not warrant a discussion of waterway opening.
- Be concise and to the point.
- Do not just copy or re-type the original inspection report.
- Photos from the inspection report may be used.
- The synopsis is to be signed and sealed by the District Bridge Engineer as the Department's Professional Engineer responsible for the safety of the structures.

**<u>Special Meetings on Structure Conditions</u>**

In certain situations, it may be in the Department's best interest to discuss structure conditions with involved parties to reassure them that we are taking a proper and prudent course of action. Such situations may include bridges with visible deterioration that causes public concern, sudden or critical bridge postings/closures, etc.

A Special Meeting, conducted by a registered Professional Engineer from the District Bridge Unit, is advised to ensure that the structure information is understood. Selected inspection documents may be viewed, but none are to be copied or released. A Summary Letter may be released.

**1.8.2.6** REQUESTS FOR INSPECTION INFORMATION ON RESEARCH PROJECTS AND STUDIES

The Department may release certain inspection information research purposes and other special studies. Forward all requests for such information to the BIS for response.

### 1.8.3 Not for Public Record Notice and Stamp

For Department structures, place the Not for Public Record Notice shown in Figure IP 1.8.3-1 in the front of any structure safety inspection that is completed by the Department or for the Department. It may be printed on the document cover or on the face of a file folder in which the document is contained.

**Not for Public Record – Structure Safety Inspection Study**

**This document is the property of the Commonwealth of Pennsylvania, Department of Transportation. The data and information contained herein are part of a structure safety inspection study. This safety study is only provided to those official agencies or persons who have responsibility in the highway transportation system and may only be used by such agencies or persons for safety-related planning or research. The document and information are not public pursuant to federal law and may not be published, released or disclosed without the written permission of the PA Department of Transportation.**

**Figure IP 1.8.3-1 Not for Public Record Notice for Department Structures Inspection Documents**

For Department structures, affix the STAMP shown in Figure IP 1.8.3-2 <u>on each page</u> of any structure safety inspection information that is released.

**This document includes structure safety inspection information that is not public pursuant to federal law and may not be published, released or disclosed without the written permission of the PA Department of Transportation.**

**Figure IP 1.8.3-2 Not for Public Record Stamp for Department Structure Inspection Documents**

For non-Department structures, place the Not for Public Record Notice shown in Figure IP 1.8.3-3 in the front of any structure safety inspection that is completed by the Department or its consultants for a local bridge owner. It may be printed on the document cover or on the face of a file folder in which the document is contained.

**Not for Public Record – Structure Safety Inspection Study**

**This document is the property of the << Insert bridge owner's name>>. The data and information contained herein are part of a structure safety inspection study. This safety study is only provided to those official agencies or persons who have responsibility in the highway transportation system and may only be used by such agencies or persons for safety-related planning or research. The document and information are not public pursuant to federal law and may not be published, released or disclosed without the written permission of the<<Insert bridge owner's name>>.**

**Figure IP 1.8.3-3 Not for Public Record Notice for Non-Department Structure Inspection Documents**

## 1.9 ELIGIBILITY OF BRIDGE INSPECTION ACTIVITY COSTS FOR FHWA REIMBURSEMENT

The Department has the responsibility and authority for the distribution of Federal highway funds in PA. The Department has determined that, in general, inspection costs for NBIS/NTIS bridges are eligible for reimbursement from federal funds. The federal reimbursement is generally limited to 80% of the eligible costs. The remaining 20% and all other non-eligible costs must be borne by the bridge owner.

To be eligible for reimbursement of these costs, the bridges must be National Bridge Inventory (NBI) bridges or National Tunnel Inventory (NTI) tunnels on public roads and not supported by tolls. See Publication 93 for approved process for selection-based criteria required for federal reimbursement eligibility. Any questions on eligibility of individual bridges, other structures or activities are to be forwarded to the BIS for resolution.

Inspection activities that are presently eligible for reimbursement are as follows:

- Bridge inspection activities including field inspections, underwater inspections, inspection reports, bridge analysis and rating, scour assessments, recommendations for repairs and improvements, testing, etc.
- Inspection of other structures including tunnels, sign structures, retaining walls and drainage structures.
- QA and QC efforts at District and BIS.
- Inspection administration activities (at BIS, District, and other owners) including scheduling,

inspection agreement management, critical deficiency meetings with locals, etc.
- BMS2 activities including data entry, analysis, etc.
- Non-professional services to support NBIS/NTIS inspections such as Maintenance and Protection of Traffic rental and labor, temporary access platforms for inspection (rigging, scaffolding, etc.), access equipment rental (cranes, lifts, etc.).
- The initial inventory of bridges on public roads.
- Neither the design of repairs nor the construction of repairs are eligible for NBIS/NTIS Safety Inspection monies.

Districts are responsible to see that federal funds are being properly utilized for NBIS/NTIS bridge inspections.

Reimbursement agreements for bridge inspections between the Department and other bridge owners are to be reviewed with the above guidelines in mind. The Reimbursement Agreement System (RAS) contains the standard reimbursement agreement to be used between the Districts and local Bridge Owners. While eligibility for reimbursement starts with approval of the Form 4232, the standard reimbursement agreement should be fully executed prior to the start of work. The standard reimbursement agreement provides NBIS/NTIS requirements that must be satisfied by the local bridge owner, default conditions, and requires Department approval of consultant agreements before start of work.

The first-time weight restriction posting of an NBI/NTI bridge is eligible for 80% federal reimbursement through a separate reimbursement agreement from the inspection agreement.

## 1.10 ENGINEERING AGREEMENTS FOR SAFETY INSPECTION WORK

Safety inspection work may be done by qualified in-house staff of the bridge owners or through qualified engineering consultants.

### 1.10.1 Applicability

Engineering agreements may be used to perform a wide variety of safety inspection work for bridges, culverts and other structures. Inspection related work may include, but is not limited to:
- Routine and In-Depth Inspections
- Special Inspections
- Underwater Inspections
- Bridge capacity analysis and ratings
- Special Studies and testing
- Other related work

Districts are to utilize engineering agreements for inspection work to augment staff resources and expertise to ensure NBIS/NTIS compliance and maintain public safety.

Where structure systems at a single location are designated by multiple Bridge Management System (BMS2) numbers, ensure consistent inspection procedures, documentation, reporting, and overall quality control throughout the inspection process. When using an engineering agreement, a single prime engineering consultant shall be used to inspect the structure system to obtain these objectives.

### 1.10.2 District Inspection Agreements

It is preferred that the Districts maintain their own inspection agreements to provide for their ongoing and anticipated needs. For emergencies, unanticipated needs, or when special expertise is required, utilize BIS inspection agreements.

For underwater inspections by divers of Department or local bridges, it is preferred to utilize BIS's Statewide agreements established for that purpose, rather than separate District agreements. At this time, only Engineering District 3-0 is maintaining dive teams capable of performing NBIS underwater inspections.

Some Districts have maintained District-wide or Countywide engineering agreements for local bridge inspection, where the local bridge owners have needed the Department's assistance routinely to maintain NBIS/NTIS compliance. This approach can be advantageous where greater District control is desired or when there are a large number of bridges or owners involved.

Funding for NBIS Length Local Bridge inspection work under ECMS agreements is provided through the Bureau of Bridge fiscal year budget. Accordingly, all ECMS District owned agreements for local bridge inspection work requires funding approval from BIS for all parts, work orders and amendments of those agreements prior to being created under the ECMS agreement. Funding approval will require the District to submit to BIS a detailed cost estimate for all parts, work orders and amendments as well as a bridge list or list of tasks associated with all parts, work orders and amendments. Once the part, work order or amendment is in legal processing the District is required to submit to BIS a Project Expenditure Approval Form (Pink Sheet) for processing by the Bureau of Bridge (contact BIS for a blank form).

### 1.10.3 BIS Inspection Agreements

BIS maintains several engineering agreements to be used by the Districts and local bridge owners for inspection-related work to:

- Ensure NBIS/NTIS compliance for all PA bridges
- Ensure adequate resources are available for immediate response to emergencies
- Provide specialized expertise

#### **1.10.3.1** UNDERWATER INSPECTION AGREEMENTS

BIS routinely has multiple Statewide underwater inspection agreements available for use on Department and local bridges. The use of these contracts is preferred, rather than separate agreements.

Early in January of each calendar year, BIS will request the Districts to prepare a list of Department and local bridges that need to have underwater inspections performed under the auspices of the BIS agreements. BIS will assign the bridges to the appropriate agreement based upon special inspection needs, consultant workload, geographic economy and other factors. This early request for the list of needed underwater inspections assists in project planning and economy.

If an emergency arises, contact BIS for immediate assistance.

#### **1.10.3.2** ACCESS TO BIS AGREEMENTS FOR LOCAL BRIDGES

Local bridge owners cannot directly access the BIS agreements. The Districts are to coordinate inspection needs for local bridges and include them in Work Order requests to BIS.

#### **1.10.3.3** PROCEDURES TO UTILIZE BIS INSPECTION AGREEMENTS

In addition to the instructions in the Department's Policy and Procedures for the Administration of Consultant Agreements, Publication 93, the following procedures were established to make these agreements available to all Districts and to assist in their management:

1) Request for Work Order – Districts are to contact BIS's bridge inspection agreement manager to request authorization to utilize a BIS Agreement. A telephone call will suffice. BIS will assign a Work Order # for the appropriate agreement.

   More than one Work Order may be required if bridges involved are of more than one inspection funding category.

2) Scope of Work – Districts are responsible to determine the scope of work required at each bridge for the Work Order. BIS will provide assistance for unusual or special situations. The District may work with the consultant in developing the scope. Each agreement is based on the Scope of Work standard at the time of development, so only special requirements usually need to be identified.

3) Work Order Costs – Districts are responsible for funding the cost of the Work Order.

   For COST PER UNIT OF WORK agreements, only the list of bridges and units of work on each are required for scope and cost. The Work Order cost will be based on contract unit prices.

4) Work Order Preparation – District prepares and executes Work Order. Routing as per Publication 93 will result in circulation to BIS for approval.

5) Control of Work – District is responsible for quality control review of work products produced. District reviews inspection reports to ensure acceptability. District updates BMS2 as needed. Note, for local bridge inspections, additional data is needed on BMS2's 1A, 4A, 6A, 6B and 7A screens to recoup local share from Liquid Fuels tax allocation.

6) Invoices – District forwards signed invoices to BIS for processing and payment.

7) Close out – District closes out work order or part immediately after final invoice and acceptance in BMS2. This is critical to uncommit funding for future Work Order creation and increasing capacity of contracts.

### 1.10.4 Preparation of Safety Inspection Agreements

This section supplements Department's Publication 93 to provide guidelines for the preparation of safety inspection agreements to ensure Statewide compliance with Department policies.

#### **1.10.4.1** STANDARD SCOPES OF WORK FOR SAFETY INSPECTION AGREEMENTS

Standard scopes of work have been established for various types of safety inspection agreements and are contained in the following appendices:

- Appendix IP 01–F General Scope of Work - Safety Inspection of State and Local Bridges
- Appendix IP 01-G General Scope of Work – Safety Inspection of State and Local Tunnels
- Appendix IP 01–H General Scope of Work - Underwater Inspection of Bridges

Utilize these standard scopes of work as the basis for safety inspection engineering agreements whenever possible. Electronic versions of these standard scopes of work are available from the BIS.

The Scope of Work (SOW) for all the inspection types indicated above, except for tunnels, requires draft inspection reports to be submitted to the Department within 30 days from the date of the field inspection. The SOW for tunnels indicates the draft inspection report is to be submitted to the Department within 60 days of the completion of the field inspection. Particular emphasis needs to be placed on the draft inspection report to facilitate review and comment prior to acceptance of the final inspection report. This also ensures that BMS2 is updated within timeframes specified by NBIS/NTIS. The timeframe to update BMS2 for all State and local bridges is 3 months from the date of the field inspection. The timeframe to update BMS2 for all tunnels is 3 months after the completion of field inspection.

#### **1.10.4.2** GUIDELINES FOR PREPARATION OF SAFETY INSPECTION AGREEMENTS

To assist the Districts, Guidelines for Preparation of Safety Inspection Agreements are contained in Appendix IP 01-J.

## 1.11 FUNDING CATEGORIES FOR SAFETY INSPECTION

Funding categories have been established for some ongoing safety inspection work, based upon Federal Aid agreements with FHWA and on Department fiscal management requirements. The Multi-Modal Project Management System Numbers (MPMS), Federal Project Numbers (FPN) and Work Breakdown Structures (WBS), as shown in Table IP 1.11-1 (below) are to be utilized for in-house staff and inspection agreements.

All Inspection Agreements shall use General Ledger (**GL**) Account #**6341100**.

The need to include Fiscal Year (FY) in coding of the WBS element number has been eliminated, and this practice should no longer be used. WBS elements can only be used on one (1) MPMS number and therefore should be used on multiple agreements to accommodate WBS creation in the future.

**Table IP 1.11-1 Multi-Modal Project Management System Numbers (MPMS), Federal Project Numbers (FPN) and Work Breakdown Structures (WBS) for Inspection Work**

| Description | MPMS | FPN | WBS |
|---|---|---|---|
| State DOT Owned Structure Inspection (Non-NBIS Length) | 114961 | N/A | 0IN**dd**N09**###**-**unit**-611 |
| State DOT Owned Structure Inspection (NBIS Length) | 114961 | N/A | 0IN**dd**N09**###**-**unit**-311 |
| Locally Owned Structure Inspections (NBIS Length) | 114973 | NBIS-080-Y240 | 0IN**dd**N09**###**-**unit**-315 |
| State DOT Owned Sign Structure Inspection | 115813 | SIGN-070-Y240 | 0IN**dd**S09**###**-**unit**-383 |
| State DOT Owned Retaining Wall Inspection | 115813 | WALL-070-Y240 | 0IN**dd**W09**###**-**unit**-383 |
| State DOT Owned High Mast Light Inspection | 115813 | WALL-070-Y240 | 0IN**dd**H09**###**-**unit**-383 |

**LEGEND for WBS Element Coding Key:**

dd = District/Agency where work is performed (2 digits):
dd = District Number; 20 = DCNR; 21 = Other State Agency (OSA); 49 = State Wide

### = Corresponds to Work Order, Part or Department staff (3 digits):
### = Work Order # or Part Number # (ex: 001) or “111” for Department Staff

unit = Funding Code for District completing the inspection (4 digits) for Routine Inspections or for Central Office (4954) managed specialized inspections (i.e. Underwater, NSTM, etc.)

Note: For Federal definition of NBIS, See Publication 100A, Item 5E01, NBIS Bridge Length.

## 1.12 STATEWIDE BRIDGE INSPECTION CRANE PROGRAM

### 1.12.1 Purpose of Bridge Inspection Crane Program

The Department maintains a Statewide fleet of under-bridge inspection cranes to assist with the inspection and maintenance of PA bridges. The main purposes of this inspection crane program are:

- Better bridge inspection information – by providing inspectors easy access to remote portions of bridges that are difficult to access by climbing, more hands-on inspection is possible, providing more complete and accurate data.
- Improved emergency response – the cranes provide instant access to remote portions of the bridge allowing the inspectors and engineers better and timelier data. The cranes also provide access for persons who could not climb the structure.
- Improved safety for inspectors – the cranes provide a secure platform allowing inspectors to workhands free and avoid difficult climbing
- Cost savings – by reducing the time needed for inspectors to reach portions of the bridge and also avoid the erection of costly temporary scaffolding or platforms, significant time and costs savings are realized.
- Improved bridge maintenance – improved access is also available for in-house bridge maintenance activities, affording the opportunity to provide timely maintenance or repairs and avoid costly

scaffolding or rigging.

The Statewide bridge inspection crane program is organized to share crane equipment on a regional basis to ensure that bridge inspection and maintenance needs throughout the State are met efficiently and in a timely manner.

### 1.12.2 Statewide Organization and Operations

The Department under-bridge inspection crane fleet consists of 6 truck-mounted cranes organized in a Statewide program to share crane equipment on a regional basis.

#### 1.12.2.1 CRANE DEPLOYMENT

The Department's under-bridge crane fleet consists of six truck-mounted cranes with rotating and articulated arms to carry inspectors from the deck to the underside of the bridges. The six cranes are deployed on a regional basis to better serve the Statewide bridge needs and the individual Districts' inspection efforts. The State is roughly split into East and West along Engineering District boundaries and each half has three cranes. The individual cranes are assigned to a Home District and each Home District (except District 11-0) has a sharing District to which it is primarily responsible for crane services. The deployment of the Statewide fleet is shown in Table IP 1.12.2.1-1.

**Table IP 1.12.2.1-1 Deployment of the Department's Statewide Bridge Inspection Crane Fleet**

| Region | Primary Districts | Home District | Crane # | Crane Details |
|---|---|---|---|---|
| EAST | 4-0 & 5-0 | 5-0 | # 9 | Aspen Aerial<br>50' reach 3 person bucket GVW = 64,800 lbs. |
| | 2-0 & 3-0 | 3-0 | # 5 | Aspen Aerial<br>62' reach 3 person bucket GVW = 69,500 lbs. |
| | 6-0 & 8-0 | 8-0 | # 6 | Aspen Aerial<br>62' reach 3 person bucket GVW = 66,300 lbs. |
| WEST | 1-0 & 10-0 | 10-0 | # 10 | Aspen Aerial<br>55' reach 3 person bucket GVW = 68,000 lbs. |
| | 11-0 | 11-0 | # 13 | Aspen Aerial<br>62' reach 3 person bucket GVW = 64,500 lbs. |
| | 9-0 & 12-0 | 12-0 | # 14 | Aspen Aerial<br>30' reach 3 person bucket GVW = 36,800 lbs. |

#### 1.12.2.2 MANAGEMENT AND STAFFING FOR CRANES

- MANAGEMENT: Home Districts are responsible for overseeing the day-to-day operations of the crane assigned to that region, including full-time crane operators, maintenance, transport, inspection equipment, etc. This management role is generally assigned to the Assistant District Bridge Engineer for Inspection.
- STATEWIDE COORDINATION: The BIS is available to assist with the coordination of the overall Statewide crane inspection efforts among all the Districts.
- PERMANENT CRANE OPERATORS: Each Home District shall provide two permanent crane operators from the Bridge Inspection Crane Technician series from PA Civil Service to staff the Home District crane.
  - Lead Operator – Classification: Bridge Inspection Crane Technician Supervisor (BICTS)
  - Assistant Operator – Classification: Bridge Inspection Crane Technician (BICT)

  The permanent crane operators have the primary responsibility for the transport and operation of the crane, traffic setups, and bridge inspection duties.
- BACKUP CRANE OPERATORS:
  - Each Home District is responsible to have a qualified backup crane operator available to the

inspection program.
- o Each Home District is encouraged to have 2 persons from District or County staff trained and qualified to operate the bucket of the crane.

▪ BRIDGE INSPECTORS: Each District is responsible to provide certified bridge safety inspector(s) to take the technical lead and responsibility for the inspection. The BICT and BICTS may assist in the inspection. While not required, BICTs are encouraged to get their bridge safety inspection certification.

**1.12.2.3** SAFETY AND MAINTENANCE OF TRAFFIC

The overall crew and public safety at the bridge site, including crane operations and traffic considerations, shall be the responsibility of the Lead Crane Operator. The Lead Operator shall have the authority to direct the maintenance and protection of traffic (MPT) set up and the inspection operations at the bridge site to ensure its safety. All traffic lanes below the crane bucket shall be closed during inspection operations to prevent objects or debris from falling onto traffic.

When additional equipment or staffing is needed, the District in which the inspection is being performed is responsible to obtain and schedule it to coincide with the crane operations.

Each District is responsible to see that its bridge site is properly prepared for the bridge crane and all pertinent safety plans are in place. For example, tree branches that may interfere with the crane deployment should be removed and electrical lines de-powered prior to the crane inspection for safer and more efficient operation.

**1.12.2.4** CRANE SCHEDULING

The season of operation for the cranes is typically from mid-March to mid-December of each year. This annual schedule avoids severe winter weather that can affect the safety and efficiency of the operation. This scheduled downtime also allows for the necessary annual crane major maintenance inspection and service. The Department tries to maintain one operational crane at all times for emergencies.

The Districts are to incorporate local bridge inspection needs into their request when it is feasible and reasonable.

An important aspect of the scheduling is the sharing of the cranes with Districts outside their primary region. The cranes have varying capabilities and each District will have special inspection needs so the cranes must be shared to address PA's critical inspection needs. The cranes were purchased and deployed on a Statewide basis and should not be considered as District cranes. Districts should also plan key bridge maintenance activities where the use of the cranes is advantageous.

Inspection staff and crane operators shall meet to review the various District requests and set a schedule to best accommodate them. If during this review the need for one of the State's other cranes is identified, the District should contact BIS for scheduling assistance.

If a bridge emergency arises after the annual crane schedule is issued, contact the District Bridge Engineer in the Home District for your region. If the Home District crane is not available or not suited for the emergency, the Home District will assist in obtaining a crane. BIS can also assist in emergencies.

**1.12.2.5** CRANE USAGE REPORTING

Each Home District is to submit a monthly crane usage report to BIS who will then compile a Statewide crane usage report.

### 1.12.3 Crane Maintenance and Repairs

Each Home District will be responsible for routine maintenance on the bridge inspection cranes. Each Home District will be also responsible for the maintenance and replacement of the support van and equipment.

Major crane repairs (costing more than $5,000), annual crane inspections, major rebuilds, and replacements will be part of the Statewide equipment budget.

## 1.13 DRONE USE FOR BRIDGE INSPECTION

Unmanned Aircraft Systems (UASs), commonly referred to as drones, provide an opportunity to assist in collection of bridge inspection data. Drones can provide close-up visual inspection of areas which may be inaccessible without costly traffic control or access equipment (snooper or lift). They can also assist in accessing areas which are otherwise inaccessible possibly due to flooding or close proximity of a railroad.

Drones can be utilized to obtain close up visual inspection as determined by the District Bridge Engineer. This visual observation from the drone can be used to identify areas of defects which require physical inspection methods such as sounding to define the extent and severity of the defect. While drones can be utilized during NSTM inspections, they cannot replace the requirement of hands on for all NSTM components. Any time a drone is utilized for bridge inspection, a certified team leader shall be on site at the bridge for the duration of the flight.

All drone use shall be in accordance with the latest version of the Department's Unmanned Aircraft System (UAS) Policy, Publication 832. The Department's policy includes references to State and federal laws governing operation and use of UASs that must be adhered to at all times.

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## 2.1 GENERAL REQUIREMENTS

The requirements of this manual apply to all bridges and structures located on or over public roads in PA, as defined in IP 1.5.

### 2.1.1 Department Organization for Bridge Safety Inspection

DEPARTMENT OF TRANSPORTATION: §650.313 of the NBIS (§650.513 of the NTIS) requires that each State provide an inspection organization capable of performing inspections, preparing reports, and determining ratings in accordance with NBIS/NTIS. In PA, the Department of Transportation was named and empowered to perform these duties as necessary for all public road bridges greater than 20' in length through Act 44 of 1988.

BRIDGE INSPECTION SECTION: The Department assigned the responsibilities for overseeing and managing these inspections to the Bridge Inspection Section (BIS) in the Bureau of Bridge. BIS also has the responsibility to collect, maintain and report the required data to the Department and ultimately, the FHWA. To perform these duties, BIS is to maintain an adequate and qualified staff and an effective Bridge Management System (BMS2).

DISTRICT BRIDGE UNIT: In each of the Department's Engineering districts, the District Bridge Unit is responsible for inspecting and/or directing the inspection of the bridges and structures in its area. The following guidance for staffing key functional positions in the Bridge Unit is provided:

- District Bridge Engineer: Must be knowledgeable of inspection policies and procedures. Responsible for reviewing bridges in critical condition and handling emergency situations involving all structures within their District.
- Assistant District Bridge Engineer for Inspection: Must be qualified to be the individual in charge of the NBIS inspection program, and the NTIS inspection program if tunnels are present within their District. Supervises or manages the day-to-day operations of the District inspection efforts. For Home Districts, this person is designated as the supervisor/manager for the crane operation.
- Bridge Inspection Teams: Each District Bridge Unit shall have qualified, certified, and adequately equipped inspection teams comprised of either full time PennDOT employees or consultants.
- Load Rating Engineer: Each District shall have qualified engineer(s) to perform or review bridge analyses and ratings for inspections. Load ratings shall be performed by or under the direct supervision of a registered Professional Engineer.
- APRAS Review Engineer: Each District shall have qualified engineer(s) to use APRAS and perform needed reviews for heavy hauling permit reviews.
- BMS2/BMS3 Coordinator: Each District shall designate a staff member to ensure that inspection data is entered into the BMS3 Inspection Module and BMS2 in a timely and correct manner.
- Bridge Inspection Crane Operators:
  - Each Home District shall have two permanent crane operators in the BICT series assigned to the crane inspection program (see IP 1.12).
  - Each Home District shall have one back-up operator certified as a Crane Operator at the BI-level.
  - Each Home District shall have two back-up operators (from inspection staff or maintenance) certified as Bucket Operators [equipment certification level BB] to assist with the crane (see IP 1.12).
- Local Bridge Inspection Coordinator: Each District shall designate a staff person to coordinate bridge inspections with local bridge owners to ensure PA's compliance with NBIS/NTIS.

Depending upon District organization, resources, and workload, some of the above positions may not warrant a full-time staff person in an individual District, while others may require more than the minimum listed. For example, a District experiencing a large number of heavy overload permits applications may need more staffing in the bridge rating and/or APRAS review areas. However, for each functional area above, the Districts are to designate a lead person. Additional resources may be added through in-house staff or consultants.

By January 31 of each year, the Districts are to submit a staffing report for their inspection efforts in their District to the Assistant Chief Bridge Engineer - Inspection. The report should include an organization chart and staffing list. The staffing list is to include any required certification (such as PE licensure, inspection certification (including training and experience), CDL, etc.).

### 2.1.2 Inspection Organizations in Other PA State Agencies

Other PA State agencies that maintain an independent inspection organization, such as the Turnpike and DCNR, are to demonstrate to the Department, on an annual basis, that their organization meets the NBIS/NTIS requirements for an inspection organization as applicable.

### 2.1.3 Qualifications for Program Manager and Safety Inspectors

#### 2.1.3.1 PROGRAM MANAGER

The Program Manager (PM) must meet the minimum qualifications as described in the NBIS §650.309 and NTIS §650.509. The Assistant Chief Bridge Engineer - Inspection will serve as the overall PM for bridge and tunnel inspections. The PM is responsible for completing the training requirements for a team leader in Section IP 7.4.2 for bridges as well as FHWA approved tunnel inspection training class and refresher training every sixty (60) months for both tunnels and bridges. The PM must maintain supporting documentation of completed trainings. In addition, the PM needs to determine when inspection team leaders must meet additional requirements for complex tunnels.

#### 2.1.3.2 BRIDGE/CULVERT SAFETY INSPECTORS

For State and locally owned bridges, all inspection personnel must be current with their training as described in IP Chapter 7. Department Team members shall become certified within 12 months of their hire date. If certification is not achieved within that time period, the individual will not be allowed to serve as a Department team member until certification is achieved. Uncertified Department team members must be accompanied by a qualified team leader. Non-department team members must obtain certification before serving as an official team member. To ensure proper certification within BMS2, users must enter records with accurate inspection dates as follows:

- Bridge Team Leader (NBIS Length)
  - NHI-Basic Course (130055 or equivalent) or NHI-PE Basic Course (130056)
  - NHI-Refresher Course (130053 or equivalent) if Basic Course has expired
  - Bridge Inspection Team Leader and Qualifications
  - Inspector ID Number (Assigned by Central Office Bridge Inspection Section)
- NSTM Team Leader
  - Certifications listed under Team Leader above
  - NHI-NSTM Inspections (130078 or equivalent)
- Underwater Diver
  - Underwater Diver Certification and Qualifications
  - Corresponding NHI Course from Qualifications

Individual bridge inspectors are required to log their training within the User Preference screen in BMS2. If requested, users will provide copies of their certifications to confirm the information recorded within BMS2 is accurate. The Department's Assistant Chief Bridge Engineer - Inspection will make the final determination of an individual inspector's qualifications.

#### 2.1.3.3 TUNNEL SAFETY INSPECTORS

Tunnel inspection team leaders shall meet the minimum qualifications as described in NTIS §650.509. General tunnel inspectors shall have the training and expertise to inspect tunnels. It is recommended that all tunnel inspectors be Nationally Certified Tunnel Inspectors (NCTIs), however, it is not a requirement. The TOMIE Manual, Chapter 4 outlines additional recommended team members for tunnel inspections and their minimum qualifications including discipline specific specialists and specialty contractors. All tunnel safety inspectors are responsible for maintaining supporting documentation of completed trainings and qualifications. A registry of qualified team leaders and NCTIs is maintained by the Department's PM (see IP 2.1.3.1). To ensure proper certification within BMS2, users must enter records with accurate expiration dates as follows:

- Tunnel Team Leader
  - NHI-Tunnel Basic Course (130110)
  - NHI-Tunnel Refresher Course (130125) if Tunnel Basic Course has expired
  - Tunnel Inspection Team Leader and Qualifications
  - Inspector ID Number (Assigned by Central Office Bridge Inspection Section)

## 2.2 INSPECTION PROCEDURES

### 2.2.1 Bridge and Structure Inventory and Inspection Records

Records containing the inventory and condition information for bridges and structures are a vital key to managing these critical assets and assuring public safety. Accordingly, inventory and inspection records are to be prepared and maintained in accordance with IP 5 and IP 8.

### 2.2.2 Load Rating and Posting

Each bridge or structure carrying vehicular traffic requiring inspection under this Manual shall be rated to determine its safe load carrying capacity in accordance with Part IP, Chapter 3. For tunnel ratings also reference the TOMIE Manual, Section 5.4. Load ratings must be performed by, or under the direct supervision of, a registered professional engineer.

Re–rating of a bridge is required when changes in its member conditions or its loadings occur. This re-rating must be completed and entered into BMS2 as soon as practical. The new rating must be approved in BMS2 within 2 months of the inspection end date where such changes were noted or other notification of such changes. The need for a re-rating analysis shall be evaluated during each inspection. Considerations for re-rating are outlined in Publication 100A in Section 2.5.6.1 and on the Load Ratings Tab of BMS2.

Bridges that have been closed for rehabilitation must be re-rated and have load rating data entered into BMS2 before the bridge is re-opened to the public. For phased re-construction, each portion of the bridge is to be load-rated before going back into service.

All new bridges must be load rated and must have load rating data entered into BMS2 before the bridge is opened to the public. The review of bridge load capacity ratings, done in conjunction with safety inspections, is to be documented in BMS2.

If it is determined that the maximum legal load configurations exceed the load allowed at the Operating Rating level (see IP 3.2.2), then the structure must be posted for load restriction in accordance with Part IP, Chapter 4 of this Manual and Chapter 6 of the MBE.

### 2.2.3 Identification of Bridge Needs

One of the functions of the bridge (and structure) inspection program is to identify the needs of bridges for repairs, maintenance, preservation, reconstruction and replacement. Bridge owners need this information to respond to those critical findings warranting immediate attention as defined in NBIS §650.305 and discussed in NBIS §650.313 and NTIS §650.513 and for the long-term management of these critical infrastructure assets. Critical findings and associated actions are discussed in further detail in IP 2.13 and IP 2.14. The FHWA requires the major improvement needs for NBIS bridges for nation-wide planning. The BMS2 will use these major improvement needs and identified maintenance needs to predict future costs to achieve a desired level of service for PA bridges and structures.

Identify needed maintenance items (e.g., on-demand repairs, preventative maintenance, preservation) for each bridge using the standard list of Maintenance Activities on the BMS3 Maintenance page. Major improvement needs (e.g., rehabilitation, replacement) can be recorded as per the instructions for the 3A and 3B Screens in the BMS2 Coding Manual, Publication 100A.

### 2.2.4 Use of BMS3 for Electronic Collection of Bridge Inspection Information

The Department developed an application to enable all inspectors to collect bridge inspection data electronically at the bridge site and to transfer that digital data to the BMS2 Oracle database. The two main advantages the Department gained from this software are:

- Higher quality data through on-line edit checks and elimination of redundant data entry to BMS2
- Time savings for field inspectors

The application is named BMS3 and its purpose is to electronically collect bridge safety inspection information. BMS3 collects all field inspection data for all inspection types. The screen layout is patterned after the D-450 series inspection forms. Both the BMS2 Items and the narrative comments supporting those ratings are stored in the BMS2 database. BMS3 also prints an inspection report.

The Department's BMS3 Inspection Module is to be used for all highway bridge inspections.

### 2.2.5 Identification of Bridge Utility Occupancies

On Department bridges greater than 8' in length, all bridge occupancies are to be identified and recorded on the Features (FT) Screen of BMS2. This information allows the Districts to identify the responsible party when a problem or deficiency is noted on the utility itself or its supports. During Routine Inspections, bridge inspectors do not need to inspect the utility facility, but should note if mounting hardware, utility joints, utility occupancies, etc. present a hazard to the public or are detrimental to the bridge condition.

Bridge deck de-icing systems and traffic monitoring detection systems, while not strictly a utility occupancy, should also be recorded on the BMS2 FT screen.

### 2.2.6 Bridge and Structure Information for APRAS

The Districts are to maintain in BMS2 accurate and up-to-date load capacity and clearance information for APRAS for all bridges and structures that carry or cross over State Routes or that are owned by the Department.

For bridges that carry State Routes or are owned by the Department, the bridge load capacity data and clearance information are required in the APRAS screens in BMS2. For locally owned bridges on State-owned Roads (LOBSTORs), the Districts are responsible to gather and maintain the APRAS data (BMS2's SC, SL and SS Screens) and RMS location references (on BMS2's 5C and 6C screens) to enable the automated processing of Heavy Hauling Permits.

The Districts are reminded that the Department should not be issuing permits to travel over LOBSTORs without the consent of the owner. Blanket authority to allow the Department to issue permits over LOBSTORs when the Department follows its permit review procedures may be granted by the Owner.

For bridges that carry local roads (or Turnpike) over State Routes, the Districts are responsible to gather and maintain the APRAS data (except bridge load capacity information) and RMS location references (on BMS2's 5C and 6C screens) to enable the automated processing of Heavy Hauling Permits.

For non-highway bridges or structures over State Routes, the Districts are responsible to gather and maintain the APRAS clearance data and RMS location references (on BMS2's 5C and 6C screens), regardless of the structure ownership, to enable the automated processing of Heavy Hauling Permits.

### 2.2.7 Functional Systems in Tunnels

Functional systems present in tunnels may include mechanical, electrical/lighting, and/or fire/life safety/security systems as identified in the SNTI. All systems shall be visually inspected for general condition and operation of the system at all levels during each routine inspection. Past maintenance and testing records shall be reviewed during each routine inspection to ensure the system is being tested and maintained at the interval specified in the tunnel specific inspection procedure document. The team leader for the inspection shall oversee some functional systems testing as part of an in-depth inspection with the interval specified in the tunnel specific inspection procedure document (see IP 8.3.9).

When assessing the Condition State of a functional system, the design capacity and design intent of the system shall be taken into account. For example, when assessing a ventilation system with one non-functional fan, a condition state of 4 would be warranted if all fans are required by design, whereas a lower value could be justified for a system which is overdesigned. See TOMIE Chapter 4, for additional guidance for inspection of tunnel elements and systems.

Tunnel inspectors shall provide sufficient justification within the inspection report to back up the condition state coding of all functional systems assessed during the inspection. Documentation shall include the level of assessment (visual observation, system operation, testing results, etc.) utilized and specific condition (defects/deficiencies) noted in determining if the functional system is experiencing isolated breakdowns without compromising effectiveness (CS2), when a system's condition is compromising effectiveness (CS3) and when the system is in critical condition (CS4).

### 2.2.8 Required Photographs and Sketches

PHOTOGRAPHS: Where an adverse bridge condition such as section loss due to corrosion in a steel member is present and the extent of the defect has been used in the load rating, provide photographs of the member condition at the time the load rating was performed and of the current condition, arranged side-by-side, to allow for condition comparison and the determination of the need for re-rating. The photographs should have sufficient clarity, have reference to scale as appropriate, and be labeled appropriately to indicate the date taken and the condition information intended to convey in the inspection report. Photographs should be taken after cleaning when cleaning is required to establish the remaining section of the member.

SKETCHES: Inspectors shall sufficiently document, with sketches, all defects in structure members. For example, section loss due to corrosion in a bridge member shall be documented to show the specific location and geometrics of the deficiency and the numerically-measured remaining section, including thickness, to a level of completeness that facilitates the load rating of a bridge and/or the determination of the need for re-rating. The sketches shall be prepared to accurately represent the structure member.

### 2.2.9 Basis of Changed Condition Ratings, Changed Characterization of Conditions, and Assessment of Changed Conditions

BASIS OF CHANGED CONDITION RATINGS: When upgraded or downgraded condition ratings and/or conditions are established during an inspection cycle, the basis for the change should be adequately documented in the inspection report. For example, if a superstructure condition is downgraded from Satisfactory to Poor, discussion on the cause of the change shall be documented in the inspection report. At a minimum, if a condition rating change for fields appearing in the key field comparison within BMS2 is changed during an inspection, it should be documented in Field 2A02 with an explanation of why the rating changed.

CHANGED CHARACTERIZATION OF CONDITIONS: When the characterization of a condition is changed in an inspection report from that of the previous report, the basis of the change should be documented in the inspection report. For example, if a component was characterized as "poor to serious condition" in the previous inspection report, there must be documented reasoning if the condition characterization in a subsequent inspection report is changed to "poor condition."

It could be that the current inspector disagrees with a past characterization of a condition, or the change may be the outcome of a completed repair or a result of ongoing deterioration of a condition or element. Regardless, appropriate documentation for the change must be provided.

ASSESSMENT OF CHANGED CONDITIONS: When a bridge or structure member condition change is identified in a bridge inspection report, an assessment of the condition change shall be provided in the inspection report. For example, if the web or transverse stiffener of a steel girder is documented as irregular or bowed, the inspection report shall contain an assessment about the likely cause and consequence of the change.

### 2.2.10 Weathering Steel

#### 2.2.10.1 GENERAL

Appendix IP 02-I, *Uncoated Weathering Steel - Bridge Safety Inspection and Maintenance Manual,* provides bridge safety inspection procedures and requirements for uncoated weathering steel bridges. These procedures and requirements shall be used in conjunction with NBIS bridge safety inspections.

**2.2.10.2** UWS SUBJECT MATTER EXPERTS

The Bureau of Bridge, Bridge Inspection Section shall assign a staff person to serve as a Statewide UWS Subject Matter Expert (UWS- SME). The Bureau of Bridge UWS-SME shall be well-versed in UWS bridge safety inspection and maintenance knowledge, procedures, and operations. The Bureau of Bridge UWS-SME shall also facilitate UWS training and provide UWS technical support to Districts and other bridge owners.

Each District shall assign a staff person, preferably District staff but can also be supplemental support staff, to serve as its UWS-SME. The District UWS-SME shall be well-versed in UWS bridge safety inspection and maintenance knowledge, procedures, and operations. The District UWS-SME shall be knowledgeable of the inventory and conditions of each UWS bridge within the District, including District and locally-owned bridges. The District UWS-SME shall also be responsible for coordinating needed UWS maintenance actions and monitoring of this work for completion within the timeframes established in the Inspection – Maintenance Section of BMS.

Local bridge owners are encouraged to establish their own UWS-SME. Alternatively, local owners may request assistance from their respective District UWS-SME with developing UWS bridge safety inspection procedures and bridge maintenance actions.

## 2.3 TYPES OF BRIDGE SAFETY INSPECTIONS

As described in Section 4.2.3 of the MBE, there are seven general types of bridge inspections: Initial, Routine, Damage, In-Depth, Special, Underwater and Fracture Critical Inspections. FHWA revised the terminology from "Fracture Critical" to "nonredundant steel tension member (NSTM)" in the 2022 NBIS. Publication 238 has been updated to reflect this change. The 2022 NBIS also added two additional inspection types for compliance which are Service Inspections and Scour Monitoring Inspections. The TOMIE Manual describes five general types of inspections for tunnels which are in line with the first five listed above.

The scope, intensity, and interval of various types of bridge safety inspections are discussed here to provide a better understanding of the purpose and use of each inspection type and to assist in the development of scope of inspection work for individual inspections. When a limited scope inspection is performed (Underwater, NSTM, In-Depth, Special, Damage, Scour Monitoring, Problem Area, Service), the scope of that inspection shall be documented in BMS2 field B.IE.11 (Inspection Note). This field shall also be used to document bridge-specific inspection procedures as applicable and as required for In-Depth inspections. Future Inspection Scope Comment Type 481 on the Notes and Comments screen of BMS2 and the BMS3 Notes page shall be used to enter the scope/inspection procedures of the next scheduled inspection. The scope/inspection procedures are to be developed by the engineer-in-charge of the inspection contract or group of bridges (either consultant or District). The District Bridge Engineer must review and approve the scope of work and interval of reduced interval inspections to ensure compliance with this manual and for eligibility for federal reimbursement of inspection costs. These two fields should not be used for Routine or Initial Inspections since the scope of those inspection types is always well defined and includes the entire structure.

At least one Team Leader shall actively participate in the inspection at all times during each inspection of bridges/culverts greater than 20' length, tunnels, and sign structures over traffic regardless of inspection type. See IP 2.1.3 for Team Leader Qualifications.

An inspection event, particularly for large or deficient structures or structures with complex features, often requires that a variety of inspection types be performed, using a variety of methodologies. For example, an NSTM will routinely receive a hands-on inspection, while the remainder of the bridge may not. In another example, the underwater inspection of a particular structure may require that specific elements receive in-depth inspections, while other underwater elements may require only routine inspection. The following sections of this chapter describe each of the seven general types of inspections listed above (Initial, Routine, Damage, In-Depth, Special, NSTM, and Underwater), the two additional compliance inspection types identified by FHWA (Service and Scour Monitoring), and four additional inspection types used by PennDOT (Problem Area, Element, Quality Assurance, and Inventory Only). Each section provides a description of the inspection type along with the purpose, level of work effort or intensity, and interval.

The Department has developed a standard General Scope of Work for the Safety Inspection of State and Local Bridges and General Scope of Work for the Safety Inspection of State and Local Tunnels to be used as the basis for inspection agreements. These Scope of Work (SOW) documents are intended to provide the framework to cover the types of inspections listed above and allow the user to define additional special requirements and/or efforts. Refer to IP 1.10 for instructions on developing bridge inspection agreements.

Owners of bridge structures with complex features and other unique characteristics, including all tunnels, must have published inspection procedures and inspection team qualification requirements for the structure(s). Examples of complex features include:

- Cable-related members of suspension and cable-stayed structures, including cable anchorages.
- Redundancy/retrofit systems (e.g. catcher-beams).
- Pin hangers/Steel pinned connections.
- Steel box beams.
- Segmental concrete box beams.
- Fatigue prone details (E or E').
- Movable spans.
- Tied arch bridges.
- Other areas identified by the District Bridge Engineer to be critical.

For more information on tunnel specific inspection procedures, see Section IP 8.3.9. The procedures should cover the inspection interval, scheduling and requirements for In-Depth inspections, traffic control, special equipment, methods of access, and locations or specific components on the structure where hands-on inspection is required for each inspection. The requirements for any special inspection techniques such as NDT shall be identified including the locations where applicable. In addition, indicate all discipline specific specialists and specialty contractors as required. The bridge owner is to maintain a copy of the complex feature inspection plan/procedures with an electronic copy in BMS2 through the Documents link.

An effective plan must address inspection of each of the various systems or subsystems of the complex structure. For example, a movable bridge inspection must include the machinery, electrical, mechanical and hydraulic systems that are typically present, in addition to the structural load path components. The plan should address the inspection techniques, tests and measurements that should be applied. Health and safety considerations should also be evaluated. Coordinate with local and State police, fire departments, ambulatory and medical services in advance, particularly when dealing with confined space.

Within these types of inspections, the term "inspection type" is used for two other purposes:

- Primary Inspection Type (BMS2 Item 7A03) for inspection management
  - Created by the Department
  - Used to better describe the type and function of the inspection
  - See Table IP 2.3.6.6-1 and Table IP 2.3.6.6-2 for correlation between Item 7A03 and Compliance Inspection Types for NBIS and NTIS respectively
- Compliance Inspection Type (BMS2 Items 7A06-7A10)
  - Required by NBIS/NTIS
  - BMS2 data used for scheduling and monitoring of compliance of inspections required by NBIS/NTIS
  - See IP 2.3.6 for additional information

### 2.3.1 Initial Inspections

#### **2.3.1.1** DESCRIPTION OF INITIAL INSPECTIONS

An Initial Inspection is the first inspection of a new or existing structure, as it becomes part of the structure inventory. Additionally, major reconstruction of structures may also require an Initial Inspection to document more extensive modifications of the structure's type, size, or location. In general, widening, deck and superstructure replacements warrant an Initial Inspection. The Initial Inspection is to include an analytical determination of load carrying capacity. An Initial Inspection is also a Routine Inspection and is considered a Compliance Inspection for both NBIS and NTIS. This inspection type should not be used for the sole purpose of reviewing repairs to limited elements of the structure.

**2.3.1.2** PURPOSE OF INITIAL INSPECTIONS

The Initial Inspection is to verify the safety of a bridge, in accordance with the NBIS/NTIS and Department standards, before it is put into service or within the timeframe detailed in IP 2.3.1.4. The inspection serves to provide required inventory information of the "as-built" structure type, size, and location for BMS2 (and the NBI/NTI) and to document its structural and functional conditions by:

- Providing all Structure Inventory & Appraisal (SI&A) data required by Federal regulations along with all other data required by Department standards and the local owner.
- Determining baseline structural conditions. Clearance envelopes (for features carried and those intersected) and bridge waterway openings are to be documented at this time.
- Identifying and listing existing problems.
- Determining the need for establishing or revising a weight restriction on the bridge.
- Identifying and listing concerns of future conditions.
- Identifying maintenance needs, including preventative maintenance activities.
- Noting the existence of elements or members requiring special attention, such as NSTMs, fatigue-prone details, and underwater members.

Documents including, but not limited to, photographs, drawings (design, as-built and shop drawings), scour analysis, foundation information, hydrologic and hydraulic data are to be inserted into the bridge file and uploaded to BMS2 Documents link. Selected construction records (e.g., pile driving records, field changes, photos) may also be of great use in the future and should be included. Include maintenance records for existing bridges.

**2.3.1.3** INTENSITY OF INITIAL INSPECTIONS

The level of effort required to perform an Initial Inspection will vary according to the structure's type, size, design complexity, and location. An Initial Inspection is to be a close-up, hands-on inspection of all members of the structure to document the baseline conditions. Traffic control and special access equipment may be required.

For tunnels, Initial Inspections are to include all structural, civil, mechanical, electrical and lighting, fire and life safety, security, signs and protective systems.

**2.3.1.4** INTERVAL OF INITIAL INSPECTIONS

Best practice is for Initial Inspections to be performed for each structure after construction is essentially complete and before the bridge is put into service (or returned to service for bridges that have had a major reconstruction). However, at a minimum:

- Department and local bridges are to be inspected and inventoried within three (3) months of the bridge opening to traffic. Inspection reports must be accepted in BMS2 within three (3) months after the month when the field portion of the inspection is completed.
- For tunnels, the Initial Inspection shall take place after all construction is completed, after all functional systems are tested, but prior to opening the tunnel to traffic.

### 2.3.2 Routine Inspections

**2.3.2.1** DESCRIPTION OF ROUTINE INSPECTIONS

Routine Inspections are also known as "NBI" or "NTI" inspections because they update the condition and inventory information required for the NBI/NTI. Routine Inspections document the existing physical and functional conditions of the structure. The Routine inspection should pay particular attention to critical areas of the structure such as at or under deck joints and drains, at bearings, at splices, connections, etc. All changes to required inventory items that have occurred since the previous inspection are also to be documented. The written report will include appropriate photographs and recommendations for major improvements, maintenance needs (preservation, preventative maintenance or on-demand repairs), and follow-up inspections. Load capacity analyses are re-evaluated only if changes in structural conditions or pertinent site conditions have occurred since the previous analyses.

**2.3.2.2** PURPOSE OF ROUTINE INSPECTIONS

A Routine Inspection is to satisfy the data collection for the NBI/NTI record and be performed in accordance with the NBIS/NTIS and Department standards. Routine Inspections serve to document sufficient field observations, measurements and load ratings needed to:

- Determine the physical and functional condition of the structure.
- Identify changes from the previously recorded conditions.
- Determine the need for establishing or revising a weight restriction on the bridge.
- Determine improvement and maintenance needs.
- Ensure that the structure continues to satisfy present service and safety requirements.
- Identify trends and predict future life expectancy of components.

**2.3.2.3** INTENSITY OF ROUTINE INSPECTIONS

The level of scrutiny and effort required to perform a Routine Inspection will vary according to the structure's type, size, design complexity, existing conditions, and location. All Routine Bridge Inspections shall be performed in accordance with MBE Section 4.2 as supplemented by Section IE 4.2 of this document. For tunnels, Routine Inspections are intended to be comprehensive covering the structural, civil, mechanical, electrical and lighting, fire and life, safety, security, signs and protective systems. The scope of a routine inspection is consistent, regardless of inspection interval.

Generally, every element in a bridge does not require a hands-on inspection during each Routine Inspection to provide an acceptable level of assurance of the bridge's ongoing safety. The difficulty is that the areas not needing close-up scrutiny can be determined only after the entire bridge has been inspected and non-critical areas identified. Accordingly, to provide a reasonable level of confidence in the safety of the bridge, knowledge of the structure and good engineering judgment are necessary when considering those portions that will not receive the close-up scrutiny with each inspection.

The following guidance is offered when determining the level of scrutiny needed for adequate inspection of individual bridges: Areas/elements that may be more difficult to access but that warrant hands-on inspection in each Routine Inspection, include, but are not limited to:

- Load carrying members in Poor* condition.
- All NSTMs
- All redundancy retrofit systems (e.g., Catcher-beams) for NSTM details (pin hangers, etc.).
- Critical sections of controlling members on posted bridges.
- Scour critical substructure units.
- End regions of steel girders or beams under a deck joint.
- Cantilever portions of concrete piers or bents in Fair or lesser condition*.
- Ends of Prestressed concrete beams at continuity diaphragms.
- Precast concrete bridge barriers.
- Fascia beams of non-composite adjacent box beam bridges with open joints in the barrier.
- All elements and system components within any tunnel.
- Other areas determined by the District Bridge Engineer to be potentially critical.

* The condition of the bridge element noted above is based on the Condition Rating code list on Page 3-4 of Publication 100A. While the Publication 100A language is referring to the major NBI components (deck, superstructure, substructure, and culvert), the condition descriptions can also be applied to portions or elements thereof.

If hands-on scrutiny of an element is performed during a Routine Inspection, it should be so noted in the inspection report. An example of inspection notes might read: "P/S Beams in good condition with no cracks or spalling noted when a hands-on inspection was completed from a crane."

The application of these guidelines for intensity of inspection does not relieve the engineer-in-charge of the inspection from the responsibility to perform other hands-on inspection tasks and/or tests needed to ascertain the condition of the bridge and assure its safety.

Routine Inspections are generally conducted from the deck, ground and/or water levels, ladders and from permanent work platforms or walkways, if present. Inspection of underwater members of the substructure is generally limited to observations during periods of low flow and/or probing/sounding for evidence of local scour. Special equipment, such as under-bridge cranes, rigging, or staging may be a necessary or more practical means of accessing areas of the structure being inspected hands-on.

**2.3.2.4** INTERVAL OF ROUTINE INSPECTIONS

INSPECTION DATE AND INSPECTION INTERVAL: For the purpose of monitoring NBIS/NTIS compliance, FHWA measures Routine Inspection interval by the month and year only, not the exact date of inspection. The exact date of inspection (BMS2 Item 7A01) is still to be recorded but will not be used to check NBIS/NTIS compliance.

Routine Inspections are inspections that are regularly scheduled and of sufficient scope and intensity to provide the updated condition and inventory information required for the NBI/NTI. The maximum intervals of Routine Inspections and Special Inspections have been established for bridges in PA in Table IP 2.3.2.4-1 and Table IP 2.3.2.4-2.

NTI Routine Inspections shall be completed every two years unless authorized for an extended interval by the BIS. PennDOT must make a written request justifying the extended interval to FHWA for review and comment prior to implementation. Table IP 2.3.2.4-1 provides criteria which reduces the inspection interval. The target month for NTI inspections shall be based on the target date. NTIS allows the inspection to be performed two months before or two months after the target date month and still be considered on time. The target date for Routine NTI inspections should not be changed once established without coordination with BIS and FHWA.

BRIDGES THAT LOSE ELIGIBILITY FOR EXTENDED INTERVAL: For bridges that are scheduled for a 48-month Routine interval, which no longer qualify, the following procedure shall be followed:

- Last Routine inspection occurred within the last 24 months: Create a Damage inspection to initiate the change in inspection interval. Complete the next Routine inspection 24 months after the last Routine was completed. Bridge must remain on a 24-month interval for at least one more cycle.
- Last Routine inspection occurred more than 24 months ago: Create a Damage inspection to initiate the change in inspection interval.- Perform a Routine inspection within 30 days of the date of the Damage inspection to keep the bridge in compliance (Note, a Routine inspection may be completed in lieu of the Damage inspection to keep the bridge in compliance instead of completing a Damage Inspection and follow-up Routine Inspection). Bridge must remain on a 24-month interval for at least one more cycle.

REDUCED INTERVAL INSPECTIONS FOR BRIDGES: When the bridge conditions have deteriorated to the point where more frequent inspection, in addition to the 24-month NBI inspections, is needed to ensure public safety, a Special Inspection is to be required and scheduled. Tables IP 2.3.2.4-1 and IP 2.3.2.4-2 specify the reduced inspection interval based on a structure's condition. See IP 2.3.5 for additional information on Special Inspections.

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<table>
<tr><th colspan="4">Table IP 2.3.2.4-1 Intervals of Routine and Special Inspections for State and Local Owned Bridges > 20' Length</th></tr>
<tr><th colspan="2">7A09 Insp Interval (months)</th><th rowspan="2">Bridge Description</th><th rowspan="2">Comments</th></tr>
<tr><th>R</th><th>SP</th></tr>
<tr><td>48</td><td>NR</td><td>Bridge must meet all the following criteria:<br>▪ State or PTC owned (5A21= 01 or 31)<br>▪ State or PTC maintained (5A20 = 01 or 31)<br>▪ Deck (1A01), Superstructure (1A04), Substructure (1A02), Culvert (1A03), Scour (1A13), Channel (1A05), and Channel Protection (1A05b) Condition Rating ≥ 6 or N<br>▪ SCBI (4A08) = 4, 5, 8, 9 or N<br>▪ Known Foundation Type (IN13) ≠ P or X when Service Type Under (5A18) contains a waterway (i.e., 5 thru 9)<br>▪ Min. Vert. Underclearance (4A17) ≥ 14.5'<br>▪ Min Vert Overclearance (4A15) ≥ 14.5'<br>▪ Bridge with no NSTMs (6A44 ≥ 5)<br>▪ Structure Type is not Adjacent Non-Composite Box Beams (6A26-6A29 ≠ 42107)<br>▪ Posting Status (VP02) = A (Open, no restrictions)<br>▪ Load Rating ≥ State Legal Load:<br>HS20 IR ≥ RF 1.0 (≥ 36 tons)<br>ML80 IR ≥ RF 1.0 (≥ 36.6 tons)<br>TK527 IR ≥ RF 1.0 (≥ 40 tons)<br>PHL-93 IR ≥ RF 1.0<br>▪ For a new, rehabilitated, or structurally modified bridge: an Initial and one Routine Inspection (24 months later) must be completed<br>▪ Bridge does not have any active or deferred Priority 0 or 1 Maintenance Items<br>▪ Steel Bridges without E or E' Fatigue Details (IF07 = N)<br>▪ Structure Material Type is Steel or Concrete (6A26 Main ≠ 5,6,7,9 and 6A26 Approach ≠ 5,6,7,9)<br>▪ Structure Config Exclusions (6A29 Main ≠ 11 thru 21 or 26 and 6A29 Approach ≠11 thru 21 or 26)<br>▪ Scour Vulnerability (IU29) = A, B, N or AB-T[1]<br>▪ Substructure Material Exclusions (IN12 ≠ 6, 9, 21, 24)<br>▪ Routine Permit Loads (Item B.LR.08) = A or N[2]</td><td>Bridges meeting the extended NBI inspection interval of 48 months will be identified in BMS2 on the Ratings and Schedule Screen (7A19 = Yes through May 2024; 7A21 = Yes after May 2024).<br><br>In addition, each District will approve the use of extended NBI inspection interval in BMS2 on the Ratings and Schedule Screen (7A20 = Yes).<br><br>This interval is not applicable for tunnels.</td></tr>
<tr><td rowspan="3">24</td><td rowspan="3">NR</td><td>Highway bridges without restrictions or conditions described below</td><td></td></tr>
<tr><td>Closed highway bridges</td><td>See IP 2.7 for inspection requirements</td></tr>
<tr><td>Non-highway bridges and structures over State Routes</td><td>See IP 2.10 for inspection requirements</td></tr>
</table>

Table 1 of 2

Note: R = Routine Inspection
SP = Special Inspection
NR = Not Required

[1]AB-T is a temporary coding and will be removed after one inspection cycle is completed on all bridges.

[2]This criteria is based on SNBI coding item B.LR.08. A coding field will be added to BMS2 in the future to correspond with this item. All NBIS length bridges in PA meet the criteria of coding of A or N.

**Table IP 2.3.2.4-1 Intervals of Routine and Special Inspections for State and Local Owned Bridges > 20’ Length**

| 7A09 Insp Interval (months) R | 7A09 Insp Interval (months) SP | Bridge Description | Comments |
|---|---|---|---|
| **24** | **12** | Highway bridges with weight restrictions (e.g., posted weight limit and/or restricted to One Truck at a Time) (VP02 = P or R) | Special Inspections of critical areas only of a bridge are to be used to meet the reduced interval between Routine Inspections.<br><br>When a reduced inspection interval is warranted, the inspection report is to list the justification, scope, intensity and interval for the next Special Inspection in the recommendation section.<br><br>For Highway Tunnels Special Inspections shall be determined by owner and District Bridge Engineer and outlined in the Inspection Procedure document located in BMS2.<br><br>For any structure with a Special Inspection at a reduced interval that requires divers to assess the condition, an underwater inspection must also be scheduled at the same interval to ensure divers are scheduled for the work. |
| | | Highway bridges with a Condition Rating = 3 for Channel (1A05) or Channel Protection (1A05b) | |
| | | Highway bridges with only one of the following Condition Ratings = 3: Deck (1A01), Superstructure (1A04), or Substructure (1A02) | |
| | | Highway bridges with a Condition Rating = 4 for NSTM[3] (1A15) | |
| | | Highway bridges of metal arch culvert construction (6A26 = 1 or 7 AND 6A29 = 20, 30, 32, 33, or 35) with a Condition Rating = 4 for Superstructure (1A04), Culvert (1A03) or Substructure (1A02) | |
| | | Highway bridges with an open steel grid deck (6A38 = 06) with a Condition Rating = 4 for Deck (1A01) or Superstructure (1A04) | |
| | | Highway bridges of masonry arch construction with a Condition Rating ≤ 4 for Superstructure (1A04), Culvert (1A03) or Substructure (1A02). See IP 2.5.3 and IP 2.5.4 for more instructions. | |
| **24** | **6** | Highway bridges with an SCBI (4A08) = 6 | |
| | | Any temporary structure or other temporary conditions exist (VP02 = D or E or 5E03 = T) | |
| | | Highway bridges with an outstanding Priority 1 Maint Item | |
| **12** | **NR** | Highway bridges with a Condition Rating = 3 for two or more of the following components: Superstructure (1A04), Substructure (1A02), or Deck (1A01)[4] | |
| | | Highway bridges with a Condition Rating = 3 for Culvert (1A03) or Scour (1A13) | |
| **12** | **6** | Highway bridges with a Condition Rating = 3 for NSTM[3] (1A15) | |
| | | Highway bridges of metal arch culvert construction (6A26 = 1 or 7 AND 6A29 = 20, 30, 32, 33, or 35) with a Condition Rating = 3 for Superstructure (1A04), Culvert (1A03) or Substructure (1A02) | |
| | | Highway bridges with an open steel grid deck (6A38 = 06) with a Condition Rating = 3 for Deck (1A01) or Superstructure (1A04) | |
| | | Highway bridges with Condition Rating = 2 for Superstructure (1A04), Substructure (1A02), Deck (1A01), Channel (1A05), Channel Protection (1A05b), Culvert (1A03), Scour (1A13) or SCBI (4A08) | |
| | | Highway bridges with an Observed Scour Rating (IN03) ≤ 3 | |

Table 2 of 2

Note: R = Routine Inspection
SP = Special Inspection
NR = Not Required
[3] See Table IP 2.3.6.4-1 for NSTM Inspection Interval.
[4] For NCABB bridges without an independent deck, a 12-month Routine is only required when all three Condition Ratings = 3 (Deck, Superstructure, and Substructure).

<table>
<tr><th colspan="4">Table IP 2.3.2.4-2 Intervals of Routine and Special Inspections for State Owned Bridges 8' to 20' in Length</th></tr>
<tr><th colspan="2">7A09 Insp Interval (months)</th><th rowspan="2">Bridge Description</th><th rowspan="2">Comments</th></tr>
<tr><th>R</th><th>SP</th></tr>
<tr><td>48</td><td>NR</td><td>Bridge must meet all the following criteria:<br>▪ State or PTC owned (5A21= 01 or 31)<br>▪ State or PTC maintained (5A20 = 01 or 31)<br>▪ Deck (1A01), Superstructure (1A04), Substructure (1A02), and Culvert (1A03) Condition Rating ≥ 6 or N depending on the structure type<br>▪ SCBI (4A08) = 5, 7, 8, 9 or N<br>▪ Known Foundation Type (IN13) ≠ P or X when Service Type Under (5A18) contains a waterway (i.e., 5 thru 9)<br>▪ Min. Vert. Underclearance (4A17) ≥ 14.5' (Over Non-Principal Arterial, 5C22 ≠ 1, 2, 3, 11, 12 or 14)<br>▪ Min. Vert. Underclearance (4A17) ≥ 16.0' (Over Principal Arterial, 5C22 = 1, 2, 3, 11, 12 or 14)<br>▪ Bridge with no NSTMs (6A44 ≥ 5)<br>▪ Structure Type is not Adjacent Non-Composite Box Beams (6A26-6A29 ≠ 42107)<br>▪ Structure Material is not Stone Masonry (6A26 ≠ 6)<br>▪ Structure Type is not a moveable lift structure (6A29 ≠ 26)<br>▪ Posting Status (VP02) = A (Open, no restrictions)<br>▪ Load Rating ≥ State Legal Load:<br>HS20 OR ≥ 1.1 * 36 tons<br>ML80 OR ≥ 1.1 * 36.6 tons<br>TK527 OR ≥ 1.1 * 40 tons<br>▪ Bridge does not have any active or deferred Priority 0 or 1 Maintenance Items</td><td>Bridges meeting the extended NBI inspection interval of 48 months will be identified in BMS2 on the Ratings and Schedule Screen (7A19 = Yes).<br><br>In addition, each District will approve the use of extended NBI inspection interval in BMS2 on the Ratings and Schedule Screen (7A20 = Yes).</td></tr>
<tr><td rowspan="3">24</td><td rowspan="3">NR</td><td>Highway bridges without restrictions or conditions described below</td><td></td></tr>
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Table 1 of 2

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<tr><th>R</th><th>SP</th></tr>
<tr><td rowspan="6">24</td><td rowspan="6">12</td><td>Highway bridges with weight restrictions (e.g., posted weight limit and/or restricted to One Truck at a Time) (VP02 = P or R)</td><td rowspan="14">Special Inspections of critical areas only of a bridge are to be used to meet the reduced interval between the biennial Routine Inspections.<br><br>When a reduced inspection interval is warranted, the inspection report is to list the justification, scope, intensity and interval for the next Special Inspection in the recommendation section.<br><br>For Highway Tunnels Special Inspections shall be determined by owner and District Bridge Engineer and outlined in the Inspection Procedure document located in BMS2.<br><br>For any structure with a Special Inspection at a reduced interval that requires divers to assess the condition, an underwater inspection must also be scheduled at the same interval to ensure divers are scheduled for the work.</td></tr>
<tr><td>Highway bridges with no NSTMs (6A44 = 5, 6 or 9) with a Condition Rating = 3 for Superstructure (1A04), Substructure (1A02), Channel (1A05) or Culvert (1A03)</td></tr>
<tr><td>Highway bridges with a Condition Rating = 4 for NSTM[3] (1A15)</td></tr>
<tr><td>Highway bridges of metal arch culvert construction (6A26 = 1 or 7 AND 6A29 = 20, 30, 32, 33, or 35) with a Condition Rating = 4 for Superstructure (1A04), Culvert (1A03) or Substructure (1A02)</td></tr>
<tr><td>Highway bridges with an open steel grid deck (6A38 = 06) with a Condition Rating = 4 for Deck (1A01) or Superstructure (1A04)</td></tr>
<tr><td>Highway bridges of masonry arch construction with a Condition Rating ≤ 4 for Superstructure (1A04), Culvert (1A03) or Substructure (1A02). See IP 2.5.3 and IP 2.5.4 for more instructions.</td></tr>
<tr><td rowspan="8">24</td><td rowspan="8">6</td><td>Highway bridges with a Condition Rating = 3 for NSTM[3] (1A15)</td></tr>
<tr><td>Highway bridges of metal arch culvert construction (6A26 = 1 or 7 AND 6A29 = 20, 30, 32, 33, or 35) with a Condition Rating = 3 for Superstructure (1A04), Culvert (1A03) or Substructure (1A02)</td></tr>
<tr><td>Highway bridges with an open steel grid deck (6A38 = 06) with a Condition Rating = 3 for Deck (1A01) or Superstructure (1A04)</td></tr>
<tr><td>Highway bridges with Condition Rating = 2 for Superstructure (1A04), Substructure (1A02), Deck (1A01), Channel (1A05), Culvert (1A03) or SCBI (4A08)</td></tr>
<tr><td>Highway bridges with an Observed Scour Rating (IN03) ≤ 3</td></tr>
<tr><td>Highway bridges with an SCBI (4A08) = 6</td></tr>
<tr><td>Any temporary structure or other temporary conditions exist (VP02 = D or E or 5E03 = T)</td></tr>
<tr><td>Highway bridges with an outstanding Priority 1 Maintenance Item</td></tr>
</table>

Table 2 of 2

Note: R = Routine Inspection
SP = Special Inspection
NR = Not Required
[3] See Table IP 2.3.6.4-1 for NSTM Inspection Interval.

LATE INSPECTION: A Routine NBI inspection exceeding 24/48 months will result in a reduced period until the next Routine NBI inspection. To prevent schedule creep following a late inspection, the required date of the next Routine inspection will be entered as 24/48 months from the previously scheduled date, and not from the actual date of the late inspection. This will result in a period of less than 24/48 months between a late inspection and the next required Routine inspection. For example, a Routine NBI inspection scheduled in August and conducted in October would be due again 24/48 months from August.

A Routine NTI inspection may be performed within two months before or two months after the target inspection month. If the inspection occurs within this time frame, the next inspection month is based on the target inspection month from this cycle. If an NTI inspection occurs more than two months past the target inspection month, this will result in a reduced period until the next NTI inspection. The next required NTI inspection date will still be based on the target date.

EARLY INSPECTION: An NBI inspection conducted before its scheduled date, i.e., less than 24/48 months, will result in a change to the date for all future NBI inspections. For example, to adjust work load in a given period, the inspection of a given bridge may be conducted before its 24/48-month interval. The required date of the next NBI inspection following an early NBI inspection will then be based on the 24/48-month interval established from the date of the early NBI inspection to ensure the time lapse between inspections does not exceed the NBI interval.

It is not recommended that an NTI inspection is conducted more than two months before its scheduled target date. If there is a valid reason to conduct the inspection sooner than two months before the target date, the Owner shall contact the Assistant Chief Bridge Engineer - Inspection and FHWA for written approval.

EXTENDED NBI INSPECTION INTERVALS (LONGER THAN 24 MONTHS) FOR NBIS BRIDGES: When an NBI (routine) inspection interval longer than 24 months is proposed for an NBIS highway bridge (greater than 20 feet), the bridge must meet the criteria in Table IP 2.3.2.4-1.

### 2.3.3 Damage Inspections

#### 2.3.3.1 DESCRIPTION OF DAMAGE INSPECTIONS

Damage Inspections are performed following extreme weather-related events, earthquakes, fires, explosions, vandalism and vehicular/marine traffic crashes, as directed by the District Bridge Engineer. In many ways, a Damage Inspection is a Special Inspection that is necessitated by an extreme event. When major damage has occurred, the inspectors will need to evaluate fractured or failed members, determine the amount of section loss, make measurements for misalignment of members, and check for any loss of foundation support. For Damage Inspections 7A03 = B.

When severe damage occurs, the bridge shall remain closed until a damage inspection has been completed.

#### 2.3.3.2 PURPOSE OF DAMAGE INSPECTIONS

Damage Inspections serve to first determine the nature, severity, and extent of structural damage following extreme weather-related events, seismic events and vehicular or marine traffic collisions/accidents for use in designing needed repairs. A post-flood inspection is to be coded as a Damage Inspection only when a new P0 or P1 Maintenance Item is identified during the inspection (see Sour Monitoring Inspections, IP 2.3.9). A Damage Inspection is to determine the immediate need to place an emergency restriction on a bridge (e.g., weight restriction or closure) for vehicular traffic. If a bridge is closed to vehicular traffic, the need to close it to pedestrian traffic should also be determined. A Damage Inspection does not satisfy the reduced interval for inspections.

The findings of a Damage Inspection may be used to re-coup the costs of inspection and needed repairs or reconstruction from involved parties or other governmental agencies. Accordingly, documentation of the inspection may be critical in these efforts. For Department bridges, the extent of damage and estimated costs of repair should be reported to the District Bridge Maintenance coordinator. Photographs, videos and sketches can be extremely helpful. This documentation should be released in accordance with IP 1.8. See IP 2.9 for additional information regarding reporting bridge and structure emergencies.

#### 2.3.3.3 INTENSITY OF DAMAGE INSPECTIONS

The amount of effort expended on this type of inspection will vary significantly depending upon the extent of the damage, the volume of traffic encountered, the location of the damage on the structure, and documentation needs. The scope of a Damage Inspection will be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic, and to estimate the level of effort necessary to accomplish repairs. The scope of the Damage Inspection shall be documented in each inspection report using fields in BMS2 (see IP 2.3). The capability to make an on-site determination of the need to establish emergency load restrictions may be necessary.

Structural materials may need further evaluation as identified in the MBE. For additional information on Damage Inspections for tunnels, see TOMIE Section 4.6.3.

**2.3.3.4** INTERVAL OF DAMAGE INSPECTIONS

A Damage Inspection is an unscheduled inspection to assess the structural damage resulting from environmental factors or human actions. Damage Inspections are performed on an as-needed basis.

### 2.3.4 In-Depth Inspections

**2.3.4.1** DESCRIPTION OF IN-DEPTH INSPECTIONS

An In-Depth Inspection is a close-up detailed inspection of one or more elements or functional systems above or below the water level, using visual or nondestructive evaluation techniques as required to identify any deficiencies not readily detectable using Routine Inspection procedures. Hands-on inspection may be necessary at some locations. In-Depth Inspections are typically limited to certain elements, span group(s), or structural units of a structure, and need not involve the entire structure. In-Depth Inspections can be conducted alone or as part of a Routine or other type of inspection.

**2.3.4.2** PURPOSE OF IN-DEPTH INSPECTIONS

In-Depth Inspections serve to collect and document data to a sufficient detail needed to ascertain the physical condition of a bridge. This “hard-to-obtain” data is more difficult to collect than data collected during a Routine Inspection. An In-Depth Inspection is not to be used to satisfy the reduced interval for Routine Inspections.

**2.3.4.3** INTENSITY OF IN-DEPTH INSPECTIONS

The level of effort required to perform an In-Depth Inspection will vary according to the structure’s type, size, design complexity, existing conditions, functional systems, and location. Traffic control and special equipment, such as under-bridge cranes, rigging, or staging may be needed for In-Depth Inspections. Personnel with special skills such as divers, riggers and certified technicians may be required. Non-destructive field tests and/or material tests may be performed to fully ascertain the existence of or the extent of any deficiency. The scope of the In-Depth inspection, including bridge-specific inspection procedures, shall be documented in each inspection report using fields in BMS2 (see IP 2.3).

For large or complex structures, these inspections may be scheduled separately for defined segments of the bridge or for designated groups of elements, connections, details or functional systems 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, details or functional systems should be clearly identified as a matter of record and shall be assigned an interval for inspection. Components that may receive an In-Depth inspection can include pins, suspension cables, steel box beams, segmental concrete box beams, etc. The activities, procedures, and findings of In-Depth Inspections will be completely and carefully documented to an even greater extent than is necessary for Initial and Routine Inspections. Stated differently, In-Depth Inspection reports will generally be detailed documents unique to each structure that exceed the documentation of standard or routine inspection forms.

In-Depth Inspections of tunnels may involve testing of tunnel systems, components, and materials. Disassembly and cleaning of equipment and components may also be required.

A structural analysis for load carrying capacity may be required with an In-Depth inspection to fully evaluate the effect of the more detailed scrutiny of the structure condition.

**2.3.4.4** INTERVAL OF IN-DEPTH INSPECTIONS

An In-Depth Inspection can be scheduled in addition to a Routine Inspection, though generally at a longer interval, or it may be a follow-up to a previous inspection. The interval of In-Depth Inspections will be established by the bridge owner and shall be outlined in the bridge specific inspection procedures/inspection scope field.

In-Depth Inspections should be routinely scheduled for selected bridges based on their size, complexity and/or condition. Large bridges (longer than 500 feet) represent large capital investments and warrant closer scrutiny to ensure that maintenance work is identified and completed in a timely manner. Large bridges tend to be more critical to local and area transportation because of the usual lack of suitable detours.

In-Depth Inspections will be coded as such in BMS2 item 7A03 and can be part of, or independent from, a Routine Inspection.

In-Depth Inspections do not reduce the level of intensity for Routine Inspections as discussed in IP 2.3.2.3.

**2.3.5 Special Inspections**

During Special Inspections, attention should be given to primary load carrying members of posted bridges; poor, serious and critical condition ratings; and high priority maintenance recommendations. The Special Inspection must address all work items specified in the in the scope of work for the inspection. The results of the inspection of such items and need for follow-up should be properly noted on the reporting forms for each structure for which the items are applicable.

**2.3.5.1** DESCRIPTION OF SPECIAL INSPECTIONS

Special Inspections are scheduled by the bridge owner to examine bridges or portions of bridges with known or suspected deficiencies which require monitoring. Special Inspections tend to focus on specific areas of a bridge where problems were previously reported or to investigate areas where problems are suspected. By definition, Special Inspections do not fulfill NBIS/NTIS requirements for Routine Inspections. Special Inspections are structured to fulfill the need for Interim Inspections between the 24-month Routine Inspections. Special Inspections are conducted until corrective actions can remove high priority maintenance recommendations, the component is removed from service, or further study determines conditions are no longer deteriorating at accelerated levels.

**2.3.5.2** PURPOSE OF SPECIAL INSPECTIONS

Special Inspections are used to monitor posted bridges; poor, serious and critical condition ratings; bridges with severe scour issues or known high priority maintenance recommendations and fulfill the need for more frequent inspections. Special Inspections are intended to satisfy the reduced inspection interval specified for Special Inspections in Tables IP 2.3.2.4-1 and IP 2.3.2.4-2.

**2.3.5.3** INTENSITY OF SPECIAL INSPECTIONS

The level of effort required to perform a Special Inspection will vary with its purpose and the structure's type, size, design complexity, existing conditions, and type of deficiency being investigated.

When a bridge's condition requires a reduced inspection interval (as per Tables IP 2.3.2.4-1 and IP 2.3.2.4-2), only a portion or portions of the bridge that are in critical condition may warrant more frequent and/or intense inspection to ascertain its safety. For such situations, a Special Inspection of limited scope for the critical portions is to be used to satisfy the reduced interval requirement and reduce overall inspection cost. The engineer-in-charge of an inspection contract or group of bridges (either consultant or District) is to develop an appropriate scope of work. The District Bridge Engineer must review and approve the scope of work and interval of Special inspections to ensure compliance with this manual and for eligibility for federal reimbursement of inspection costs. The scope of the reduced interval inspections shall be documented in each inspection report using fields in BMS2. Future Inspection Scope Comment Type 481on the Notes & Comments screen of BMS2 and the BMS3 Notes page shall be used to enter the scope of the next scheduled Special Inspection, and B.IE.11 (Inspection Note) shall be used to enter the scope/inspection procedures of the current reduced interval inspection. If the limited scope Special Inspection has not been authorized (scope field in BMS2 is blank), a Routine Inspection is to be performed to fulfill the reduced interval inspection requirement and meet NBIS/NTIS compliance requirements. In this case, the NBIS/NTIS target date will be reset. See IP 2.3.2 for more information on Routine Inspections.

Examples where a Special Inspection may be utilized on a reduced interval to sufficiently monitor and evaluate deteriorated portions of a bridge in a cost-effective manner include, but are not limited to:

- Inspecting substructure in serious condition due to settlement, but not inspecting the deck and superstructure in satisfactory condition. The stream channel and IR screen items did not need interim inspections.
- Inspecting only the deteriorated ends of a steel stringer bridge that control condition rating and load rating. The satisfactory condition of the deck and substructure do not warrant an interim inspection.

Special Inspections must be performed by a qualified team leader. The personnel performing a Special Inspection should be familiar with the nature of the known deficiency and its functional relationship to satisfactory bridge performance.

#### **2.3.5.4** INTERVAL OF SPECIAL INSPECTIONS

Special Inspections are scheduled by the noted interval in Tables IP 2.3.2.4-1 and IP 2.3.2.4-2 or more frequently at the discretion of the bridge owner. The determination of an appropriate Special Inspection interval should consider the nature, severity and extent of the known deficiency, as well as age, traffic characteristics, public importance, and maintenance history.

When a reduced inspection interval is required by Table IP 2.3.2.4-1 or IP 2.3.2.4-2, check the data box for Special Inspections in BMS Item 7A07 Required Inspection. Use the interval from Table IP 2.3.2.4-1 or IP 2.3.2.4-2 to populate 7A09 Inspection Interval for Special Inspections.

### 2.3.6 NSTM Inspections

#### **2.3.6.1** DESCRIPTION OF NSTM INSPECTIONS

As stated in the NBIS, a "nonredundant steel tension member (NSTM) is a primary steel member fully or partially in tension, and without load path redundancy, system redundancy or internal redundancy, whose failure may cause a portion of or the entire bridge to collapse." Tension elements of a bridge member consist of those portions of a flexural (bending) member that are subject to tension or reversal stress.

An NSTM Inspection is a hands-on inspection of an NSTM. These NSTM Inspections must be recorded in the NBI file submitted to FHWA – this recordation is to be accomplished through the required BMS2 data.

NSTM Inspections are not applicable to tunnels.

#### **2.3.6.2** PURPOSE OF NSTM INSPECTIONS

The purpose of an NSTM Inspection is twofold:

- Identify and record the location of fatigue and fracture sensitive details and any problems or potential problems at these locations in order to determine the safety of the structure. For bridges with fatigue-prone details, these inspections provide a history of cracking (time of initiation, rate of growth, etc.) that can greatly assist the engineer in determining the need and priority of repairs and in estimating the remaining life of the bridge.

- Conduct a hands-on inspection of the complete NSTMs, details, and connections to identify and record the location of fatigue prone and fracture prone details and to identify section loss, deterioration or other conditions that may affect bridge safety and performance.

**2.3.6.3** INTENSITY OF NSTM INSPECTIONS

NSTMs require a hands-on inspection of NSTMs, details and connections to discover cracking, section loss and deterioration. The inspector should plan to have proper access to reach the members to be inspected. In addition, the inspector should have a magnifying glass and dye penetrant kits on site, at a minimum, as they will be most helpful in the initial investigation. Lighting to ensure details are visible may also be critical.

NSTM Inspections are to be completed in conjunction with a Routine, Special, or In-Depth Inspection. The engineer-in-charge should determine the intensity of the inspection for each NSTM detail. Factors for consideration include criticality of detail, tension stress level, overall member condition, estimated remaining fatigue life of detail, ADTT, retrofits, etc.

When an NSTM Inspection is required at a reduced interval, the inspection must address all work items specified in the Fatigue and Fracture (F&F) Plan. The F&F Plan must identify those NSTMs that require hands-on inspection during the interim and provide guidelines and procedures on what to observe and/or measure during the inspection (see IP 2.4.5.1).

When a reduced interval is required by Table IP 2.3.6.4-1 a hands-on re-inspection of all NSTMs is required unless an F&F plan designates those NSTMs that require inspection as part of a limited scope Special Inspection. The F&F Plan must document that the proposed Special Inspection would include hands-on inspection of the portions of the NSTMs that necessitate the reduce interval, namely:

- Those NSTMs in poor or worse condition having advanced or serious sections loss.
- NSTMs with a load capacity at Safe Load Capacity (SLC) level that is less than legal loads.

For the other NSTMs on the bridge, the engineer may detail in the F&F Plan those NSTMs (or portions thereof) that do not require hands-on inspection during Special Inspections. The scope of a limited Special Inspection, including inspection procedures, must be in the F&F Plan and approved by the District Bridge Engineer (State owned bridges) or by a professional engineer representing the local owner (locally owned bridges). The BMS2 note fields for scope of a reduced interval inspection (see Ip 2.3) shall reference the F&F Plan.

Concrete-encased NSTMs are to be inspected with other NSTMs on a bridge. However, when the encasement is intact, hands-on inspection may not greatly add to information garnered by careful observation during a non-hands-on visual inspection. Accordingly, some adjustment to the F&F Plan for these concrete-encased NSTMs may be appropriate and is to be addressed in the F&F Plan. Concrete-encased NSTMs of higher susceptibility to fracture or fatigue may include: those with details of fatigue Category D or higher (more fatigue susceptible), known out-of-plane bending details, cracked details without retrofit, advanced section loss, severe collision damage to steel, or other considerations.

**2.3.6.4** INTERVAL OF NSTM INSPECTIONS

An NSTM inspection shall always coincide with a Routine or Special Inspection. Table IP 2.3.6.4-1 shall be used in conjunction with Tables IP 2.3.2.4-1 and Table IP 2.3.2.4-2 to determine the NSTM interval. The B.IE.01 Inspection Type = “04-NSTM”, should not be checked without the Routine or Special performed box being checked.

The first NSTM Inspection for new bridges or for bridges with rehabilitation of NSTMs shall be performed as soon as practical, but within 3 months of the bridge opening to traffic.

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| Table IP 2.3.6.4-1 Intervals of NSTM Inspections | |
|---|---|
| **7A09 NSTM Insp Interval (months)** | **Bridge Description** |
| **24** | Highway bridges containing NSTMs (6A44 < 5) without restrictions or conditions described below |
| **12** | Highway bridges containing NSTMs (6A44 < 5) with an NSTM Condition Rating (1A15) = 4 |
| **6** | Highway bridges containing NSTMs (6A44 < 5) with an NSTM Condition Rating (1A15) < 4 |

### 2.3.7 Underwater Inspections

#### 2.3.7.1 DESCRIPTION OF UNDERWATER INSPECTIONS

As defined by the NBIS, an underwater inspection consists of the inspection of the underwater portion of a bridge substructure and the surrounding channel, which cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.

During periods of low flow, underwater members will be inspected visually and by feel using probing rods, water boxes, sounding lines or other hand tools while wading. Since the majority of inspections of the underwater portions of bridges can be performed using wading and probing methods, they typically fall within the scope of Routine Inspections and the capabilities of both in-house Department inspection staff and inspection consultants. When the physical condition of the substructure members or the integrity of their foundations cannot be determined using the probing tools due to high water, high flow, turbidity, etc., inspection by divers is required. Only underwater inspections performed by divers are defined as “underwater” and coded as such in BMS2. Some technologies such as ground sensing radar, ultrasonic techniques, remote video recorders, and others have proven to be alternate methods for underwater inspections of substructure foundations in selected situations.

Key information to be determined in every underwater inspection about the integrity of the foundations is the top of streambed relative to the elevation of the substructure foundations. Because scour can vary significantly from one end of a footing to the other, a single probing reading is not sufficient. Baseline streambed conditions should be established by waterway opening cross sections and by a grid pattern of probing readings around the face of a substructure unit. This baseline information makes future monitoring and assessment easier and more accurate. The current streambed conditions and changes since the last inspection are critical inputs to the bridge scour assessment.

Due to the challenge of maintaining trained inspector divers and adequate equipment for a small number of underwater diving inspections, BIS maintains Statewide open-end engineering consultant agreements to provide underwater inspection services to the Districts. Under these agreements, consultants perform scheduled and emergency underwater inspections of State and local bridges and provide additional expert services in this area as needed. Where possible and practical, these Statewide open-end agreements are to be used by the Districts and local bridge owners for diving inspections, instead of contracting separately for these services. Probing and other underwater inspection methods are also available through these agreements. See IP 1.10 for further guidelines on the use of these agreements. See Appendix IP 01-H for the standard scope of work used for underwater inspections.

Underwater Inspections are not applicable to tunnels.

#### 2.3.7.2 PURPOSE OF UNDERWATER INSPECTIONS

Regularly scheduled Routine Inspections include the inspection and evaluation of all pertinent bridge components to ensure that the structure continues to satisfy present safety and service requirements. The purpose of

Underwater Inspections is to provide similar information on underwater portions of a bridge to evaluate their overall safety and, especially, to assess the risk of failure due to scour.

**2.3.7.3** INTENSITY OF UNDERWATER INSPECTIONS

Each bridge shall have local benchmarks established near each substructure unit to enable inspectors to quickly and accurately determine the depth of adjacent scour. These benchmarks can be as simple as a painted line or PK survey nail driven into the wall in a place visible during high water. The location of these scour-monitoring benchmarks should be referenced in the inspection records. Use previously established benchmarks when possible to provide a long-term record of scour conditions. If new benchmarks need to be established, provide conversion from new to old datum.

During Underwater Inspections, particular attention should be given to foundations on spread footings where scour or erosion can be much more critical than at deep foundations on piles or caissons. However, be aware that scour and undercutting of a pier or abutment on a deep foundation can also be quite serious. The foundation's vertical support capacity normally will not be greatly affected unless the scour is excessively severe, but the horizontal stability may be jeopardized. This condition becomes particularly unstable when erosion has occurred on only one face of the substructure unit, leaving solid material on the opposite face. Horizontal loads may also have been produced by earth, debris, or rock fills piled against or adjacent to substructure units whose loads were obviously not provided for in the original design. Such unbalanced loading can produce an unstable condition, requiring corrective action.

BMS3 INSPECTION MODULE, BMS2 AND UNDERWATER INSPECTIONS: The BMS3 Substructure page allows inspectors to record each underwater component. The BMS2 6B, 7A, IU, and IN inspection screens also have data fields to indicate the underwater inspection requirements, including information to assess the scour criticality, required underwater inspection interval, level of inspection, and foundation data to manage and schedule the underwater inspections.

**2.3.7.4** INTERVAL OF UNDERWATER INSPECTIONS

Underwater inspections are intended to investigate two critical issues regarding the condition of bridge substructures located in water:

- The condition of structural components (including pier shaft, abutment walls, footings, etc.) under water.
- The integrity of the substructure foundation (including underlying soil, piles, caissons, etc.) against scour at each substructure unit in water.

The inspection of the underwater portions of a substructure unit and the determination of its ongoing resistance to scour is critical for the overall safety of the bridge. Because the integrity of the foundation against scour can suddenly and dramatically change in a relatively short time (as compared to physical condition of the structure components), shorter intervals for inspection of the foundation may be warranted. The recommended intervals for underwater inspection of the foundation of substructure units for bridges over water are based upon a scour assessment of each unit and are shown in Table IP 2.3.7.4-1.

The first Underwater Inspection for new bridges or for bridges with underwater portions rehabilitation shall be performed as soon as practical, but within 12 months of the bridge opening to traffic.

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**Table IP 2.3.7.4-1 Intervals of Underwater Inspections for Underwater Portions of Bridges**

| 7A09 UW Insp Interval (months) | Bridge Description |
|---|---|
| 72 | Bridge must meet all the following criteria:<br>▪ Underwater Inspection (1A14), Scour (1A13), Channel (1A05), and Channel Protection (1A05b) Condition Rating ≥ 6 or N<br>▪ Scour Vulnerability (IU29) = A, B, or AB-T[1]<br>▪ Bridge has received a First-Time inspection within 12 months of opening or rehabilitation to the underwater portion of the bridge |
| 48* | Highway bridges without restrictions or conditions described below |
| 24* | Underwater Inspection (1A14), Scour (1A13), Channel (1A05), Channel Protection (1A05b) or SCBI (4A08) Condition Rating = 3 |
| 6* | Observed Scour Rating (IN03) = 3 |
| | Underwater Inspection (1A14), Scour (1A13), Channel (1A05), Channel Protection (1A05b) or SCBI (4A08) Condition Rating = 2 |

UW = Underwater
[1]AB-T is only a temporary coding and will be removed after one inspection cycle is completed on all bridges.
* Refer to the Scour Plan of Action (POA) for scour critical bridges for instructions on the need for inspection after a high-water event.

The inspection interval for underwater inspections is to be coded in BMS2 Item 7A09 and item 7A07 shall be checked.

### 2.3.8 Service Inspections

#### 2.3.8.1 DESCRIPTION OF SERVICE INSPECTIONS

A Service Inspection is an inspection to identify major deficiencies and safety issues. This inspection type may be used as a cursory inspection for bridges on a 48-month extended interval or for non-State-owned structures located over a public highway (previously Highway Environs Inspection).

#### 2.3.8.2 PURPOSE OF SERVICE INSPECTIONS

The purpose of a Service Inspection is to determine the overall serviceability of the structure by identifying major deficiencies and/or safety concerns.

#### 2.3.8.3 INTENSITY OF SERVICE INSPECTIONS

Service Inspections are a cursory review from a distance to review the overall functionality of the structure.

#### 2.3.8.4 INTERVAL OF SERVICE INSPECTIONS

There is no set interval for Service Inspections.

### 2.3.9 Scour Monitoring Inspections

#### 2.3.9.1 DESCRIPTION OF SCOUR MONITORING INSPECTIONS

Hold for future.

**2.3.9.2** PURPOSE OF SCOUR MONITORING INSPECTIONS

Hold for future.

**2.3.9.3** INTENSITY OF SCOUR MONITIORING INSPECTIONS

Hold for future.

**2.3.9.4** INTERVAL OF SCOUR MONITORING INSPECTIONS

Hold for future.

### 2.3.10 Problem Area Inspections

**2.3.10.1** DESCRIPTION OF PROBLEM AREA INSPECTIONS

Problem Area Inspections are typically limited scope inspections which are not scheduled in advance. These inspections focus on a specific issue on the bridge and are typically used to document a change when another inspection type is not applicable.

**2.3.10.2** PURPOSE OF PROBLEM AREA INSPECTIONS

Typically, a Problem Area Inspection will be performed to document an inspection which occurs outside of the interval of Routine or Special Inspections. This inspection type could be used to document a repair associated with a Priority 1 Maintenance Item and eliminate the need for a Special to occur at the 6-month interval after the last inspection which identified the Priority 1 Maintenance Item.

A Problem Area Inspection may also be used to document an inspection that was prompted through a public complaint or other unexpected communication. If damage is noted during the inspection, it should be coded as a Damage Inspection instead of Problem Area (refer to IP 2.3.3 for Damage Inspections).

**2.3.10.3** INTENSITY OF PROBLEM AREA INSPECTIONS

The level of effort required to perform a Problem Area Inspection will vary with the scope of the inspection to be performed. Typically, the problem area in question should be examined up close to properly assess the effects of its condition on the overall structure. The scope of the inspection including any specialized procedures shall be documented in field B.IE.11 (Inspection Note) for each Problem Area inspection.

**2.3.10.4** INTERVAL OF PROBLEM AREA INSPECTIONS

A Problem Area Inspection is not a regularly scheduled inspection and does not have an interval. This inspection type will be performed on an as-needed basis.

### 2.3.11 Element Inspections

**2.3.11.1** DESCRIPTION OF ELEMENT INSPECTIONS

Element Inspections are used to collect condition information at the element level for all State-owned bridges greater than 8 feet in length, locally owned NBIS bridges on the NHS, and all tunnels. While Element Inspections are not a compliance inspection type required by FHWA, element-level data is submitted annually to FHWA as part of the NBI/NTI. An Element Inspection shall be completed with every Initial, Routine, and NSTM Inspection since each of these compliance inspection types require element level data be collected. The collection of element-level data for other inspection types is dependent on the scope of that inspection and determination should be made on a case-by-case basis. Anytime element level data is collected as part of an inspection, the inspection shall also be coded as having an Element Level Inspection completed.

The Initial Inspection of bridges shall include a complete inventory of all elements and their condition states. It is recommended that the element inventory be completed before the field work is initiated. The inspector shall utilize design plans and calculations to help quantify element data. When an Element Inspection is completed for any other inspection type, the inspector must verify that the inventory of elements is complete and accurate for the bridge and add any missing elements.

For local bridges and other non-Department bridges where element-level data collection is not required, the Department recommends that owners specify for their consultant to complete Element Inspections for their bridges. BMS2 is available to all owners for element-level data collection. When element-level data is completed for an inspection, the inspector must verify that the inventory of elements is complete for the bridge and add any missing elements.

For tunnel inspections, element-level data shall be collected and reported in BMS2 in accordance with the SNTI for all tunnels and all inspection types, regardless of ownership. Publication 100A, Appendix D provides correlations between SNTI coding and BMS2 coding.

**2.3.11.2** PURPOSE OF ELEMENT INSPECTIONS

The Element Inspection type was created by PennDOT to track if and when the element data was being updated for a given structure. One of the primary purposes of element-level data is the development of deterioration and cost models to predict the future condition of bridges and the attendant budgeting needs.

**2.3.11.3** INTENSITY OF ELEMENT INSPECTIONS

The intensity of Element Inspections will vary with the compliance inspection that the element level data is tied to for a given inspection.

### 2.3.12 Quality Assurance Inspections

**2.3.12.1** DESCRIPTION OF QUALITY ASSURANCE INSPECTIONS

Quality Assurance (QA) Inspections are not a compliance type of inspection but are used internally by PennDOT to ensure the quality of compliance inspections. QA Inspections are intended to replicate a Routine Inspection for a sampling of bridges. The QA Inspection is then compared to the latest Routine Inspection to assess the level of quality achieved by that inspection.

Refer to the Department's Publication 240, *Bridge and Tunnel Safety Inspection Quality Assurance Manual* outlines for more detail on QA Inspections.

**2.3.12.2** PURPOSE OF QUALITY ASSURANCE INSPECTIONS

Quality Assurance Inspections have the sole purpose to gage the quality of other compliance inspections being performed in Pennsylvania. Refer to Publication 240 for additional information.

**2.3.12.3** INTENSITY OF QUALITY ASSURANCE INSPECTIONS

The intensity of a QA Inspection shall be the same as that of a Routine Inspection. Refer to Section IP 2.3.2.3 for intensity of Routine Inspections.

**2.3.12.4** INTERVAL OF QUALITY ASSURANCE INSPECTIONS

Quality Assurance (QA) Inspections are performed through a special contract which is administered through Central Office on a biennial basis. Each inspection cycle, 20 bridges have a QA Inspections completed in each District. Other owners also have a sampling of their bridges regularly undergo QA Inspections. Refer to Publication 240 for more information.

### 2.3.13 Inventory Only Inspections

**2.3.13.1** DESCRIPTION OF INVENTORY ONLY INSPECTIONS

The Inventory Only Inspection is a PennDOT specific inspection type automatically created by the system and will not be reported to FHWA.

**2.3.13.2** PURPOSE OF INVENTORY ONLY INSPECTIONS

PennDOT created the Inventory Only Inspection type for tracking when the inventory for any given structure is initially created and/or when it changes due to a major rehabilitation (new BRKEY created).

**2.3.13.3** INTENSITY OF INVENTORY ONLY INSPECTIONS

An Inventory Only Inspection shall be detailed enough to collect and/or verify the minimum amount of inventory information required for the NBI/NTI. If the inventory information is gathered from the design or rehabilitation drawings, the Inventory Inspection shall include field verification of all inventory data.

**2.3.13.4** INTERVAL OF INVENTORY ONLY INSPECTIONS

Inventory Only inspections are only completed when a new structure is built or after an existing structure undergoes major rehabilitation in which a new BRKEY is warranted.

### 2.3.14 Monitoring of Inspection Interval for NBIS/NTIS Compliance

The NBIS/NTIS require data collection for specific categories of highway bridge inspections which must be monitored for compliance. These inspection categories, called Compliance Inspection Types to differentiate them from the types in BMS Item 7A03, are utilized in BMS2 Inspection Scheduling items 7A06-7A10. The NBIS has nine Compliance Inspection Types which include Initial, Routine, Underwater, NSTM, Damage, In-Depth, Special , Service, and Scour Monitoring. The NTIS has four Compliance Inspection Types which are Routine, Damage, In-Depth and Special.

**2.3.14.1** RESPONSIBILITY FOR COMPLIANCE

The bridge owner has the primary responsibility to ensure that the individual bridges and structures under their purview are inspected in compliance with the standards as set forth in this Manual. Under the NBIS/NTIS regulations, the Department has a responsibility to ensure that public highway bridges are inspected. To ensure that the bridges and structures are inspected in a timely manner for public safety and for compliance with regulations, the responsibilities for monitoring and ensuring the compliance of safety inspection intervals are as follows:

- District Bridge Engineer for the following individual structures in their District:
  - Department bridges/structures
  - NBIS/NTIS bridges owned by local municipalities and Counties
  - Non-highway bridges/structures over State Routes
- BIS for the following individual structures Statewide:
  - NBIS/NTIS bridges owned by PTC, DCNR, and other State and toll agencies
- BIS for overall Statewide compliance of all structures.

To facilitate compliance by the owners, the BIS places four spreadsheets on the P-Drive for each District called the Monthly Compliance (M1) Report by the 5th of each month. In addition to the M1, four spreadsheets pertaining to underwater inspection are placed on the P-Drive for each District called the Monthly Underwater Compliance (UW M1) Report. If a District has a tunnel inspection coming due in the next 2 months, there will be an additional spreadsheet placed on the P-Drive folder for the tunnels called Tunnel M1. Emails are sent to the District Bridge Engineer and the Assistant District Bridge Engineer for Inspection as a notification that the spreadsheets have been posted to the P-Drive locations. Separate notifications are sent by the BMS2 Manager to the other State and toll agency bridge owners. Both the M1 and the UW M1 spreadsheets include inspections due in the current month and future months, as well as those due in the prior month as shown below:

- PennDOT NBIS (bridges > 20’ in length)

- PennDOT non-NBIS (8’ to 20’ long bridges)
- Non-PennDOT NBIS (bridges > 20’ in length)
- Past Due

The spreadsheet titled “Past Due” is necessary due to the lag in time allowed to upload inspection data to BMS2. In accordance with IP 8.1, the submission of the BMS3 inspection report is permitted to be submitted 10-days after the field inspection for bridges and 30-days for tunnels. Since some of the prior month’s inspection data is inconclusive on the 5th of the month, the District must complete a status of each bridge on the “Past Due” spreadsheet for the actual inspection dates to indicate whether the inspection occurred on-time or late. Similarly, the BMS2 Manager will coordinate completion of the actual inspection dates for bridges owned by other State and toll agencies. The completion of the actual inspection dates in the “Past Due” spreadsheet must be completed 15 days before the end of the month. BMS2 Manager will re-run the “Past Due” spreadsheet 10 days before the end of the month to confirm the completion of the inspections due in the previous month.

There is only one Tunnel M1 spreadsheet which lists all tunnels due in the next three months since the inspections for tunnels are allowed to begin up to two months prior to their next inspection due date.

In addition, the Assistant District Bridge Engineer for Inspection is also required to update the PennDOT NBIS, PennDOT non-NBIS and Non-PennDOT spreadsheets as well as the Tunnel M1 spreadsheet (if applicable) on the P-Drive 15 days before the end of the current month to provide either the actual or planned inspection dates for the current month. The BMS2 Manager is required to coordinate the actual or planned inspection dates for NBIS bridges owned by other State and toll agencies. After reviewing the dates compiled in the spreadsheets, the BMS2 Manager will report all outstanding district responses to the Assistant Chief Bridge Engineer - Inspection who will then contact the District Bridge Engineer or the other State and toll agencies to determine whether use of a State-wide open-end inspection agreement is necessary to complete either past due or current inspections. Consultants shall be responsible for submitting monthly schedules and progress updates to each District Bridge Engineer as required by the standard scope of work for bridge inspection agreements.

Annual lists for underwater inspections by divers will be prepared by each District and compiled by the BIS for State, local and other agency bridges. The November UW M1 Report spreadsheets will be used each year and completed by each District and Central Office by the end of January each year. Bridges needing an underwater inspection with a due date between April in the following calendar year and March of the subsequent calendar year will be included with the annual lists to ensure underwater inspections are scheduled and completed within the required interval. Underwater inspections by divers are completed predominately with Central Office Statewide engineering agreements and funded by the BIS; the completion of the annual list will also be used to forecast fiscal year budgetary commitments.

In those cases where a structure is due for a Routine Inspection and Table IP 2.3.2.4-1 or Table IP 2.3.2.4-2 requires an inspection interval less than 24 months, the coding for Item 7A09 (Inspection Interval) must reflect the reduced interval.

To facilitate the monitoring of compliance of the inspection interval with the maximum 24/48-month interval allowed by NBIS/NTIS and any reduced interval due to bridge conditions, the coding in Table IP 2.3.2.4-1 and IP 2.3.2.4-2 should be used for BMS2 data item 7A03 (primary inspection type) and 6B20 (next inspection type).

**2.3.14.2** TOLERANCES FOR INTERVALS OF COMPLIANCE TYPES OF INSPECTION

The tolerance allowed for variations between the actual interval and the required interval for each inspection type will be measured in months with the schedule based on the last Routine Inspection. The tolerance outlined herein is only applicable to the following inspection types for bridges and culverts: Initial, Routine, Special, NSTM, and Underwater Inspections. Bridges inspected within the tolerances listed below will be considered “on time” and compliant with required inspection interval.

- The acceptable tolerance for intervals of less than 24 months for the next inspection is up to two (2) months after the month in which the inspection was due.

- The acceptable tolerance for intervals of 24 months and greater for the next inspection is up to three (3) months after the month in which the inspection was due.
- There is no acceptable tolerance for intervals of 6 months and less for the next inspection. These inspections have a zero (0) month tolerance and must be completed in the month they are due.
- Exceptions to the inspection interval tolerance due to rare and unusual circumstances must be approved by FHWA and PennDOT in advance of the inspection due date.

For tunnels, the 24-month Routine Inspection interval is a regulatory requirement, and the NTIS allows a tolerance of +/- 2 months (before or after) the scheduled month. Tunnels are not to be inspected prior to the two-month tolerance without written approval from FHWA and PennDOT. If the regularly scheduled Routine Inspection is to occur when the tunnel is under repair or rehabilitation, the next Routine Inspection shall occur after all construction is complete and systems are tested, but prior to opening the tunnel to traffic.

**2.3.14.3** TRACKING DATA FOR INSPECTIONS COMPLETED FOR NBIS/NTIS

See Publication 100A coding description of 7A03 for tables which indicates the appropriate values for Item 7A06 Inspection Performed for the 7A03 Primary Inspection Type and the interval of inspection for bridges and miscellaneous structures. Tunnels are covered under a separate table in the same location. To facilitate the monitoring of compliance of the inspection interval with the 24/48-month NBI/NTI interval and any reduced interval Special Inspections due to bridge conditions, the coding values shown in these tables should also be used for BMS2 data 6B20 Next Inspection Type.

**2.3.14.4** COMPLIANCE MONITORING DATA

Bridge owners are to utilize the Inspection Scheduling data fields on the BMS2 Ratings and Schedule screen to document the completion of the various inspection types required by the NBIS/NTIS regulations and Department policy. The Department uses the Inspection Scheduling data fields to report required information to the NBI/NTI and to monitor inspection interval compliance. Additional guidance for this data may be found in Publication 100A Inspection Scheduling Items 7A06-7A10 and Item1A09 – Inspection Status.

Table IP 2.3.14.4-1 lists the Compliance Inspection Types for NBIS and NTIS and indicates how they are reported to the NBI/NTI. BMS2 automatically populates NBI items 90-93 for the annual report to FHWA during the NBI Extract process when the BMS2 Scheduling data for items 7A06-7A09 is correctly coded. Not all of the 7A03 Inspection Types are applicable to the NBI/NTI; therefore, not all inspection types are reported to FHWA. Similarly, BMS2 automatically populates D.2 through D.6 for tunnels for the NTI annual report.

Due to changes in the 2022 NBIS/SNBI, PennDOT is working on modifying the inspection types and associated BMS2 data fields to better align with these revised documents. The fields shown in Table IP 2.3.14.4-1 are now back-populated withing BMS2 through the use of new fields. This table will still be applicable through 12/31/2025 at which time PennDOT will officially stop using these fields.

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| Table IP 2.3.14.4-1 Compliance Inspection Types and Applicable 7A03 Types for NBI/NTI Extract File | | | | | |
|---|---|---|---|---|---|
| **Compliance Inspection Types** | **Category Description** | **Applicable 7A03 Types** | **NBI/NTI Items computed from BMS Data *** | | |
| | | | **Req'd 7A07** | **Freq 7A09** | **Insp Date 7A01/7A08*** ** |
| **NBIS Compliance Inspection Types (Bridges and Culverts)** | | | | | |
| NBI | National Bridge Inventory Inspection | R, C, F, W | ** | 91 | 90 |
| NSTM | NSTM Inspection | R, C, F, I, W | 92 A | 92 A | 93 A |
| UW | Underwater Inspection by Divers | U, W, F | 92 B | 92 B | 93 B |
| SP | Special Inspection | I, R, C, W | 92 C | 92 C | 93 C |
| **NTIS Compliance Inspection Types (Tunnels)** | | | | | |
| NTI | National Tunnel Inventory Inspection | R, F | ** | D.3 | D.2 |
| | Damage Inspection | B | D.5 | ◊ | ◊ |
| | In-Depth Inspection | D | D.4 | ◊ | ◊ |
| SP | Special Inspection | I | D.6 | ◊ | ◊ |

* NBI Items are described in the SI&A Coding Manual. NTI Items are described in the SNTI.
** Required for all NBIS/NTIS structures as applicable - no separate data item is provided.
*** The more recent of 7A01 or 7A08 is reported to NBI
◊ Not captured by NTI

**2.3.14.5** INSPECTIONS REQUIRED BY NBIS/NTIS

A summary of the Compliance Inspection Types required by the NBIS and NTIS are listed in Table IP 2.3.14.4-1. BMS2 uses Item 7A07 Required Inspections to initiate compliance monitoring for the individual bridges.

<u>NOTES on 7A07 Required Inspection:</u>

- The 7A07 Required Inspection is key for other compliance monitoring data.
  - If 7A07 does not indicate "REQUIRED" for an inspection type, BMS2 does not update 7A08 Last Inspection and it does not report that inspection type with the NBI/NTI.
  - If that inspection type is required by policy and the 7A07 data box is improperly unchecked for that type, the bridge would be non-compliant with NBIS/NTIS based on the 7A08 data.

- Once entered into BMS2, Item 7A07 for the compliance inspection types is copied by BMS2/BMS3 for the subsequent inspections.
  - The engineer-in-charge should revise 7A07 and 7A09 after an inspection has determined that bridge conditions have changed sufficiently to warrant their revision.
  - NOTE: Having 7A07 data box checked to indicate a required Compliance Type Inspection does not necessarily mean that each required inspection type must be performed for every inspection. The required types of inspection due at each inspection will be governed by the interval specified in 7A09.

- For NBI/NTI bridges with a reduced inspection interval, enter the intervals for 7A07 NBI and 7A07 Special Inspection as listed in Tables IP 2.3.2.4-1 and IP 2.3.2.4-2.

- This will correctly result in occasions where NBI/NTI and Special Inspection are both due (i.e., every other year for a Special Inspection interval of 12 months). For these "dual due dates", perform an NBI/NTI inspection and check the 7A06 data boxes for both NBI/NTI and Special Inspection.

Due to changes in the 2022 NBIS/SNBI, PennDOT is working on modifying the inspection types and associated BMS2 data fields to better align with these revised documents. The fields shown in Table IP 2.3.14.5-1 are now back-populated withing BMS2 through the use of new fields. This table will still be applicable through 12/31/2025 at which time PennDOT will officially stop using these fields.

**Table IP 2.3.14.5-1 Compliance Inspection Types Required for NBIS and 7A07 Req'd Inspection Data**

| Compliance Insp Types | Req'd 7A07* | Compliance Inspection Type required when: | See Also | Frequency 7A09 |
|---|---|---|---|---|
| NBI | ** | NBIS Indicator (BMS2 Item 5E01) = Y | IP 2.3.2 | Table IP 2.3.2.4-1 |
| NSTM | ☑ or ☐ | Bridge has NSTMs (BMS2 Item 6A44 ≤ 4) | IP 2.4 | Table IP 2.4.7-1 |
| UW | ☑ or ☐ | Inspection by UW divers is needed to ascertain bridge substructure and/or foundation condition. | IP 2.6 | Table IP 2.6.2.4-1 |
| SP | ☑ or ☐ | A reduced interval (< 24 months) inspection is required. | IP 2.3.2<br>IP 2.3.5 | Table IP 2.3.2.4-1 |
| *7A07 checked box ☑ = Required, unchecked box ☐ = Not required<br>** There is no 7A07 NBI data box – All bridges require NBI inspections | | | | |

**2.3.14.6** INTERVALS FOR COMPLIANCE INSPECTION TYPES

The NBIS/NTIS requires the Compliance Type Inspections to be performed on a regular interval. Table IP 2.3.2.4-1 provides information on interval to be entered into BMS2 Item 7A09. For intervals of Underwater Inspections see Table IP 2.3.7.4-1. The owner may elect to perform inspections more frequently than required. Authorization for longer intervals must be requested from BIS and FHWA.

## 2.4 NSTM/FATIGUE

### 2.4.1 General

One of the most important aspects of steel bridge inspection is the determination of the bridge's potential for fatigue and/or fracture. Fatigue and fracture can lead to premature and possibly sudden failure of a portion of the bridge or of the entire bridge. Therefore, it is essential that fracture critical inspections be performed to identify these potential failures before they occur.

Fatigue failure of a material is the initiation and propagation of cracks due to repeated application of loads. Fatigue failures develop at stresses well below the material's yield point stress. Three factors are needed for fatigue in metal: tensile stresses, repetitive loading, and poor details that create high tensile stress concentrations. It is imperative that a fatigue crack is not left unchecked because it may propagate to a size that would trigger fracture. Fracture is the sudden failure by cracking of a member.

### 2.4.2 Hold for Future

### 2.4.3 Inventory of Bridges with NSTMs

The NBIS (§650.313(f)) requires the individual in charge of the bridge inspection organizational unit to:

- Determine and designate NSTMs on the individual inspection and inventory records for those bridges which contain NSTMs.

- Maintain a master list of the location and description of such members on the bridge and the inspection interval and procedures for inspection of such members.

To meet those NBIS requirements, the Department developed five BMS2 Items (6A44-6A48) to identify and classify bridges with NSTMs so that this required "Master List of Bridges with NSTMs" could be maintained in the BMS2 to assist in the inspection management of these bridges. In addition, the Department developed the BMS2 6A and NSTM/Fatigue Screens, "Agency Bridge" and "Inspection – NSTM/Fatigue", to record a summary of NSTM inspection findings on each bridge with NSTMs to assist the Districts and other bridge owners to have a snapshot of these critical bridge elements. The data in the 6A and NSTM/Fatigue Screens should be more fully documented in the bridge inspection reports.

All NBIS bridges with NSTMs, regardless of ownership, are to maintain inventory and inspection information in BMS2 through the use of the BMS2 Items (6A44-6A48), the 6A Screen, NSTM/Fatigue Screens and the Bridge Inspection Scheduling data (7A57-7A60).

The Districts are to monitor their inspection reports and those of local bridge owners to ensure these requirements for the inventory and inspection data for NSTMs are met.

### 2.4.4 Classification of NSTMs

Bridges deemed to be fracture critical have no load path redundancy and if the load path fails, the bridge, or a portion thereof, collapses. Stated differently, bridges with NSTMs are non-redundant structures. The other types of redundancy, System redundancy and Internal (or Member) redundancy, help to determine the criticality of non-redundant (or fracture critical) structures.

#### 2.4.4.1 LOAD PATH REDUNDANCY

A main load-carrying member represents a structure's load path. A bridge with four or more load paths is said to have redundant load paths and is defined as a redundant structure. Structures with load path redundancy are not fracture critical. A bridge with three load paths requires structural analysis to determine if it has load path redundancy. A bridge with only one or two load paths is defined as a non-redundant load path structure and is considered fracture critical. See AASHTO Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members for additional information.

#### 2.4.4.2 SYSTEM REDUNDANCY

Bridge structural types that provide continuity of load path from span to span are referred to as system redundant. A bridge with NSTMs with no system redundancy is more critical (susceptible to failure) than a bridge with NSTMs with system redundancy. System redundancy provides a mechanism to prioritize or rank bridges with NSTMs. It is not appropriate to apply the concept of system redundancy to bridges with load path redundancy (i.e., non-fracture critical bridges).

#### 2.4.4.3 INTERNAL REDUNDANCY

Internal redundancy exists when a bridge member contains several elements bolted or riveted together so that multiple local (internal) load paths are formed. Failure of one member element would not cause total failure of the member. Internal redundancy is also known as "member" redundancy. A built-up riveted plate girder is an example of a member with internal redundancy. Like system redundancy, internal redundancy provides a mechanism to prioritize or rank bridges with NSTMs. It is not appropriate to apply the concept of internal redundancy to bridges which are load path redundant. For additional information on internal redundancy, see AASHTO Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members.

#### 2.4.4.4 NSTM GROUP NUMBER – BMS2 ITEM 6A44

The inspection engineer is to classify each NBIS bridge according to its fracture critical nature in accordance with the guidance provided in Pub 100A. Possible Group numbers range from 1 to 9 in which Group 1 bridges are more fracture critical than Group 2, Group 2 are more critical than Group 3, and so on. Refer to BMS2 Item IF09 in Pub. 100A for additional NSTM detail guidance.

**2.4.4.5** CRITICAL RANKING FACTOR (CRF) OF NSTM

A numerical index, the Critical Ranking Factor (CRF), is used to prioritize the bridges within each of the NSTM groups. The CRF (6A49) is the sum of four digits that characterize the NSTM. Each NSTM will receive a CRF. The lowest member CRF controls the bridge's CRF.

The four components of the most critical NSTM are to be entered into BMS2 in Items 6A45-6A48 (see Publication 100A, BMS2 Items 6A45-6A48). The four components of the CRF are described below.

**2.4.4.5.1** Type of Member – CRF First Digit

The type of member is ranked by criticality based on judgment as to its use within a structure. Direct tension members, even if they may have no welding associated with them, are the most critical. Eyebars and hangers that have been repaired by field welding are highly susceptible to fatigue cracking and should be placed at the top of the list of structures with these members.

The next most critical members are subject to bending stresses and have been welded in the tension zone. Many of the problems being discovered on in-service bridges are associated with weld terminations or defects that are inherent to the welding process. Welds made in the field are especially susceptible to fatigue cracking. Even tack welds could initiate cracking under certain conditions.

Although members with bolted or riveted connections and members of plain rolled shapes are not immune from defects, field experience shows a very low record of failure due to fatigue cracking. Members of this type are least critical.

**2.4.4.5.2** Fatigue Crack Susceptibility – CRF Second Digit

This portion of the CRF determination is intended to assess the criticality of the "as fabricated" or retrofitted condition of the member as it exists in the field today. It closely parallels the allowable fatigue stress determinations contained in AASHTO LRFD Bridge Design Specifications, as used for new designs. The assumption is made that the lower the fatigue stress limit of a member, the more critical the member is. The AASHTO fatigue Category E' details are the most critical and have a ranking factor of 1. Plain, unblemished steel plates, or rolled shapes, are fatigue Category A and are assigned a ranking factor of 6. Needless to say, any defects such as section loss due to corrosion or flaws discovered through the bridge safety inspection would immediately change the ranking factor to 1, even for a rolled member, until the defect is adequately repaired.

Because welding process and procedure controls are more difficult in a field welding situation, all field welds, repairs, or attachments, should be given a ranking factor of 1.

Any member with details subject to out-of-plane distortion induced fatigue cracking will be given a factor of 1. (Examples: Tie-plates for girder or floor beam brackets, web plates where transverse connection plates are not rigidly attached to the girder, flanges at cross-frames, etc.).

**2.4.4.5.3** Material – CRF Third Digit

Welding of steel structures had a major influence on the ultimate safety of a steel tension member. Although steel may have an inherent flaw that could reduce the service life of the member, the majority of defects discovered on in-service bridges that could propagate to catastrophic failure are related to welding. This influence shows up in three ways:

1. Poor quality welding has inherent defects that initiate cracking.
2. Welded attachments create stress raisers that initiate and propagate cracking due to cyclic (fatigue) loading.

3. Welding of "non-weldable" steels or improper weld procedures creates a "brittle" condition, and if located in a single-stress-path member can fail catastrophically by unstable crack growth with little or no advance warning.

Items 1 and 2 above become apparent when visible cracks appear in the member. Ideally, the material properties will exhibit sufficient elastic or plastic deformation (stable crack growth) to allow crack detection and retrofit to take place prior to unstable crack growth and collapse. When Item 3, a brittle condition, exists in conjunction with either poor quality welding and/ or fatigue sensitive details, a "super-critical" condition may exist warranting special in-depth and frequent inspections.

Since the weldability of steel is not apparent without a chemical analysis, use the "Year Built" (BMS2 Item 5A15) as a preliminary indication as to criticality. Although steels used prior to the introduction of "weldable" steels may have had chemical properties suitable for welding, the inspector should assume "non-weldable" steel unless design plans are available, or a chemical analysis is performed indicating a weldable grade. Generally, eyebars were manufactured to the same specification as structural steel, so these comments apply to eyebars, plates and rolled sections.

In the absence of a chemical analysis, use the material characteristics from Table IP 2.4.4.5.3-1.

**Table IP 2.4.4.5.3-1 Material Properties for Coding CRF Digit -3 Without Chemical Analysis**

| Year Built | Primary Fabrication Process | Probable Grade of Steel | Weldable |
|---|---|---|---|
| Before 1957 | Riveted (See Note Below) | A7, A8, A94, or A242 | No |
| 1957 – present | Riveted (See Note Below) or Bolted or Welded | A7, A8, or A94 | No |
| 1957 – present | Riveted (See Note Below) or Bolted or Welded | A373 or A242 Weldable | Yes |
| 1965 – present | Riveted (See Note Below) or Bolted or Welded | A36, A441, or A242 | Yes |
| 1965 – present | Riveted (See Note Below) or Bolted or Welded | A7, A440, A8, or A94 | No |
| 1965 – present | Welded or Bolted | A36, A572, or A588 | Yes |
| 1965 – present | Welded or Bolted | A514 or A517 | Yes |
| Note: Specifications allowed welding of secondary members and miscellaneous details | | | |

Further, steel can have a chemistry suitable for welding but still exhibit low values of toughness as indicated by Charpy 'V' notch (CVN) testing. If CVN data is available, and it is above 15 ft/lb at 40° F, then a ranking factor of 2 or 4 is appropriate. Otherwise, a 1 or 3 should be assigned dependent upon weldability determination. A514/A517 steel (90,000+ psi yield strengths) has exhibited low toughness values in field situations. Therefore, even though it is a weldable grade, a factor of 3 should be assigned for all A514/A517 steel. For important Fracture Critical bridges where the toughness properties are not known, Charpy V Notch tests are recommended to better establish this key information.

NSTMs designed since 1977, should have been fabricated in accordance with a fracture control plan. If the fracture control plan was used, a factor of 4 (except for A514/A517) is appropriate assuming no unauthorized field welds are discovered.

**2.4.4.5.4** Cumulative Truck Traffic – CRF Fourth Digit

Although truck traffic is not always the cause of fatigue cracking, without question it is the predominant influencing factor. Therefore, routes that have been, or will be, subjected to high truck volumes are highly susceptible to fatigue cracking. The CRF $4^{th}$ Digit can be determined by using the table in Pub 100A for item 6A48.

### 2.4.5 Components of NSTM Inspections

NSTM Inspections look specifically at details that are susceptible to fatigue and/or fracture. Key components of NSTM Inspections are:

- Fatigue and Fracture Plan
- Field Inspection Results, including testing

- Bridge Analysis and Computation of Remaining Fatigue Life
- BMS2 data (6A26-6A29, 6A44-6A48, 6A and NSTM/Fatigue Screens)
- BMS2 Inspection Scheduling data (7A57-7A60) – used for NBIS compliance tracking.

The inspection engineer has to determine how each of these components are applied to individual bridges in order to properly monitor the fatigue and fracture prone details on the bridge so as to recognize, from field observations, when the conditions have changed and take appropriate actions.

**2.4.5.1** FATIGUE AND FRACTURE PLAN

The Fatigue and Fracture (F&F) Plan is a key first step in performing a thorough and complete investigation of the threat of fatigue and/or fracture to the bridge. It provides a "map" of NSTMs and their details on the structure to identify all fatigue prone or NSTM details for the inspectors. This assures that the conditions at all critical components will be inspected adequately, and the field results presented in an organized manner to enable the inspection engineer to ascertain the bridge's safety in a timely manner. The complete NSTM is required to receive a hands-on inspection if any portion is in tension. If a portion of the member is in compression and cannot be accessed easily due to girder depth/spacing, etc., a visual inspection of that portion may occur. The inspection procedures must document the specific locations where this is acceptable, with the neutral axis of the member and the tension zones clearly indicated.

The F&F Plan must be included as part of the bridge file and must be updated during every Routine Inspection. It provides a method for establishing and monitoring the history of the behavior of fatigue and fracture prone details on a structure with NSTMs. It should also identify methods of access required for the inspection. Appendix IP 02-H includes a blank cover page and a blank details table which shall be utilized on all F&F Plans. An excel version of this form along with a completed sample is available on the BMS2 home screen under the PennDOT Bridge Inspection Forms and Templates link.

A F&F Plan shall include the following:

- Sketch(es) of superstructure with locations of all fatigue and fracture prone details identified
  - Use grid diagram (framing plan) with detail locations labeled by letters or numbers and a legend explaining the number or letter scheme.
  - Use an elevation view for truss
  - Classify similar fatigue/fracture prone details as "types" (e.g. end of partial cover plate)
- A table of fatigue/fracture prone details indicating
  - Type of detail (e.g. end of partial cover plate, short web gap, etc.)
  - Location of each occurrence of detail
  - AASHTO Fatigue Category of detail
    - Category D, E, E', and worst category fatigue details (if no D/E/E') are required to be entered
    - Any other fatigue details with known issues are required be entered
  - Identify retrofits previously installed
  - Table can be organized by Span or type of detail
  - NSTM details should be entered per Span and can be grouped within the Span
- An inspection procedure is required for all fatigue/fracture prone details. Refer to BMS2 Item IF10 in Pub. 100A for examples.
- Specific instructions including inspection procedures for a limited scope Special Inspection used to satisfy a more frequent inspection interval.
- Completed F&F Plan Quality Control Verification Checklist (Appendix IP 06-C).

For bridges with NSTMs, the following documents are to be stored in the BMS2 Documents link, in addition to any hard copies maintained in the owner's permanent bridge file:

- Fatigue and Fracture Plan
- Design and Shop Drawings for Superstructure (when available)
- Fatigue Computations (when available)

Bridges with concrete-encased NSTMs pose a special challenge to bridge inspection. The F&F Plans for such structures are to contain bridge-specific instructions for inspection to:

- Identify portions of encased NSTMs of higher susceptibility to fracture or fatigue.

  - Where encasement is fully intact, visual observation alone may be used on an interval not to exceed 24 months when it will sufficiently ascertain the NSTM condition.
  - Where encasement is not intact, hands-on inspection is warranted.
- Identify portions of encased NSTMs with lesser susceptibility to fracture or fatigue.
  - Visual observation alone may be used on an interval not to exceed 24 months if it will sufficiently ascertain the NSTM condition. Otherwise, hands-on inspection is required.
- Identify portions of encased NSTMs that do not warrant hands-on inspection.
- Provide photos to document condition of concrete encasement, especially at fatigue or fracture-prone details, for F&F Plan. Update photos during subsequent inspections.
- Provide for testing (via sounding, etc.) of encasement integrity in critical locations on an on-going basis.

Submit F&F Plan of bridges with encased NSTMs to the BIS for review and comment in conjunction with FHWA. Without an approved F&F Plan noting special inspection instructions, a full hands-on inspection is required (see IP 2.3.6.3).

#### **2.4.5.2** FIELD INSPECTION RESULTS

The condition of each NSTM inspected should be noted in the field documentation. While noting any cracking, section loss, or deterioration is essential, a documented finding of “no cracking” or “no section loss” in an NSTM detail also helps the engineer track the overall condition of the bridge. The BMS3 reports generally need to be augmented with additional note sheets. Photos and sketches, properly referenced to field notes, are also part of a good NSTM Inspection.

The results of the field inspection should be carefully documented and compiled in accordance with the F&F Plan, again to assist in tracking the condition of the various details.

#### **2.4.5.3** BRIDGE ANALYSIS AND COMPUTATION OF REMAINING FATIGUE LIFE

Knowledge of the stress level that an NSTM detail is subjected is an important key to establishing the criticality of the detail and estimating its remaining fatigue life. See the Department’s DM4 Policies and Procedures Section 5.1 for the method of calculating remaining life.

#### **2.4.5.4** BMS2, BMS3, AND NSTM INSPECTIONS

The findings of the NSTM Inspection should be summarized on the BMS3 NSTM/Fatigue page to facilitate NBIS requirements and to provide a synopsis for the engineer in charge to plan needed follow-up actions. Entries are required for all NSTMs and NSTM details and connections. Refer to BMS2 Item IF09 in Pub. 100A for NSTM detail guidance.

Whenever an NSTM inspection is required, items 7A57 (Required inspection) and 7A59 (Inspection Interval) must be checked and the interval entered respectively.

### 2.4.6 Hold for Future

### 2.4.7 Hold for Future

### 2.4.8 Hold for Future

### 2.4.9 Fatigue

Fatigue is the tendency of a member to fail at a stress level below yield stress when subjected to cyclical loading. Three factors are used to determine probability for fatigue to occur or, conversely, the remaining fatigue life):

- **Stress Range of Cyclic Load**
  The probability of fatigue increases with increased stress ranges.
- **Number of Cycles of that stress range**

The probability of fatigue increases with increased number of load cycles for a given stress range.

- **Type of Detail**
  Certain details are more susceptible to fatigue than others are.
  AASHTO defines categories of details for their susceptibility to load-induced fatigue.

Fatigue damage can be categorized as due to either load-induced or displacement-induced stresses. Load-induced fatigue damage results from fatigue crack propagation at structural details subjected to the normal in-plane stresses for which they were designed. Displacement-induced fatigue damage is the result of secondary stresses caused by (out-of-plane bending) the interaction between longitudinal and transverse members not quantified in the design of the bridge.

Fracture may occur as a result of fatigue cracking or due to other conditions (see IP 2.4.10).

**2.4.9.1** LOAD-INDUCED FATIGUE DAMAGE

The primary cause of load-induced fatigue damage is low fatigue strength (poor) details. Some examples of load-induced fatigue prone details include:

- Groove welded joints, flanges and webs
- Ends of welded cover plates
- Welded attachments and welds in tension zones
- Coped connection plates, especially if the cope is flame cut
- Eyebar links and pin/hanger assemblies

Typically, these details are quantified by AASHTO as Category D or higher (more fatigue susceptible) in Table AD 6.6.1.2.3-1. The greater susceptibility of these types of details to fatigue damage was recognized by AASHTO by assigning them lower allowable stress ranges in design for a given number of stress cycles.

For a fatigue analysis, the live load stress acting on a detail is determined by distributing the load of a single design truck as specified in AD 3.6.1.4 and PD 3.6.1.4. If the fatigue live load stress remains below the allowable stress range (fatigue limit) for a given detail, that detail has infinite fatigue life. However, if the stress range exceeds the fatigue limit, the detail has a finite fatigue life and its remaining fatigue life can be determined (see DM4 PP 5.1.1.1.2).

If a crack or cracks have been detected at a detail, the above procedure for determining remaining fatigue life cannot be used. A more complex fracture mechanics analysis is required. At this point most of the fatigue life has been exhausted and a retrofit is needed.

**2.4.9.2** DISPLACEMENT-INDUCED FATIGUE DAMAGE

This type of fatigue damage is usually associated with relatively small out-of-plane movements. These relative movements, measured as small as hundredths of an inch, have been sufficient to cause displacement-induced fatigue damage. The following two conditions are required for this type of damage to occur:

1. A periodic or cyclic out-of-plane force or displacement.
2. An abrupt local change in stiffness where the force/displacement is applied.

Because this damage is caused by cyclic out–of–plane forces in localized areas, cracks can occur in either the primary compression or tension zones of a member. Cracks in a primary tension zone of the member are more critical, especially if their orientation begins to propagate perpendicular to the direction of the primary tension stress and should be retrofitted. A crack in the primary compression-only zone of a member may not propagate any further once the stiffness constraint has been relieved by the crack.

SMALL WEB GAPS: One common problem of displacement-induced fatigue damage occurs at small web gaps where high localized stresses are created because the connection is too rigid to allow the displacement to occur. Some examples of small web gaps include; floorbeam-to-girder connections, stringer-to-floorbeam connections, lateral connections to girder or floorbeam webs, diaphragm connections to girder or stringer webs. For welded connections, small web gaps are defined as gaps that are less than the greater of 4 to 6 times the web plate thickness or two inches (2”). For bolted connections, due to the additional stiffness of connection angles or other connection shapes, the web gap is defined as gaps that are less than the greater of 4 to 6 times the web plate thickness or four inches (4”) (see Figure IP 2.4.9.2-1).

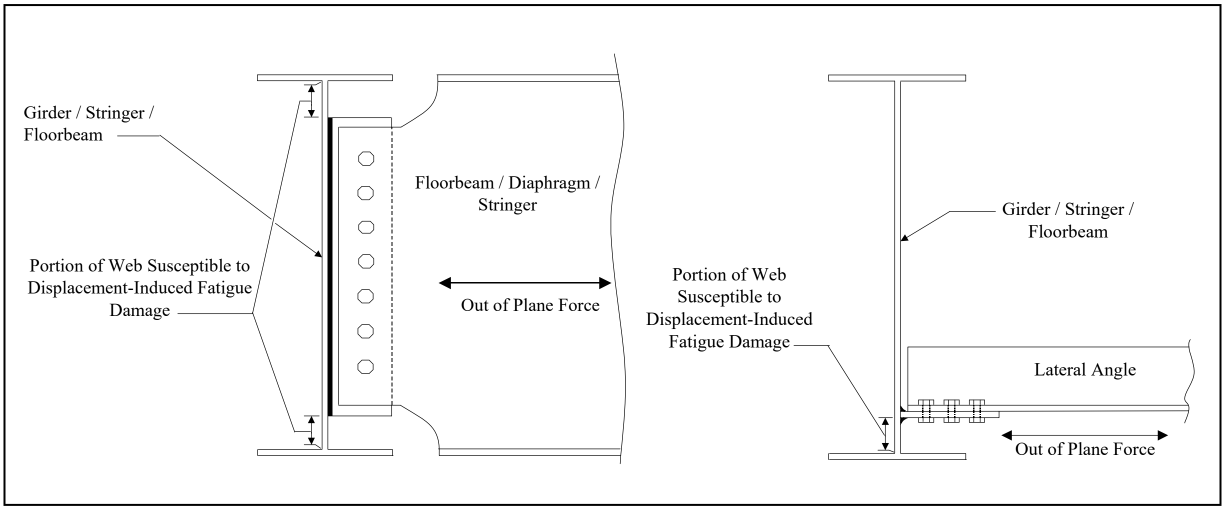


**Figure IP 2.4.9.2-1 Details Susceptible to Displacement-Induced Fatigue Damage**

**2.4.9.3** ANALYSIS OF DETAILS SUSCEPTIBLE TO DISPLACEMENT-INDUCED FATIGUE DAMAGE

It is difficult to analytically assess existing bridges for the remaining life of details with displacement-induced fatigue. The determination of stresses requires detailed refined analysis methods to model the actual conditions. The presumed FEM model for the un-cracked detail will not be valid once cracking has occurred. It may not be feasible to accurately estimate the number of cycles the details have experienced. Accordingly, the use of the analytical approach may be limited to special cases on critical bridges with approval of the Assistant Chief Bridge Engineer - Inspection.

**2.4.10 Other Details Susceptible to Fatigue and Fracture**

Other bridge details common on non-redundant bridges that are susceptible to fatigue and fracture include pin and hanger assemblies and truss gusset plates.

**2.4.10.1** GUSSET PLATES FOR TRUSS MEMBERS

Lessons learned from the 2007 collapse of the I-35W truss in Minnesota include the fact that truss gusset plates need to be thoroughly inspected and load rated to fully ascertain the safety of the bridge. Gusset plates may be deficient due to original design, section loss and/or overload.

Perform a hands-on inspection of all truss member gusset plates during each Routine Inspection and during Special Inspections when the gusset plate deterioration warrants. In the F&F Plan, specify the gusset plates to receive hands-on inspection during Special Inspections.

Measure remaining section of gusset plates and fasteners, noting any plate distortion, and document findings in such a way to fully support their analyses. See IE 4.3.5.6.6 for additional information on gusset plate inspection. Digital photos of gussets with contour lines indicating remaining section thickness is one best practice for inspection and documentation.

Also, include load ratings of gusset plates as separate members in the bridge analysis. See MBE Appendix L6A for additional discussion of gusset plate analysis.

**2.4.10.2** PIN AND HANGER ASSEMBLIES FOR NON-REDUNDANT GIRDER BRIDGES

Pin and hanger assemblies (with or without catcher systems) on non-redundant steel girder bridges are critical features for inspection. The pin and hangers are to receive hands-on inspection along with catcher system during every NBI inspection and during Special Inspections when their deterioration warrants. Specify portions of structure requiring hands-on inspection during Special Inspection in the F&F Plan. See IE 4.3.5.6.11 for further discussion.

In the F&F Plan, specify the documentation of the positions of pin and hanger assemblies and their catcher system in both longitudinal and transverse directions. Record temperature at time of inspection.

### 2.4.11 Fracture

Fracture is the sudden and unstable failure of a member in tension. Fracture occurs when a crack reaches a critical size relative to the material toughness, temperature, and loading rate. Fracture is known to have occurred due to member defects and at details with intersecting welds.

#### **2.4.11.1** MEMBER DEFECTS

Member fabrication defects such as notches, gouges, and nicks that occur in tension or reversal zones can lead to fracture because they create areas of unintended stress concentration. These defects may be the result of damage during fabrication, erection, rehabilitation, or due to collisions while in service. Fracture can initiate at or adjacent to these defects particularly if the orientation of these defects is perpendicular to the direction of the tension stress.

Welding also introduces defects into a member. Welds need to be examined for cracks within the weld metal, at the toe of the weld and in the base metal adjacent to the weld toe. Closely examine poor quality welds since fatigue damage is more likely to begin there.

It was common practice to use tack welds on built-up riveted members fabricated from 1945 through 1960. Of particular concern are truss bridges with tack welds used at lacing bars, tie plates, and batten plates on tension members. Tack welds were previously thought to provide a path for the crack to propagate from one of the member's elements to another, however research conducted by Dr. Robert Connor of Purdue University shows this is not the case. Reference NCHRP 20-07 Task 387, Maintenance Actions to Address Fatigue Cracking in Steel Bridge Structures for additional information.

NSTMs that have misplaced holes filled with plug welds should receive hands-on visual inspections. Past practice was to fill these holes with weld metal (plug weld) rather than leaving them open or installing bolts. Note if the weld metal is not ground smooth or of poor quality otherwise. Of particular concern are tension members on truss bridges.

Bridge inspectors are to carefully examine plug weld locations, looking for breaks in the paint and the formation of oxide powder or rust stains. If weld filled or plug welded holes are encountered in tension areas of NSTMs, the weld metal should be tested using NDT. A retrofit for this problem would be removal of the weld metal and reaming the hole for the installation of a high strength bolt. The bolt should be tightened to prevent crevice corrosion caused by the accumulation of water or debris.

#### **2.4.11.2** INTERSECTING WELDS

Intersecting welds on some bridge details in tension or reversal zones have led to brittle fracture and failure of main longitudinal bridge members without warning. Intersecting welds are defined as welds that run through each other, overlap, touch, or have a gap between their toes of less than 1/4". The intersecting welds of the web-splice-to-flange or flange-splice-to-web are not of concern here. Three-dimensional details with intersecting welds are the critical intersecting welds. These types of details introduce an out of plane force at the intersecting weld. Examples of some details that may have critical intersecting welds include:

1. Wind bracing connections to girder webs.
2. Floorbeam connections to girder webs.

More often than not, the bridge design and shop drawings will not indicate that intersecting welds are present on a bridge. However, excessive welding during fabrication or repairs has resulted in intersecting welds. Because these welds have caused girder failures on redundant, as well as, NSTM bridges, inspectors should examine steel bridges carefully for their presence.

If intersecting welds are found on a structure, immediately notify the District Bridge Engineer. Bridge closure may need to be considered. NSTM Inspections should be used to inventory and then monitor them.

Every NSTM Inspection must include a "hands-on" inspection of all intersecting weld details in tension or stress reversal zones, including the use of NDT when cracks are noted or suspected. Give priority and emphasis to bridges with NSTMs with intersecting welds. For instances where holes have been drilled to arrest cracks, inspectors must ensure that the crack is not propagating on the opposite side.

Important points to note about the failure caused by these intersecting welds:

1. The failure mode is brittle fracture which may be sudden and without warning.
2. The failure is not fatigue related and may occur under low stress levels or with very low cumulative truck traffic.
3. The failure is not material dependent and may occur in ductile material with good CVN properties.
4. While the welds may crack, failure of the member in compression zones is unlikely.
5. On an NSTM, the fracture of the weld may lead to complete structure failure.

Because the failure mode of intersecting welds is sudden and unpredictable nature, repairs and retrofits to cracked and uncracked locations must be given high priority.

The intensity and interval of these inspections are discussed in IP 2.3.6.3 and IP 2.3.6.4.

## 2.5 CULVERT AND STONE MASONRY ARCH STRUCTURE INSPECTIONS

### 2.5.1 Culvert Inventory and Inspection Requirements

Culvert and drainage structures that meet the definition of a bridge will be considered a bridge culvert. Bridge culverts will be inventoried and inspected as required of any bridge. For a complete discussion of Culvert Inspections, refer to FHWA's Bridge Inspector's Reference Manual.

### 2.5.2 Multi-Plate Corrugated Metal Culverts

Large-span multi-plate culverts, including box culverts, arches, pipe-arches, and circular pipes are relatively flexible soil interaction structures and more susceptible to failure when they lose their original global cross-sectional geometry. The inspection of these multi-plate culverts is to be sufficiently detailed to detect and monitor deformations (e.g., bulging; non-uniformity of the arch soffit, longitudinally or transversely; misalignment of plates; tearing) that could lead to a partial or complete collapse of the structure. Culverts under shallow earth fill are especially vulnerable to such deformations.

Bridge inspectors will monitor the integrity of the culvert's shape as the primary indicator of any structural distress. The inspection file is to contain sketches indicating the as-built geometry and subsequent measurements to monitor the structure's performance at a minimum of two cross-section locations. Paint marks on the culvert will assist the inspectors in ensuring measurements are taken at consistent locations. See IE 4.3.5.10 for more information on inspection of Corrugated Metal Culverts.

### 2.5.3 Stone Masonry Arch Bridges

Stone masonry arch structures are some of the oldest bridges in PA. Their construction varied from rough to ashlar (cut or dressed) stonework. Regardless of the construction method, these bridges are particularly vulnerable to:

- Scour of foundations – Any movement allowed by the foundation, either due to scour or to softer materials, can lead to failure of the masonry arch.

- Failure of closed spandrel wall – The gravity-type walls are susceptible to freeze-thaw cycles in the poorly drained roadway fill.

See IE 4.3.5.6.15 for further requirements for the inspection of these bridges.

#### 2.5.4 Interval of Inspection for Stone Masonry Arch Bridges

All stone masonry arch bridges in poor condition shall have a Special Inspection completed each year per Table IP 2.3.2.4-1 and Table IP 2.3.2.4-2 (12-month interval).

For Department-owned stone masonry arch bridges the biennial Routine inspection shall be performed between March 1 and April 30 in order to have these structures evaluated just after the seasonal freeze-thaw cycle. Owners of non-Department bridges are not required to meet the March/April timeframe for their structures but are encouraged to do so. Several spandrel walls have failed quickly in the early spring, resulting in the loss of roadway material that could have caused vehicles to plunge into the stream. Inspections in early spring may identify impending failures, allowing time to prevent sudden failure.

## 2.6 INSPECTION OF BRIDGES OVER WATER

### 2.6.1 PA's General Approach to the Safety of Bridges Over Water

Nationwide and in PA, more bridges are lost each year due to scour than any other reason. Many times, these bridge losses occur during regional or localized flooding and their loss from the transportation system can make recovery from the original weather event even more difficult. To combat this loss of structures from the transportation system and protect our valued infrastructure, PA uses a threefold approach:

- Underwater inspections of bridge substructure units are used to verify the structural condition of the underwater elements, to verify integrity of their foundations and to identify critical anti-scour maintenance needs.
- An assessment of the bridge's vulnerability to scour is made so that critical bridges can be identified for closer monitoring and scour countermeasures.
- During high water events, bridges whose safety is very susceptible to scour are required to be monitored.

PA's three-pronged approach meets the NBIS requirement for the State having procedures for underwater inspection, as per the FHWA's interpretation contained in their Technical Advisory "Evaluating Scour at Bridges" (T 5140.23 October 1991) as well as the memorandum from FHWA revising the coding of SI&A Item 113 dated April 27, 2001.

### 2.6.2 Underwater Inspections

Refer to IP 2.3.7 for Underwater Inspections.

### 2.6.3 Assessment for Bridge Scour

#### 2.6.3.1 GENERAL REQUIREMENTS FOR SCOUR ASSESSMENTS

INTRODUCTION: One of the more effective ways of preventing the loss of a bridge due to scour failure is to identify those bridges most likely to be vulnerable to failure due to scour. With this determination, called a scour assessment, the bridge inspectors and owners can concentrate inspection/monitoring efforts and remedial actions to mitigate conditions at bridges with critical vulnerability. To this end, scour assessments become an important part of bridge safety inspections in PA. Moreover, such assessments are required by the NBIS because they are deemed to be a key part of a comprehensive bridge inspection program.

Since 1982, PennDOT bridge design policy has required that all bridges built using federal funds are required to be designed against failure due to scour based on theoretical scour calculations from a Hydrologic and Hydraulic (H&H) analysis. In order to address all other bridges, and as approved by FHWA, PennDOT contracted with the United States Geological Survey (USGS) in the late 1990's to develop an algorithm, known as the SCBI

Scour Calculator, and to perform the initial site assessments for all NBIS-length state and local bridges in order to provide the necessary data for the initial determination of the SCBI code for these bridges. Once these initial codes were reviewed and approved by PennDOT, these SCBI codes, and other specific data collected by USGS, were entered into the Bridge Management System database. The USGS also developed a web-based database that allowed the PennDOT Districts to perform new runs of the Scour Calculator as warranted by changes in site conditions at the bridges. This USGS database is no longer available, but the SCBI Scour Calculator was incorporated into the BMS2 database and PennDOT's BMS3 inspection module such that it is available for use by PennDOT and consultant bridge inspectors.

PURPOSE OF SCOUR ASSESSMENTS: The main purpose of the scour assessment of an existing bridge is to determine whether the bridge is vulnerable to failure due to scour. A scour critical bridge is one whose foundation(s) has been determined to be unstable for the predicted scour conditions. The BMS2 Item 4A08 - Scour Critical Bridge Indicator (also NBI Item 113) is used to record a single digit coding indicating the bridge's vulnerability to failure due to scour. See BMS2 Coding Manual, Publication 100A for Item 4A08 coding instructions.

DESCRIPTION: The results of the scour assessment are to be used in conjunction with information from Routine, Special or Underwater bridge inspections to ensure that current stream (and bridge) conditions are used to evaluate and update the current scour vulnerability status of the bridge. Scour is a dynamic process, and changes in stream and streambed conditions (e.g., scour depth/location, aggradation, degradation, debris, installation of countermeasures) discovered during inspection can dramatically affect the vulnerability of a substructure unit foundation and must be considered. Accordingly, the inspection information and the scour assessment must be used together for the evaluation of the overall safety of the bridge. The inspection information is needed to validate and support the input parameters and results of the scour assessment. The scour assessment results are used to determine if scour poses a threat to the bridge. These scour assessments must be reviewed and updated when an inspection shows that scour conditions at the bridge have changed. Therefore, it is important that all relevant documentation be included in the bridge inspection file. For PennDOT standards on measuring and documenting scour, refer to Appendix IP 02-E.

The two acceptable methods of performing scour assessments in PA are:

1. Theoretical Scour Calculations – "Computed" Scour Assessment. The bridge (or measures/countermeasure) has been designed to resist failure due to scour as determined by a formal H&H Analysis (see IP 2.6.3.2). Bridges with a "Computed" Scour Assessment must also have all IN and IU fields completed on the Underwater screen of BMS2. If the observed field conditions no longer match the as-designed conditions used in the H&H Analysis, the coding for BMS2 Field IU03 should be revised from a "Computed" Scour Assessment to an "Observed" Scour Assessment. Measures are designed before a scour issue has occurred and countermeasures are placed to correct a scour issue that has occurred.
2. PA's Observed Scour Assessment for Bridges methodology – Observed Scour Assessment. The bridge has not been designed to resist failure due to scour. Therefore, the bridge must be assessed for scour vulnerability using data collected from plans and field observations and by running the SCBI Scour Calculator (see IP 2.6.3.3).

NBIS BRIDGE & STATE-OWNED 8'-20' STRUCTURE REQUIREMENTS: For all NBIS bridges over water and whose foundations are below the water surface elevation of the 500-year flood:

1. A Scour Assessment (Computed or Observed) for current conditions must be in the inspection file.
2. BMS2 Item 4A08 Scour Critical Bridge Indicator must be determined/verified for accuracy at each Regular and Underwater Inspection.
3. The scour "calculate" button should be clicked during each inspection for bridges over waterways to ensure any changes are reflected in individual SCBI code (IU27), overall SCBI code (IU04) and Item 4A08.

REQUIREMENTS FOR LOCALLY OWNED AND OTHER STATE AGENCY 8'-20' BRIDGES: Scour assessments are not required for non-State owned, non-NBIS bridges. However, the bridge owners are urged to apply them for any bridges at risk due to scour.

**2.6.3.2** SCOUR ASSESSMENT USING THEORETICAL SCOUR CALCULATIONS

A scour assessment of a bridge using the theoretical scour calculations is a method based on H&H analyses of the stream and bridge opening. Use DM4 PP Chapter 7 and FHWA Technical Advisory "Evaluating Scour at Bridges" (T 5140.23 October 1991) for guidance on the methodology. In good design practice, the bottom elevations of foundations are established considering the calculated scour depth. These design scour computations are to be used as the basis for the scour assessment and determination of the appropriate SCBI code and should remain in the bridge inspection file.

If existing scour at the bridge is deeper than the calculated scour, the theoretical scour analysis is not correctly modeling the real conditions and the scour assessment should be reanalyzed or assessed using the SCBI Scour Calculator. Any significant change in site conditions should also warrant revisiting the scour calculations or be evaluated using the SCBI Scour Calculator. The scour calculator should be run on every Routine, Underwater or any other inspections where the IN fields are updated and the waterway is evaluated.

For H&H analyses performed on bridges not previously designed to resist scour failure, the following guidance is provided for checking the resultant calculated depth of the theoretical scour to the substructure unit foundation:

1. For spread footing foundations:
   - If the calculated scour is above the bottom of footings - Not scour critical (Item 4A08 codes 8, 7, 5, or 4). Note: Footing may be partially exposed for Item 4A08 codes of "4" and "5".
   - If the calculated scour is below the bottom of footings founded on soil or erodible rock – Scour critical (Item 4A08 code ≤ 3).
2. For deep foundations (piles or caissons):
   - If the calculated scour is above the top of footings - Not scour critical (Item 4A08 code 8)
   - If the calculated scour is below the bottom of footing and above the bottom of pile/caisson – a structural analysis of the foundation unit is needed to determine its stability. If stable, the bridge is not Scour Critical (Item 4A08 code 4 or 5). If not stable, the bridge is Scour Critical (Item 4A08 code ≤ 3).
   - If the calculated scour is below the bottom of pile/caisson – Unstable and Scour Critical

**2.6.3.3** SCOUR ASSESSMENT USING PA OBSERVED SCOUR ASSESSMENT FOR BRIDGES

The Department developed an alternative method of scour assessment based upon the observance of geomorphic, hydrologic, and hydraulic features at the bridge site. This multi-disciplinary assessment, which has been approved by the FHWA, is seen as a cost–effective approach to meeting the NBIS requirements for evaluating existing bridges without theoretical scour computations. The Department developed this observed assessment method under an agreement with the United States Geological Survey (USGS).

Under the same agreement, the USGS performed the initial observed scour assessment of some 13,600 Department and local bridges over water. These initial observed scour assessment reports were reviewed for acceptance by the Department's District Bridge Units. All data for these initial observed scour assessments were captured for the Department in a Microsoft Access® database by USGS. The initial reports were distributed to the Districts in electronic format.

The methodology for PA Observed Scour Assessment for Bridges (PA OSAB) is outlined in the USGS Open-File Report 00-64 titled Procedures for Scour Assessment at Bridges in Pennsylvania. This report is available on the BMS2 login screen at: https://docs.penndot.pa.gov/Public/Bureaus/Bridge/BMS/Procedures-for-Bridge-Scour-Assessments.pdf.

The PA OSAB uses an algorithm in a Department software program named SCBI Calculator to determine the value for BMS2 Item 4A08 Scour Critical Bridge Indicator. The SCBI Calculator Manual, which is located on the BMS2 login screen at https://docs.penndot.pa.gov/Public/Bureaus/Bridge/BMS/Scour-Calculator-Manual.pdf provides guidance on the use of the SCBI Calculator. If the Item 4A08 - SCBI value from the PA OSAB is based on conditions valid at the time of inspection, it should be used in the inspection as the value for BMS2. The SCBI Calculator can be accessed by the inspectors through BMS2 and BMS3. Inspectors should recalculate the SCBI value by selecting the "Calculate" button on the BMS3 Waterway page or the Underwater Inspection Screen for BMS2 after updating the appropriate IU and IN fields as part of the inspection. Item 4A08 shall not have a value of

6 as this code indicates incorrect or incomplete data entry for the IN and IU data items used by the Scour Calculator (e.g. incompatible subunit type and foundation type, subunit type not coded, subunit foundation type not coded, etc.). Detailed information regarding Item 4A08 codes of 6 are provided in the Scour Calculator Manual and the Procedures for Scour Assessment at Bridges in Pennsylvania document. If the SCBI has a value of 2 or less, an FHWA memo dated April 27, 2001 requires the substructure condition rating be set equal to the SCBI coding. Furthermore, bridges with an SCBI of a 2 or less are deemed to be in critical condition.

PA OSAB data is to be reviewed for changes in site conditions during inspections. Because this method is based on field observations of features, maintenance of the pertinent data through Routine, Special, Underwater, and Post-Flood Damage Inspections is vital to the accuracy of its evaluation of the bridge's safety. The Scour Calculator can be used to compute correct values for the SCBI for new conditions. All changes are to be documented in the inspection report.

Table 1 in the SCBI Calculator Manual identifies the data components needed to run the calculator. BMS2 items on the IN Inspection – Underwater Sub Units screen marked with an "(SC)" in Publication 100A are required for each substructure unit in order to recalculate Item 4A08. They are as follows:

Item IN04 – Change Since Last Inspection
Item IN05 – Scour Hole
Item IN12 – OSA Pier/Abutment Foundation Type
Item IN13 – PA Foundation Type
Item IN14 – OSA Foundation Type
Item IN15 – Streambed Material
Item IN19 – Movement Indicator.

These field items should be verified during each Routine, scour related Special or Underwater inspection to determine the validity of the current SCBI code. The SCBI calculator should be run during each inspection to ensure the changes to the scour critical inputs are reflected in the BMS2 Item 4A08 code.

It is important to note that bridges which are designed with appropriate measures for scour conditions or if no measures are required as determined through an approved H&H Report still require an observed scour assessment, however, the SCBI will be based on the computed value (i.e., BMS2 Field IU03 = C) until field conditions indicate there is clear evidence that the above situations are not applicable (e.g., scour measures/countermeasure deterioration and/or failure).

**2.6.3.4** EVALUATION OF SCOUR MEASURES AND COUNTERMEASURES

The Department coding instructions for BMS2 Item 4A08 SCBI indicate that scour measures and countermeasures are to be properly designed or verified through analysis before they can be considered as effective countermeasures against scour. Designed measures and countermeasures can include streambed paving, gabion blankets, grout bags and rip-rap. It is important to understand the difference between a scour measure and a countermeasure. A scour measure is a device designed to resist scour. This is usually installed during new construction or during a rehabilitation project as part of an H&H analysis. A countermeasure is a device installed to correct a previous scour issue at the bridge. The difference is important because bridges with countermeasures require a Scour POA to ensure they resisted scour during an event (Category D). On the other hand, bridges with scour measures do not require a Scour POA.

The other common misconception is a bridge with an Item 4A08 value of 7 is better than a value of 5. The coding of a 7 is for bridges where countermeasures have been installed to mitigate an existing problem with scour and to reduce the risk of bridge failure during a flood event. Instructions contained in a POA have been implemented to reduce the risk to users from a bridge failure during or immediately after a flood event. The coding of a 5 is for a bridge foundation determined to be stable for assessed or calculated scour conditions. Scour is determined to be within limits of footings or piles by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge), by calculations or by installation of properly designed measures.

### 2.6.4 Scour Plans of Action

In accordance with the Title 23 of the CFR, Section §650.313(0)(2), a Scour Plan of Action (Scour POA) must be prepared for each NBI Scour Critical bridge. Similarly, Scour Critical State-owned 8'-20' structures must also have a Scour POA. Scour POA's document and describe the approaches used to monitor known and potential deficiencies and to address critical findings for bridges identified to be Scour Critical. Scour Critical bridges are to be placed into one of three categories based on the descriptions below. While bridges below in Category D are not Scour Critical, they still require a Scour POA because either action is required to protect exposed foundations (SCBI = 4) or protection has been placed and needs to be monitored (SCBI = 7).

- Category A bridges include all bridges with a BMS2 Item 4A08 SCBI code of 2 or BMS2 Item IN03 Observed Scour rating of 3 or less. Bridge is considered Scour Critical.
- Category B bridges include all bridges with a BMS2 Item 4A08 SCBI code of 3 **and** a BMS2 Item IN03 Observed Scour rating of 4. Bridge is considered Scour Critical.
- Category C bridges include all bridges with a BMS2 Item 4A08 SCBI code of 3 **and** a BMS2 Item IN03 Observed Scour rating of 5 through 9. Bridge is considered Scour Critical.
- Category D bridges include all bridges with a BMS2 Item 4A08 SCBI code of 4 or 7. These bridges are not considered to be Scour Critical but do require a Scour POA. Bridges in this category will need to be inspected after significant flooding events.

The Scour POA's for Scour Category A, B, C and D bridges include a monitoring program and post-flood inspection plan based on the above categories.

A sample Scour POA form is available in Appendix IP 02-A. These Scour POA's can be generated from a standard Crystal Report titled "Scour Plan of Action" that is available on Crystal Enterprise. Most information is pre-populated on these forms from BMS2. Other attachments (i.e., plan view scour sketch, cross sections, photos, etc.) may be added and a hardcopy is to be filed in the Bridge Inspection file. It is also required that an electronic file be stored in BMS2 under the Documents link.

FHWA requires that Scour Plans of Action be developed for each Scour Critical NBI bridge by local bridge owners and other bridge owners as well. However, PennDOT also requires a Scour POA for Scour Category D bridges. The procedures detailed here should be used or modified to better meet the owner's needs as approved by the Department. The local owners' procedures shall be approved by, and a copy kept on file, by the District in BMS2.

### **2.6.4.1** SCOUR CRITICAL BRIDGE MONITORING RESOURCES

Information is available to the Districts to prepare for and respond to flooding events. The following resources are available for use to comply with the monitoring requirements of Publication 23, Section 9.11:

- **Scour Critical Bridge Lists:** The Districts and Counties will maintain lists of Scour Critical bridges that are to be monitored during significant flood events. These lists can be generated from a standard Crystal Report titled "Scour Critical Bridge Category List" that is available on Crystal Enterprise and will be updated on a monthly basis. Districts and counties must use Bridge Watch alerts and these lists as the primary source to determine which bridges are to be monitored.

- **Scour Critical Bridge Maps:** GIS maps showing the locations of all Scour Critical bridges in categories A, B and C as well as non-Scour Critical bridges in Category D will be updated by Central Office and provided to District bridge units on a monthly basis. Districts and counties can use these maps to locate the bridges to be monitored. See Publication 23, Section 9.11, for the directory of map locations.

- **Field Manual:** A field manual titled "Scour Critical Bridge Monitoring Field Manual" is available in Appendix IP 02-A. Bridge monitoring personnel are to be familiar with the contents of this manual in order to have the basic knowledge necessary to perform bridge flood monitoring effectively.

- **Weather Forecasts:** Bridge Watch notifications, National Weather Service forecasts and Accuweather alerts provided to the County Maintenance offices or Engineering Districts contain essential information to assist with preparations for the start of flood monitoring of bridges. In some months, Accuweather alerts are issued when 2 inches or more of rain is forecasted and/or when Flood Warnings are issued by the National Weather Service. Department personnel should anticipate the need for monitoring when these alerts are received. Due to shorter duration, higher intensity (popup) storms, monitoring may be required for some bridges with less than 2 inches of rainfall.

**2.6.4.2** MONITORING

Monitoring Scour Critical Bridges shall be in accordance with Publication 23, Section 9.11.

Bridges are to be monitored in order of precedence with Category A bridges having the highest priority. Category A bridges on the Interstate must be monitored without exception (BPN = 1). Using the information presented on the Scour Critical Bridge Lists and the Scour POA's, an order of precedence can be further defined using information such as:

- Scour Critical bridge Categories A through C
- Business Plan Network (BPN), BMS2 Item 6A19
- Traffic Volumes, BMS2 Item 5C10
- Overtopping frequency of bridge and/or roadway approach, BMS2 Item 1A06
- Bridge configuration (length, # spans, support types, redundancy, etc.)

The recommended minimum interval for monitoring Category A bridges is once every four (4) hours from the onset of monitoring. The recommended minimum intervals for monitoring other bridges are once every 12 hours for Category B bridges and once every 24 hours for Category C bridges. Note that each scour critical bridge being monitored requires a minimum of two (2) visits. Discontinue monitoring per Publication 23, Section 9.11.

Notes: 1) High water events and flash flooding can be localized. Conversely, flood events can be widespread and may not occur for a day or two after rain has begun. In either case, flood monitoring of bridges is to be performed when Bridge Closure/Outage occurs (see Publication 23, Section 9.11).
2) Except for Category A bridges on the Interstate, the above monitoring procedures may be waived when a Declaration of Emergency has been authorized.
3) Category A bridges must be closed at such time as the water level reaches the low chord of the bridge (i.e. pressure flow condition). If it is deemed necessary to keep a bridge open, (e.g., for emergency vehicle passage or emergency evacuations), then the bridge must be continuously monitored (i.e. placed under a 24/7 watch) until flood waters subside and a post-flood damage inspection has been completed.

**2.6.4.3** INSPECTION

Post-Flood Damage Inspections are required for Scour Critical Bridges Categories A through C, and Category D bridges in areas or watersheds where significant flooding events have been reported by the counties or bridge closures have occurred. Depending on the size of the area affected by flooding, numerous flood damage inspections may need to be completed in a short time frame. Statewide bridge inspection open-end contracts may be a source for accomplishing these inspections.

Once flood waters have subsided, Scour Category A, B, C and D bridges are to be inspected for flood damage as follows:

- Category A: Inspect all bridges for flood damage where a significant flooding event was experienced. Bridges in this category that have been closed due to approach roadway or bridge overtopping or pressure flow must be inspected for flood damage before reopening to traffic.
- Category B and C: Inspect all bridges for flood damage where a significant flooding event was experienced or if the bridge has been closed due to either approach roadway or bridge overtopping or pressure flow must be inspected for flood damage before reopening to traffic.
- Category D: Inspect for flood damage after each significant flooding event or if the bridge has been closed due to overtopping or pressure flow.
- Refer to the Scour POA's for additional guidance.

Any additional bridge not defined in the bullet points above that was subjected to overtopping or pressure flow shall be subjected to a post-flood inspection. This inspection should be completed within 7 days of the bridge being reopened to traffic.

All Category A bridges which have been closed as a precautionary measure are to be inspected first so that bridges can be re-opened as quickly as possible. It is recommended that the inspection of the remaining bridges be prioritized in order of Category (Category A, B, C, D) since these represent the level of vulnerability to scour.

Scour Critical Bridges that have been closed due to the flood event require District Bridge Engineer approval prior to re-opening.

**2.6.4.4** REPORTING

Bridge flood monitoring personnel are to complete a Bridge Flood Monitoring Log (paper or in Bridge Watch) for each Category A, B or C bridge monitored. Entries are to be made on the log each time the bridge is visited. See Appendix IP 02-A for a sample log sheet.

Bridge closures are to be reported by the counties using the Road Condition Reporting System (RCRS) and in the Bridge Watch system. RCRS reports all active bridge and roadway closures. Bridges that have been closed as a precaution, such as those that have gone into pressure flow or have overtopped, are to be reported to RCRS with a “Cause” description of “Bridge Precaution.” Those bridges closed due to scour failure are to be reported in RCRS with a “Cause” description of “Bridge Outage.” Those bridges closed due to flooding or possible washout failure are to be reported in RCRS with a “Cause” description of “Bridge Flood Washout/Damage.” Bridge Watch reports scour critical bridge issues. For bridges closed in Bridge Watch, the user should select the appropriate closure option.

- During flood events:
  - Per Publication 23, Section 9.11, Highway Maintenance Managers are to report to the District Bridge Engineer by phone or email on a daily basis. Counties may now use Bridge Watch to update the District Bridge Engineer.
  - District Bridge Engineers are to report all counties where active flood monitoring is occurring to the Assistant Chief Bridge Engineer - Inspection on a daily basis. If bridges are reported in Bridge Watch in real time, then this satisfies this requirement.
- After flood events:
  - Highway Maintenance Managers will assemble and submit monitoring logs per Publication 23, Section 9.11. These logs may be scanned and submitted electronically. Highway Maintenance Managers may upload these logs to Bridge Watch, or monitor bridges directly in Bridge Watch from a desktop or using the mobile app in lieu of monitoring logs and weekly submittals. Highway Maintenance Managers will have the authority to add local partners as users in Bridge Watch.
  - Using Bridge Watch, the information reported by the County Managers, information reported in RCRS for bridge closures, and rainfall accumulation data, the District Bridge Unit will screen and prioritize the bridges to receive post-flood inspections by bridge inspectors. Scour Critical bridge lists, monitoring data from the counties and Scour POA forms are to be referenced for this effort.
  - District Bridge Engineers are to submit weekly status reports to the BIS identifying the bridges monitored and the summary results for bridges inspected following flood events using the High Water Inspection Weekly Report Form provided in Appendix IP 02-A. Districts may use Bridge Watch for all reporting in real time in lieu of status reports.

## 2.7 INSPECTION OF CLOSED BRIDGES

### 2.7.1 Purpose of Closed Bridge Inspections

When a public road bridge is closed to vehicular traffic but not removed from the site, continued inspection is required on a regular basis to assure adequate safety to the public having access on or beneath the structure. Accordingly, assure that necessary barricades for vehicles and/or pedestrians are in place. The physical integrity of the structure must be regularly assessed to ensure that a partial or total structural failure will not occur and endanger the public, even with no one on the bridge.

CRITERIA FOR CLOSING A BRIDGE:

- Bridge cannot safely sustain the minimum 3 Ton posting.
- Bridge condition rating less than or equal to 1.
- Water level reaches low chord of the bridge for Scour Critical Category A bridges (i.e. pressure flow condition).
- Bridge cannot safely maintain vehicular or pedestrian traffic based on engineering judgment or as directed by District Bridge Engineer.

If it is an NBIS bridge and the crossing remains on the inventory of public roads, it must be inspected in accordance with NBIS and Department standards. Closed bridges greater than or equal to eight feet (8') in length owned by the Department or carrying State Routes must be inspected in conformance with those same standards. Although a bridge may be closed, the inspection must be current. Federal-aid funding eligibility is maintained if rehabilitation or replacement is identified on the 12 Year Program.

### 2.7.2 Description of Closed Bridge Inspections

A safety inspection of a bridge closed due to structural conditions is similar to a Routine Inspection in the kinds of inspection data that must be collected. In general, rate each inspection item without being influenced by the fact the bridge is closed. For example, the ratings for BMS2 Item 4A10 Deck Geometry Appraisal Items would not increase because the ADT had decreased (to zero). Maintenance needs should list all needed activities as if the bridge were still in service.

The closure barricades must be checked for integrity and effectiveness to maintain public safety. Permanent, fixed type barricades of concrete median barrier, steel guide rail, or other fixed type barrier should be installed in a manner that positively prohibits vehicles from the bridge. If the bridge is to be closed to pedestrians, a steel chain link fence, or other suitable barrier that prohibits pedestrian access should also be installed. If pedestrians are permitted to use the bridge, the bridge's structural safety for AASHTO's pedestrian loading must be verified at each Closed Bridge Inspection. Appropriate signing must also be in place, both at and in advance of the closed bridge.

BRIDGES CLOSED FOR CONSTRUCTION:

- When a public road bridge has been closed completely for replacement, it is no longer necessary to keep the inspection record current. For Department bridge projects, a bridge being replaced essentially becomes the property of the contractor when the project starts. However, if public pedestrian traffic is to be maintained on a bridge otherwise closed to vehicles, the responsibilities for the safety of the bridge and the need for inspection should be specified in the construction contract.
- If an NBIS bridge is partially closed to vehicular traffic for a staged construction project (either rehabilitation or replacement), it is still part of the public road and the open portion is to be inspected as a Routine Inspection on the regular cycle.
- If an NBIS bridge has been completely closed for rehabilitation, re-inspection during construction is not required. However, upon the essential completion of work and prior to the bridge going back into service, a Routine Inspection (or Initial Inspection for extensive restorations) is to be performed. The inventory and inspection data describing the bridge's rehabilitated condition must be entered into BMS2 within 3 months of the bridge going back into service.

### 2.7.3 Intensity of Closed Bridge Inspections

The level of effort required to perform a Closed Bridge Inspection will vary, as do other inspections, according to the structure's type, size, design complexity, existing conditions, and location, but is generally much less than Routine Inspections of in-service bridges. The criticality of the conditions that necessitated the closing and the risk of collapse must be considered when determining the scope of inspection. The level of scrutiny that the portions of the bridge not critical to public safety receive may be reduced from the intensity of a Routine Inspection, at the discretion of the District Bridge Engineer.

The focus of the Closed Bridge Inspection is to determine if the bridge is safe to remain in place in its current condition. If pedestrian traffic is allowed, the safety of the bridge to carry this loading is to be determined.

Structural analyses of closed bridges with significant changes in structure conditions since the initial closure may be warranted.

### 2.7.4 Interval of Closed Bridge Inspections

The maximum interval of inspection of closed bridges is 24 months. More frequent inspections may be warranted for bridges in critical condition.

## 2.8 RAILROAD BRIDGE INSPECTIONS

The inspection of bridges that carry or cross railroads requires attention to safety and compliance with special rules of the railroad. For their own protection, inspectors from the Department and the bridge owner personnel are to be instructed to exercise extreme caution when working near the railroad tracks, electrified lines, high speed trains and other railroad related hazards and operations. Typically railroad owners require that all inspectors working on their property or in danger of fouling their track(s) receive an annual training certification. It is the responsibility of the inspector to research the requirements of the specific railroad and be in compliance with those requirements.

### 2.8.1 Railroad Notification

Where a highway bridge involves a railroad crossing that has been abolished by PUC Order, notification to the railroad is not required.

If portions of a highway bridge over a railroad need to be inspected within the railroad's right-of-way, notify the railroad prior to performing the inspection. Railroad right-of-way varies for each railroad. As a precaution, or when in doubt, regarding railroad right-of-way, notify the railroad.

When it is known that a highway bridge inspection will take place within the railroad right-of-way, follow the guidelines contained below. Where inspection personnel would be exposed to potentially dangerous conditions due to equipment or railroad (e.g., electrified lines, high speed trains or similar conditions), the railroad must be notified in advance and the inspection is not to proceed until arrangements have been made with the railroad. Normally, the railroad will issue a right-of-entry permit with terms and conditions that must be followed during entry.

The railroad must be notified, at least twenty (20) calendar days in advance, of the intent to enter onto their right-of-way to conduct an inspection whenever any one of the following conditions is present:

1. Equipment (such as a bridge inspection crane) or other aids (such as scaffolding) is required in the span over the railroad.
2. The bridge is located over a railroad that is electrified.
3. There is any possibility of physical interference with railroad operations.
4. There is a dangerous condition for the bridge inspection crew due to high-speed railroad operations, close horizontal clearances or other similar conditions.
5. Inspection involves work within or above the zone measured 12 feet horizontally from the center of track rails.

Notification to the railroad must include a detailed description of: work to be performed, the number of people in the inspection party, description of any equipment that will be used (bridge inspection crane, scaffolding, etc.), anticipated length of time of inspection, and other pertinent information.

### 2.8.2 Insurance Requirements When Working Within the Railroad Right-of-Way

Depending upon the scope of work involved and the type of railroad operations involved, the affected railroad, when contacted, may require Railroad Protective Liability Insurance coverage and/or watchman and flagman protection.

Requirements for Railroad Protective Liability Insurance will be established by the operating railroad and must be furnished prior to conducting the inspection by the Department and/or its consultant. The Department cannot provide this insurance. The railroad can purchase the insurance and be reimbursed for the premium either directly by the Department or through its consultant.

### 2.8.3 Railroad Flagmen or Watchmen Requirements

Requirements established by the affected operating railroad for watchmen and flagmen during the inspection will be determined prior to the performance of the inspection and strictly adhered to by the inspectors.

### 2.8.4 Cost of Special Items for Inspection of Highway Bridges over Railroads

When the Department's consultant performs the inspection, insurance (for railroads) and all watchmen and flagmen and other railroad service fees should be paid directly to the railroad by the consultant. The consultant's agreement with the Department must have provisions to pay costs of services by others for such railroad costs. At the option of the District, a separate agreement may be entered into between the Commonwealth and the railroad, to reimburse the railroad directly for the costs of insurance, watchmen, flagmen and other railroad services.

When consultants perform the inspections, they can purchase the insurance on their own or utilize the insurance coverage offered by some of the railroads under the Strike-Off Letters. The consultant shall pay such costs and bill the Department under the terms of their agreements with the Department. Because the Department is self-insured, Norfolk Southern does not require the Department to purchase railroad insurance when the Department’s own forces perform the inspection.

For non-Department bridges, similar procedures are to be used.

For NBIS bridges, the cost of these special items for railroad bridge inspection is eligible for 80% Federal funding.

## 2.9 BRIDGE AND STRUCTURE EMERGENCIES

### 2.9.1 Reporting Bridge and Structure Emergencies

Whenever a serious bridge or structure problem or emergency occurs on a Department or local route, the District Bridge Engineer is to report the situation to BIS. BIS will review the incident and report as needed to the Executive Staff. Serious bridge/structure incidents include, but are not limited to:

- Distress in primary structural members to the point where there is doubt that the members can safely carry the loads for which they are subjected, and partial or complete failure of the bridge is a possibility. Consideration shall be given to the location and redundancy of member affected and the consequence of deficiency.
- Scour at or under the substructure is such that significant movement is likely which could cause the bridge to collapse.
- Abutment movement or distress that is so excessive that there is a clear possibility that it may not be capable of supporting the superstructure and partial or complete failure is a possibility.
- Suspected cracks in pins or hangers of two girder or truss bridges.
- Fire on or under the bridge, vehicle collision damage, or other human-related damage.
- Collapse or failure of highway-related structures such as noise walls, sound walls, retaining walls, sound structures, etc.
- Any bridge related incident that creates a significant traffic accident or congestion on the Interstate system or expressways.
- Any bridge-related incident involving a vehicle with a Heavy Hauling Permit.
- Any situation where the structural integrity of the bridge is such that its safety is in question. Deficiencies may include, but are not limited to scour, damage, corrosion, section loss, settlement, cracking, deflection, distortion, delamination, loss of bearing, and any condition posing an imminent threat to public safety.
- Failure of critical systems or components of critical systems within a tunnel.
- Any structural issues causing a full or partial unplanned bridge closure.
- Any NSTM which is rated in serious or worse condition (condition rating 3 or less).

- Any deck, superstructure, substructure or culvert component to be rated in critical or worse condition (condition rating 2 or less).
- A channel condition or scour condition to be rated in critical or worse condition (condition rating 2 or less).
- Immediate load restriction or posting, or immediate repair work to a bridge, including shoring, in order to remain open to public vehicular traffic.

The purposes for this bridge emergency reporting process include:

- To establish a direct communication link between the District, Central Office, and Incident Management as the problem unfolds, providing the latest information to the Executive Staff; and in the case of emergencies involving the NHS, to the FHWA.
- To determine if appropriate steps for bridge safety are being taken by Districts or Locals.
- To provide timely, additional resources (e.g., technical assistance, emergency funding) to the District if needed.
- To build a Statewide expertise in resolving emergencies that will save time and money on future problems.

Appendix IP 02-G gives additional guidance for inspection following various emergency events including a sample flow chart for communication during a bridge emergency, inspection guidelines following a seismic event and guidelines for post-fire bridge evaluation.

BRIDGE PROBLEM REPORTS: The Bridge Problem Report (BPR), created within BMS2, will be the standard method of documenting bridge and structure problems. The purpose of the BPR is to present a concise "news" report to executive staff and other critical responders on a bridge incident as it unfolds. It is not intended to be a final report on a problem, but a method to provide timely, key information to those who may be part of the response team, including:

- Deputy Secretary for Highway Administration
- Chief Bridge Engineer
- Traffic Engineering and Permits Section
- Chief Executive
- Federal Highway Administration (Bridge Section)

The completed BPR is sent to the Deputy Secretary for Highway Administration, other executives and staff as listed above, the pertinent District Executive and ADE-Design, and other key staff as soon as possible. The Central Office Bridge Inspection Section (BIS) will also distribute, at its discretion, completed BPRs to all District Bridge Engineers for their information. A copy of the BPR is automatically saved in BMS2 under the Documents screen upon finalization of the report. BIS maintains a database of BPRs within BMS2 for historical record and additional research. BPRs are internal documents and are not for general release, especially as they may contain incomplete or unverified information from an ongoing investigation. Information suitable for public release may be gleaned from them.

### 2.9.2 Instructions for Reporting Bridge and Structure Emergencies Using the BPR

1. The District Bridge Unit is to contact BIS by telephone as soon as a problem is known to alert Central Office of a situation that may be unfolding. This allows BIS to understand the nature of the problem, determine the need for reporting to up-line staff and begin marshalling resources that may be needed.

2. BIS determines immediately if a BPR is to be filed. Examples of situations that would require a BPR include collisions, flooding, and any other events that cause significant bridge damage and/or closures.

3. The District generates a draft BPR from the Bridge Problem Report screen in BMS2 as soon as possible and notifies BIS via email that it is ready to be reviewed. Refer to Publication 100A, Sections 2.19, and BP for instructions on how to generate a BPR. BIS is to review and finalize the BPR in a timely manner.

4. If additional information becomes available or when follow-up tasks are completed, the District Bridge Unit will forward the information to BIS. BIS will update the BPR and determine if a follow-up BPR should be distributed.

### 2.9.3 Emergency Bridge Restrictions and Special Hauling Permits

When a bridge is no longer able to carry its intended loads, it is imperative for public safety to prevent further damage or collapse by controlling traffic on the bridge. The need to prevent overloads on a weakened bridge justifies a thorough and urgent response.

For such situations, the Department may impose emergency restrictions on the bridge that include closing, vehicle weight restrictions, lane closures, prohibition of permitted vehicles, and other traffic control deemed necessary. The emergency actions (determined by the District Bridge Engineer) depend upon the bridge conditions and, in large part, the likelihood of overloads. Because Special Hauling Permits are issued in advance of the actual move, it is more difficult to prevent overloads by heavy vehicles with previously-issued permits than traffic generally. When emergency bridge restrictions are needed, the Emergency Bridge Restrictions and Special Hauling Permits Action Plan is to be followed. A copy of the Action Plan is in Appendix IP 02-C.

The nine steps in the Action Plan can be summarized into 3 main activities:

1. The District Bridge Engineer determines the need and type of an emergency restriction based upon accident reports, bridge inspection reports, bridge analysis/rating, etc.
2. The District Bridge Engineer meets with other members of the ad hoc team of District managers to ascertain other ramifications of the emergency and assign resources to resolve the issues.
3. Physical restrictions are put in place while APRAS restrictions are implemented.

Department Permit and Bridge staffs have gained experience by addressing past emergency bridge restrictions and the Action Plan was developed by the Motor Carrier Division as a result. This Action Plan identifies up to nine steps that may be needed to improve response time and communication to motor carriers operating under various Permits when there is a future emergency bridge restriction, particularly when the affected bridge is carrying an Interstate highway or major Traffic Route.

It is anticipated that all nine steps will need to be pursued in response to an emergency bridge restriction when the bridge is carrying an Interstate highway or major Traffic Route.

### 2.9.4 Bridge or Tunnel Collapse

All bridge, structure or tunnel collapses are to be reported immediately to the Secretary and the Deputy Secretary for Highway Administration. BIS will report such failures to the Executive Staff based on information received from the Districts.

In the event of such a failure, the District is to establish a “Structure Collapse Team” to investigate the cause(s). The procedures for such teams have been established in the Department Publication 220, "Bridge Collapse or Tunnel Failure Board of Inquiry Investigation Teams".

PUBLICATION 220: Publication 220 is intended to serve as a handy reference for guidelines and actions to be taken in the event of a bridge collapse or failure of a tunnel’s structural components. It includes names of District and Central Office personnel nominated by the Districts and the Central Office to be responsible to respond to emergencies. These persons are to collect facts, report findings and highlight issues to improve bridge and tunnel safety practices.

Home and cellular telephone numbers are also included in Publication 220 to facilitate contact during emergencies. Because of the personal nature of these phone numbers, Publication 220 is considered to be confidential to pertinent Department personnel and is not available for public release. The Bureau of Bridge is responsible to update the names and phone numbers in Publication 220 annually.

The District maintenance and construction staffs are to be aware of the importance of a structural failure investigation and are to be instructed not to move or remove debris from the site, if possible. This evidence is extremely important in determining the causes of failure.

In the event of a bridge collapse or the closure of a structure or portion of a structure on an interstate or a structure with a significant impact on the traveling public, immediately contact BIS with as much structural information as is available.

## 2.10 INSPECTION OF NON-HIGHWAY BRIDGES AND STRUCTURES OVER STATE ROUTES

This section is applicable to all non-highway bridges and structures, except railroad bridges and sign structures over State Routes. and retaining walls. See IP 1.7.3 for a general discussion of the Department's and owner's responsibilities for inspection. The technical requirements for safety inspection of railroad bridges, sign structures and retaining walls are contained in IP 2.8, IP 2.11 and IP 2.15 respectively.

For the purposes of this manual, the term "overhead bridge" will be used to encompass all types of non-highway bridges and structures over a public highway.

### 2.10.1 General Requirements for Overhead Bridge Inventory

INVENTORY REQUIREMENTS: NBIS requires that all bridges or structures greater than 20 ft in length over Public Roads are to be inventoried and their data stored in the Department's BMS2. All bridges or structures, regardless of their length, over State Routes are to be inventoried and their data stored in BMS2.

Other miscellaneous structures over or alongside State Routes are to be inventoried in BMS2 as follows:

1. Cantilever and Overhead Sign Structures – inventory in BMS2.
2. Retaining Walls – inventory in BMS2.
3. High-Mast Lighting – may be inventoried in BMS2.
4. Utility and other structures (e.g., Pipe trusses, conveyor belts) – inventory in BMS2.
   Minor structures with vertical clearance greater than 18 ft. may be exempted from inventory in BMS2 at the discretion of the District Bridge Engineer.
5. Traffic signal standards, lights, and other such items – do not inventory in BMS2.

AGREEMENTS GOVERNING THE CROSSING: The District is to ensure that the non-highway bridges over State Routes and those bridges not involving railroads are governed by a formal agreement between the bridge owner and the Department. The appropriate type of agreement, either HOP (Highway Occupancy Permit) or HOA (Highway Occupancy Agreement), is to be used at each bridge. The responsibilities of the owner should be clearly outlined in the agreement. Also, the agreement is to outline provisions for the inspection of the bridge and recoupment of costs by the Department in the event the owner fails to inspect the bridge and report the same to the Department in a timely manner.

For non-railroad bridges, record and maintain the appropriate HOA or HOP identification number in the BMS2 Item VM05 PSC-PUC Number for the appropriate governing document with the following instructions:

- <u>HOP or HOA bridges</u>:
  Digits 1-3: Enter HOP or HOA
  Digits 4-11: Enter identifying HOP/HOA Number

- <u>For other agreements (written or informal)</u>:
  Digits 1-3: Enter XXX
  Digits 4-11: Enter number associated with agreement. If none, enter 99999

### 2.10.2 General Requirements for Overhead Bridge Safety Inspections

The inspection of these non-highway bridges is similar to Routine Inspections of highway bridges. Because of the many types and features of existing overhead bridges, this section cannot list a complete set of specific inspection requirements. The scope of work for inspection of these overhead bridges is to be detailed in the agreement document (HOA, HOP, etc.) governing the crossing.

The General Scope of Work for the Safety Inspection of State and Local Bridges contained in Appendix IP 01-F is to be used and adapted as the inspection specifications for each individual structure. See Appendix IP 01-I for minimum inventory and inspection items required for overhead bridges which is less than the requirements for State and local Bridges. The specific scope of safety inspections for an overhead bridge or structure, including any needed load ratings, must be acceptable to the District Bridge Engineer.

Load ratings are considered part of the overhead bridge inspection process just as they are for highway bridges. If appropriate, underwater inspection requirements for substructures should be included. Overhead bridge safety inspection reports must be signed and sealed by a Pennsylvania Professional Engineer.

For longer bridges and structures, the inspection report to the Department may be limited to only those spans over the highway ROW and the substructure units supporting those spans. The District Bridge Engineer must approve the elimination of portions of a bridge from these inspection requirements. Bridge owners are encouraged, but not required, to inspect remaining portions with the same intensity.

For building-to-building passageway bridges, the structural components may be covered by siding, masonry, etc., that would interfere with an inspection using normal bridge techniques. These architectural facades also prevent the deterioration normally suffered by bridge components exposed to the weather. The scope of these inspections must be developed on a case-by-case basis.

Safety inspection reports and data of all bridges over State Routes must be submitted to the Department for its review and acceptance.

While this section was developed for bridges over State Routes, other roadway owners are encouraged to adopt it for use for non-highway bridges over their roadways.

### 2.10.3 Interval of Overhead Bridge Safety Inspections

All bridges and structures, not including sign structures or retaining walls, over State Routes are to have a bridge safety inspection on an interval no greater than 24 months. The District Bridge Engineer, at their discretion, may require inspections more frequently than 24 months if structure and/or site conditions warrant.

The inventory data for all bridges and structures over State Routes shall be verified on an interval no greater than 24 months. An inspection of the highway environs of the bridge is to be made at the same time.

## 2.11 SIGN STRUCTURE SAFETY INSPECTIONS

The purpose of the sign structure safety inspection program is to verify each sign structure's inventory data, to determine its physical condition and maintenance needs and to record the same in the Department's BMS2.

### 2.11.1 Types of Sign Structures

Sign structures are typically constructed of either galvanized steel or aluminum. There are also some painted and unpainted weathering steel structures.

The five basic types of sign structures in PA are as follows:

OVERHEAD - consisting of one or more horizontal members supported at each end. Overhead structures may be multi-span. Subtypes include: planar trusses, 3 or 4 chord trusses, tubular, and rigid frame structures.

CANTILEVER - consisting of one or more horizontal members supported at only one end.

CENTER MOUNTED - consisting of one or more horizontal members supported at the center.

POLE-MOUNTED –Used exclusively for VMS, with a mounting plate on top of a single column support.

STRUCTURE MOUNTED - a sign attachment permanently mounted to the fascia beam and/or parapet to be visible to traffic beneath the bridge as shown on attached sketch. These signs are typically inspected during the bridge's NBIS inspections. Particular attention must be given to sign attachments made using powder actuated methods and materials.

Cantilever or Overhead Sign Structures that are mounted on bridge piers, barriers, brackets, etc., are to be classified as such and not as Structure Mounted. They should be inventoried separately from bridges.

### 2.11.2 Types of Sign Structure Inspections

All Sign Structure inspections shall be coded in BMS2 Item 7A03 as S-Sign Structure. Item IS01 in BMS2 allows for the coding of one of five sign structure inspection types, all of which include close visual and hands-on examination of the sign structures. A brief description of each of these is given below:

INITIAL INVENTORY (BMS2 Item IS01 = A) - This type of inspection provides for the collection of a sign structure's inventory data for entry into the Bridge Management System 2 (BMS2). All items included on the BMS3 Signs & Lights page must be completed. This work includes an In-depth inspection as described below.

IN-DEPTH (BMS2 Item IS01 = B) - A close visual and hands-on examination of each component, member, fastener, and weld on the structure and/or non-destructive field tests and/or material tests are performed to fully ascertain the existence of or the extent of any deficiency. Lane closures are anticipated to permit access to all portions of structure.

IN-DEPTH (Alternate Lanes Closed) (BMS2 Item IS01 = C) – This type of inspection involves a close visual and hands-on examination of column bases, end supports, or selected portions of horizontal members. Areas of horizontal members to have close hands-on inspection and/or non-destructive field tests and/or material tests performed to fully ascertain the existence of or the extent of any deficiency, are selected to provide overall safety while minimizing traffic disruption. Existing inventory data is to be updated.

ROUTINE (BMS2 Item IS01 = D) - A close visual and hands-on examination of all portions of the sign structure. Lane closures are anticipated to permit access to all portions of structure. Ladders can be used to access end supports away from traffic. Existing inventory data is to be updated.

SPECIAL INSPECTIONS (BMS2 Item IS01 = E) - This type of inspection will be performed to provide in-depth assessment of special conditions when significant structural deficiencies, severe section loss, collision damage, or corrosion have been noted. These inspections will be performed as directed by the District Bridge Engineer.

Inspection types Routine, In-depth and Special Inspections are performed subsequent to the initial inventory inspection and involve only a cursory review of the inventory data to verify correctness. These four different levels of effort can be used to evaluate the sign structure based on its condition and inspection history.

### 2.11.3 Inspection Intervals and Typical Cycles

The interval of inspection and level of inspection intensity for Overhead and Cantilever sign structures are influenced by structure material, structure type, condition and age. Table IP 2.11.3-1 establishes the inspection intervals for the various sign structures. Structure-mounted sign structures are to be inspected along with the other bridge components as part of the biennial NBIS safety inspections. Table IP 2.11.3-2 lists the Typical Cycles for conduction of Safety inspection for different sign structure types and varying conditions.

| Table IP 2.11.3-1 – Sign Structure Routine Inspection Interval | |
|---|---|
| **STRUCTURE TYPE** | **INSPECTION INTERVAL** |
| ALUMINUM OVERHEADS | 2 YEARS |
| GALVANIZED STEEL OVERHEADS | |
| COND. $\geq$ 5 | 6 YEARS |
| COND. $\leq$ 4 or BUILT PRIOR TO 1995 | 3 YEARS |
| MOUNTED ON BRIDGES | 2 YEARS |
| GALVANIZED STEEL CANTILEVERS | |
| GROUND MOUNTED | 6YEARS |
| MOUNTED ON BRIDGES | 2 YEARS |
| STRUCTURE MOUNTED | *2 YEARS |

NOTE: IN-DEPTH INSPECTION SCHEDULED ON AN AS NEEDED BASIS AND INCLUDES ADVANCED DETECTION TECHNIQUES

*STRUCTURE MOUNTED SIGNS ARE INCLUDED AS PART OF A BRIDGE INSPECTION

| Table IP 2.11.3-2 – Sign Structure Inspection – Typical Cycles | | | | | | |
|---|---|---|---|---|---|---|
| **STRUCTURE TYPE** | **YEAR** | | | | | |
| | **1** | **2** | **3** | **4** | **5** | **6** |
| ALUMINUM OVERHEADS | X | — | X | — | X | — |
| GALVANIZED STEEL OVERHEADS | | | | | | |
| COND. $\geq$ 5 | X | — | — | — | — | — |
| COND. $\leq$ 4 or BUILT PRIOR TO 1995 | X | — | — | X | — | — |
| MOUNTED ON BRIDGES | X | — | X | — | X | — |
| GALVANIZED STEEL CANTILEVERS | | | | | | |
| GROUND MOUNTED | X | — | — | — | — | — |
| MOUNTED ON BRIDGES | X | — | X | — | X | — |
| STRUCTURE MOUNTED | X | — | X | — | X | — |

### 2.11.4 Field Inspection Procedures

The General Scope of Work for the Safety Inspection of Sign Structures is in Appendix IP 02-D. Many of the techniques from bridge inspection are also applicable for sign structures. Use appropriate portions of these guidelines and bridge inspection procedures to inspect Structure-Mounted signs.

## 2.12 ADJACENT NON-COMPOSITE PRESTRESSED CONCRETE BOX BEAMS

The December 2005 collapse of a fascia beam on the bridge carrying State Route 1014 over Interstate 70 in Washington County resulted in a review of the procedures and practices used in the safety inspection and load rating analysis for adjacent non-composite prestressed concrete box beam bridges.

The bridge had four simple spans with a bituminous wearing surface (without a waterproofing membrane). There had been considerable damage to the failed fascia beam and interior beams due to overheight vehicle collisions. Through the years, the loss of additional prestressing strands occurred due to continuing corrosion.

Some of this strand loss was not detectable using routine visual inspection methods. The result was that the fascia beam collapsed under its own weight.

As a result of this review and studies of the failed beam conducted by the University of Pittsburgh and Lehigh University, the following sections of this Manual have been modified or added:

- Field Inspection Guidelines: For guidelines on inspection procedures and documentation of findings, see IE 4.3.5.6.3.1I.
- Load Rating: For guidelines on general requirements, see IE 6.1.5.3I. For distribution of barrier dead load, see IE 6B.6.1. For distribution of live loads, see IE 6B.6.3. Load ratings by Engineering Judgment per Appendix IP 03-B is no longer permitted for adjacent non-composite prestressed concrete box beams; see Appendix IP 03-B Applicability of Guidelines for more information.

## 2.13 STRUCTURE MAINTENANCE NEEDS

Maintenance needs for bridges and other structure types are to be identified and recorded during the safety inspection process. The inspectors are to recommend maintenance activities and the priority of their need based upon the conditions in the field. The maintenance needs and priorities recommended from the field must be reviewed to account for information that may not have been available in the field, including:

- Anticipated deterioration rate and its expected impact on structural safety.
- Implications for changes in load rating capacity and/or structural stability.
- The effect of mitigation efforts, including planned or current traffic restrictions, to ensure public safety.
- The level of confidence in condition information, as modified by interval of inspection, potential for hidden deterioration, structural redundancy or complexity, etc.

The condition rating and maintenance priority descriptions are provided in the BMS2 Coding Manual, Publication 100A.

As indicated in the TOMIE, there are three main categories for tunnel maintenance items. Critical Findings would be labeled as a Priority 0, Priority Repairs would be Priority 1, 2 or 3 and Routine Repairs would be Priority 4 or 5 maintenance items in BMS2. See TOMIE Section 4.12.2.4 for additional information.

### 2.13.1 Proposed Maintenance Needs in BMS2

The proposed maintenance needs identified by safety inspections are to be recorded in BMS2 as Flexible Actions using the list of standard maintenance activities in the BMS2 Coding Manual, Publication 100A. These maintenance items are subdivided into the following four categories for maintenance planning purposes and Department performance measures:

| | |
|---|---|
| **Bridge** | Activities that generally have either high or medium value payoff benefit for maintaining bridge structural component repair and/or replacement. |
| **Tunnel** | Activities that generally have either high or medium value payoff benefit for maintaining tunnel component repair and/or replacement. |
| **Other Structural** | Activities that generally have low value payoff benefit with respect to maintaining bridges or pertain to another structure type (i.e. sign structures). |
| **Bridge Cleaning** | Bridge and structure cleaning activities. |

The bridge maintenance category and value payoff benefit was determined by its relative impact on the life cycle costs of maintaining bridges. Accordingly, these categories and payoff values do not correspond directly to assignment of maintenance priorities, especially for Priority 0 or 1.

### 2.13.2 Critical and High Priority Maintenance Items

Critical (Priority 0) or high (Priority 1) priorities (BMS2 Item IM05) for maintenance activities are defined as having deficiencies that threaten either the structural integrity of the bridge (or other structures) or public safety. This limits the application of Priority 0 or 1 to those activities to correct such deficiencies. Damaged or missing vertical clearance or load limit signs are examples where there may be no immediate structure safety problem, but where public safety is compromised, and immediate action is required. It follows that maintenance activities such as painting and cleaning may have a high value payoff benefit for life cycle costs but should not warrant a Priority of 0 or 1. There may be situations where activities in Other Structural category of maintenance activities may warrant Priority 0 or 1.

Bridges with a superstructure, substructure, deck, culvert or SCBI condition rating of a 2 or less shall also have a critical or high priority maintenance item that will target the cause of the low condition rating. As indicated in IP 2.14 and IP 6.2, these bridges require a Plan of Action (POA) and review by the District Bridge Engineer before a bridge inspection can be moved into "Accepted" status.

The BMS2 Coding Manual, Publication 100A further details the IM05 coding instructions and provides further examples.

In general, the conditions that create the recommendation for Priority 0 and Priority 1 maintenance items would warrant the development of a Bridge Problem Report (BPR). The District should notify the Bridge Inspection Section of the problem immediately and the determination of the need for a BPR will be made.

Because of the threat to structural and public safety they pose, the following additional actions are required for Department bridges (or structures) with maintenance activities that warrant a critical or high priority (IM05 = 0 or 1 respectively):

- **Approval by District Bridge Engineer –** The inspection and maintenance recommendations must be immediately reviewed by the District Bridge Engineer to determine the appropriate mitigation and restoration for the deficient condition.
- **Approval by County Maintenance Manager –** After approval by the District Bridge Engineer, the County Maintenance Manager shall review and accept the POA maintenance activities and ensure the activities are completed.
- **Plan of Action for resolving the critical deficiencies** – See IP 2.14

**For non-Department bridges** (including those owned by municipalities, counties and other agencies), critical or high priority maintenance needs pose the same possible threats to public safety. In the Department's overarching role and responsibilities for the safety of public highway bridges, the District must take additional steps to see that the bridge owner has had the public safety issues fully explained to them along with the need to take appropriate action to mitigate or correct them.

- The standard scopes of work for bridge safety inspection agreements require the inspector to notify the owner of such critical deficiencies and also conduct a meeting to discuss all critical structural and safety related deficiencies. As part of the meeting to discuss these critical deficiencies, the consultant shall prepare a Plan of Action (POA) in coordination with the local owner (see Appendix 01-F, G and H). The District (also a recipient of the critical deficiency notice) is to contact the owner by telephone call, personal visit, or correspondence (e-mail) to determine if the owner's plan for repair is timely and sufficient. To properly convey the significant public safety issues, this District contact person should be the District Bridge Engineer or other Professional Engineer. The District Municipal Services staff may also be helpful during this contact and by attendance at the meeting to discuss the critical deficiency. The meeting and POA development shall be within three (3) calendar days from the date the critical structural or safety deficiency is discovered. For high priority structure deficiencies, the meeting and POA must be conducted within seven (7) days.
- While critical deficiencies can exist for bridge-related signs as indicated above, they do not warrant a meeting or POA since the necessary corrective action regarding signs is sufficiently clear. However, the requirements for notification described above remain in effect.
- If the proposed action by the owner is not deemed sufficient, additional District actions may be necessary to ensure public safety, up to and including closing the bridge. The District is to immediately contact the Assistant Chief Bridge Engineer - Inspection if an owner is not taking timely action and before restricting a bridge without the owner's consent as indicated in IP 1.7.2.4 and IP 4.7.2.
- The non-Department bridge owners are required to follow the full POA process as described herein. The POA process in BMS2 is a good way for them to document their actions for critical bridge deficiencies.

### 2.13.3 Critical Findings

Critical findings for bridges and tunnels are structural or safety related deficiencies that require immediate action and must be reported in accordance with the NBIS and NTIS. At a minimum, critical findings are defined by 23 CFR 650.313(q)(1)(i) as the following:

(A) Full or partial closure of any bridge; or
(B) An NSTM to be rated in serious or worse condition, as defined in the SNBI for Item B.C.14 (NSTM Inspection Condition) coded three (3) or less; or
(C) A deck, superstructure, substructure, or culvert component to be rated in critical or worse condition, as defined in the Coding Manual (or SNBI) for the Deck, Superstructure, Substructure, or Culvert Condition Rating items coded two (2) or less; or
(D) The channel condition or scour condition to be rated in critical or worse condition as defined in the Coding Manual (or SNBI) in the Channel Condition Rating or Scour Condition Rating items, coded critical (2) or less; or
(E) Immediate load restriction or posting, or immediate repair to a bridge, including shoring, in order to remain open.

Critical findings and mitigation measures shall be well documented through field notes and photographs. All documentation, including detailed descriptions and photographs, shall be made available to FHWA during the reporting process. Critical findings typically require the following action be taken in a timely manner:

- Closure or restriction until the severe defect is removed or repaired, if the defect may impact users or user safety.
- Repair the structural member or address the functional or safety issue.

It is the responsibility of the team leader on site to identify critical findings and notify the owner as soon as possible. Tunnel owners shall follow the written procedures established in the tunnel inspection agreement when a critical finding is identified to ensure FHWA is notified within 24 hours of the initial finding. When a critical finding is identified on an NBIS length State, Local, or Other State Agency owned bridge, the Assistant Chief Bridge Engineer – Inspection must be notified by the owner within 24 hours of the initial finding. The Assistant Chief Bridge Engineer – Inspection shall notify FHWA within 24 hours of the initial finding for all critical findings for NBIS length bridges on the National Highway System (NHS) which result in a full or partial closure of a bridge and/or an NSTM with a condition rating of three (3) or less. Depending on the type of finding, others may need to be notified as well such as bridge or tunnel maintenance personnel.

Once the critical finding has been identified and reported, future steps shall be identified and recorded to monitor or mitigate the concern. For example, if the strength or serviceability of the bridge or tunnel could be at risk, a structural review or systems analysis may be required. A follow-up inspection may also be required. For bridges, provide monthly, or as requested, status reports to FHWA for each critical finding until resolved.

The PennDOT Program Manager will submit an annual report to FHWA detailing all the bridge and tunnel critical findings which were reported within the prior calendar year as well as mitigation measures taken.

#### 2.13.4 Uncoated Weathering Steel Bridge Maintenance

##### 2.13.4.1 GENERAL

Appendix IP 02-I, *Uncoated Weathering Steel - Bridge Safety Inspection and Maintenance Manual,* provides bridge maintenance procedures and requirements for uncoated weathering steel bridges. These procedures and requirements shall be used on Department bridges and are recommended for use on locally-owned uncoated weathering steel bridges.

### 2.14 PLAN OF ACTION FOR CRITICAL FINDINGS AND HIGH PRIORITY MAINTENANCE ITEMS

Because of the risk to public safety, a Plan of Action (POA) is required for all recommended maintenance activities with a Priority 0 or 1 for both Department and non-Department bridges (or structures). All Priority 0 or 1 maintenance findings must be reported in writing within 24 hours of discovery during each inspection to the District

Bridge Engineer and Assistant District Bridge Engineer – Inspection for Department owned bridges (or structures) and to the owner for non-Department owned bridges (or structures). The POA must identify the action(s) to be taken to repair and/or mitigate the deficiency that warranted the critical or high priority recommendation. A POA is not intended to be a stand-alone document, but rather a compilation of notes documented in the Maintenance Section of BMS. See IP 2.14.4 for additional information.

Bridges with a Scour Critical Bridge Indicator (SCBI) having a rating equal to a two (2) must have a similar review and evaluation. An April 27, 2001, memorandum from the Federal Highway Administration (FHWA) states the substructure condition rating should be consistent with the one given to the SCBI whenever a rating factor of 2 or below is determined for the SCBI rating. Due to the critical condition of the bridge, a POA is required to ensure public safety and must identify what action(s) will be taken to improve the condition of the structure from a condition rating of a 2. A critical finding and POA is also required when an element is coded in Condition State 4 for a tunnel and possesses an immediate structural or safety related risk.

The processes involved in determining the recommended action and its priority in the safety inspection program and the development and implementation of the POA itself are highly interactive. High levels of communication and cooperation between all parties are necessary to make the required corrective actions occur in a timely manner.

In some instances, the risk to public safety could be imminent and require the bridge inspectors to take immediate actions to close a structure from vehicular and pedestrian traffic. These instances require appropriate action to ensure public safety without putting the safety of the traveling public or the inspectors at risk. Detailed procedures to follow in these instances are provided in Appendix IP 02-F.

### 2.14.1 Timeframe for POAs

Priority 0 activities are to be resolved or mitigated within 7 days of identification.

- For Priority 0 items, physical action must be taken to resolve the critical deficiency. By definition, this work cannot be deferred. By definition, monitoring alone for Priority 0 items will not suffice.
- The immediate actions taken may not resolve the deficiency but must restore or mitigate the safety problem to an acceptable level, even if level of service must be reduced. Such an example may be to close the bridge immediately until repairs can commence.
- Additional work at a later time may be needed to restore bridge to full service and safety. This additional work may be needed at a Priority 1 level.

Priority 1 activities are to be resolved or mitigated within 6 months.

- Priority 1 requires physical work to correct the deficiency.
- Monitoring alone or in conjunction with repairs may suffice if conditions will not degrade. Monitoring should be quantitative by including measurements of structure, e.g., deflection, crack width, scour depth, etc., and not just description of visual observation. Documentation, including sketches and charts showing changes over time are highly recommended. Documentation of monitoring must be added to BMS2.
- A Special Inspection must be scheduled at a maximum 6-month interval when a Priority 1 maintenance item is identified.
    - Scheduling of the Special Inspection will enable tracking of necessary follow-up inspections in case the maintenance is not completed within the 6-month time frame.
    - If the maintenance is completed within the 6-month timeframe, there are three options to document the completion:
        - If the Routine/Special inspection has not yet been accepted when the repair is completed, then the maintenance item completion can be documented during the current inspection. Photo documentation may suffice (in lieu of a site visit) for straightforward repairs including placement of posting signs. The maintenance item shall be marked completed before the inspection is accepted in BMS2. OR
        - Perform a Problem Area inspection to document the repair was adequately completed. Photo documentation may suffice (in lieu of a site visit) for straight forward repairs including placement of posting signs; however, a Problem Area

inspection shall still be opened to document the completion of the maintenance item. Mark the maintenance item complete in BMS2 and remove the scheduled 6-month Special Inspection from the BMS2 system. OR
    - Complete the scheduled 6-month Special Inspection. Mark the maintenance item complete in BMS2. A subsequent 6-month Special Inspection will no longer be required to be scheduled for the completed maintenance item.
- If, during the next inspection, repairs are not completed, a new POA shall be issued with the District Bridge Engineer's approval.
- For a Priority 1, physical work to correct the deficiencies may be deferred only if other work (e.g., rehabilitation project) is scheduled to be completed in the near future and if the condition of the structure will not further degrade and compromise safety before that scheduled work is done. In this context, the timeframe for "near future" generally should not extend beyond 2 years unless justification for this exception is provided by the District Bridge Engineer.
- Deferred Priority 1 maintenance items must still be monitored a maximum interval of 6 months by scheduling a 6-month interval Special Inspection. Monitoring of deferred items at a 6-month interval will ensure the condition has not degraded.
- For deferred actions, the POA must provide full justification for the above points.

Bridges in Critical Condition

- Because it is expected that bridges in critical condition will also have critical or high priority maintenance items (i.e., Priority 0 and 1), determine whether these maintenance items are properly identified and addressed with a POA in accordance with IP 2.13 and IP 6.2.4.
- Evaluate the current load rating for the structure and determine if the rating still applies or if a rerating is needed.
- Take appropriate action to post or reduce the posting of a structure, execute repairs in accordance with the POA, or close the bridge when conditions warrant such action.
- For bridges in critical condition with a Scour Critical Bridge Indicator (SCBI) rating of two (2) or less, review documentation of scour measurements from probing or diving to ensure the scour condition is stable and not worsening.
- Update Inspection Maintenance (IM) screen including POA data fields in BMS2 and provide a schedule in IM15b if a high priority maintenance item is deferred.

Tunnels with Critical Findings

- Critical findings shall be handled under the same timeline and rules of a Priority 0 above.
- Critical findings must be reported to FHWA within 24 hours.
- For structural elements, a structural review to determine the effect on strength or serviceability of the tunnel must be completed.
- For non-structural elements, an evaluation to determine the effect on serviceability of the element or tunnel must be completed.
- Regardless of the findings of the review of structural and/or non-structural elements, a POA to mitigate the critical finding must be submitted to the owner, PennDOT and FHWA.

Development and acceptance of a Priority 0, Critical Condition POA or tunnel critical finding POA should be complete within 3 days. Development and acceptance of a Priority 1 POA should be complete within 7 days.

The timeframe for a POA starts on the day of the inspection where deficiency is discovered (BMS2 Field IM06). Generally, the need for critical or high priority maintenance items is determined during a safety inspection. However, if a critical deficiency is found between inspections, an inspection to fully ascertain the condition must be done. This need not be a regular NBIS/NTIS inspection if a special inspection of the problem area only will suffice.

It may be necessary to take immediate actions, permanent or temporary in nature, to safeguard public safety (e.g. temporary shoring, bridge closing) before the POA is fully developed.

Exceptions to the timeline for POA documentation will be considered only for complex problems that take more time to reach a final solution. The District must request a POA documentation time extension via e-mail to the

Assistant Chief Bridge Engineer - Inspection before the end of day 3. Such extensions to the POA documentation do not extend to implementation of interim or permanent action(s) recommended to secure safety.

### 2.14.2 Responsibilities for Plan of Action

The responsibilities for the various portions of a POA can be generally described as listed below. It is important to note that the Districts may reassign the POA responsibilities outside the safety inspection program activities to meet the POA requirements as best fits their organization. However, decisions concerning public safety which require the expertise of a professional engineer cannot be reassigned to individuals not duly qualified.

<u>District Bridge Engineer</u>

- Determination of Critical or High Priority Maintenance Recommendations
- Development of POA and concurrence with actions.
- Development of repair plans, as needed
- Input and maintenance of BMS2 data
- Status Reports to Bridge Inspection Section
- Safety inspection of completed repairs

<u>District Bridge Maintenance Coordinator</u>

- SAP Notifications and Work Orders
- Coordination of Department Force efforts
- Other duties as assigned

<u>County Maintenance Manager</u>

- Review and acceptance of POA
- Initial emergency response and traffic control activities
- Management of Department forces
- Procurement of materials and equipment
- Schedule and provide resources to meet the POA needs.
- Status reports to District Bridge Engineer
- SAP data entry

<u>Other District Responsibilities</u>

- Procurement of Contractor Services
- Management and QC of construction activities
- Public information
- Funding

<u>Bridge Inspection Section and Bureau of Bridge</u>

- Status reports to Executive staff
- Technical assistance
- Summarize the critical finding and response plan for the annual FHWA report

<u>Tunnel Owner</u>

- Report critical findings to PennDOT in a timely fashion such that FHWA will receive notification within 24 hours

### 2.14.3 The POA Development Process

A general flowchart of the processes involved in the development of a POA is shown in Figure IP 2.14.3-1. The description of the POA flowchart steps is provided in Figure IP 2.14.3-2 (3 pages).

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**Flow Chart for Plan of Action**

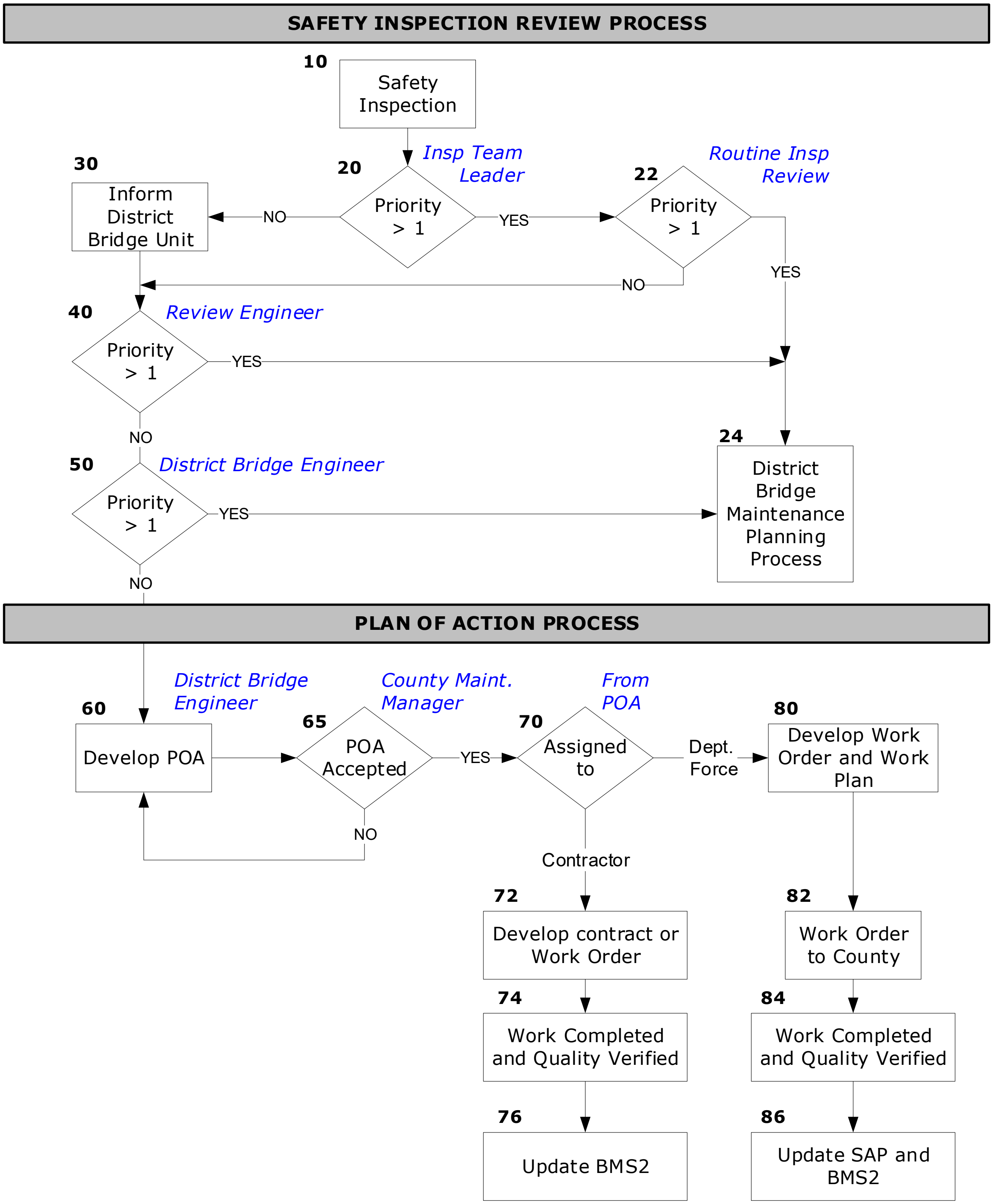


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Figure IP 2.14.3-1

## Plan of Action Flow Chart - Steps

## SAFETY INSPECTION REVIEW PROCESS

**STEP 10 Safety Inspection**
- For inspections performed by consultant or Department forces.

**STEP 20 Field Review of Recommended Maintenance**
- Review performed by Team Leader
- Team Leader is to determine if any Maintenance item is recommended at a Priority of 0 or 1.
- If any recommended bridge maintenance items have a Priority of 0 or 1, the Team Leader is to contact District Bridge Unit immediately.
- If no Priority 0 or 1 items are identified, inspection proceeds to routine review process.

**STEP 22 Routine Inspection Review**
- Review performed by Reviewer at District Bridge Unit
- If the review results in any maintenance items being elevated to Priority 0 or 1, the inspection must go to Step 40.
- If no Priority 0 or 1 items are identified, maintenance recommendations proceed to routine District bridge maintenance planning process.

**STEP 24 District Bridge Maintenance Planning Process**
- This is the normal process used by the District to plan maintenance.

**STEP 30 Team Leader Informs District Bridge Unit**
- The Team Leader makes all inspection information, including field report and photos, available to the District Bridge Unit.

**STEP 40 Review Engineer Review of Inspection**
- The inspection and recommended maintenance is reviewed in District Bridge Unit.
- If Critical or High priority items are recommended by the inspectors, this review must be done by a registered PE.
- If the review results in no maintenance items receiving Priority 0 or 1, the Review Engineer may accept the inspection in BMS2 and all recommended maintenance proceeds to routine planning process.
- If critical or high priority items remain, the inspection must be reviewed by the District Bridge Engineer.

**STEP 50 Bridge Engineer Review of Inspection**
- The inspection report and recommended high priority maintenance items are reviewed by the District Bridge Engineer.
- If this review results in no maintenance items receiving Priority 0 or 1, the Bridge Engineer accepts the inspection in BMS2 and all recommended maintenance proceeds to routine planning process (Step 24).
- If critical or high priority items remain, the District Bridge Engineer is to develop a POA to resolve and/or mitigate the deficiencies (Step 60).
- If immediate emergency action is required for public safety, the County Maintenance Manager is to be informed immediately and appropriate work initiated. Do not wait for formal POA to be adopted for these immediate actions to be started.

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Figure IP 2.14.3-2

## PLAN OF ACTION PROCESS

**STEP 60 Develop the POA for Critical and High Priority Items**

- **For Priority 0 items**, physical work is needed to resolve the critical deficiency. By definition, this work cannot be deferred.
    - Immediate actions may include closing or posting the bridge, restricting traffic from the damaged areas, temporary shoring, etc. These actions may not completely resolve the deficiency but must restore the structure safety to an acceptable level.
    - Additional work at a later time may be required to more fully restore safety and level of service. This later work and schedule should be included in POA.
    - If initial immediate actions bring the bridge safety to an acceptable level, the priority for remaining permanent repair work may be downgraded to 1.
    - Priority 0 items are to be resolved within 7 days of inspection.
- **For Priority 1 items**, physical work to correct the deficiencies may be deferred only if the needed repair, rehabilitation or replacement is scheduled such that public safety is not compromised in the interim. Deferred items must be monitored as indicated in Pub 238, IP 2.14.1.
    - Similar to the description in Priority 0 items, the POA for repairs may involve immediate actions followed by additional work.
    - Deferred repairs must be justified in the POA and recorded in BMS2.
    - Priority 1 work items are to be resolved within 6 months.
    - A Special Inspection must be scheduled at a maximum 6-month interval when a Priority 1 maintenance item is identified.
- Where emergency traffic restrictions are used, use the procedures outlined in Appendix IP-02C.
- Monitoring is required to ensure ongoing safety until work is complete. Monitoring results must be documented in BMS2. Monitoring alone is not sufficient for Priority 0 deficiencies.
- The POA is to include:
    - Scope of physical and/or design work
    - Estimated Costs
    - How work is to be performed (Contractor or Department forces)
    - Timeframe to completion

**STEP 65 Acceptance of POA by County Maintenance Manager (CMM)**

- County Maintenance Manager reviews draft POA for acceptance.
- In extreme situations, the POA may not be fully developed before initial actions have to be taken.
- If acceptable, the CMM is to inform District Bridge Engineer immediately.
- If not acceptable, CMM is to offer suggestions for changes.
- If the DBE and CMM cannot reach a timely agreement on the POA, the ADE-Maintenance and ADE-Design are to be consulted. Because of the public safety issues, the final decision on actions and recommended plans must be approved by a professional engineer.

**STEP 70 Assignment of Activities**

- District to assign activities to contractor and/or Department Force as outlined in POA
- The District Bridge Unit is to update BMS2 accordingly.
- For work assigned to Department Force, go to Step 80.
- For work assigned to contractor, go to Step 72

**STEP 72 Develop contract or Work Order**

- Responsibility for this activity to be assigned by the District.

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Figure IP 2.14.3-2

## PLAN OF ACTION PROCESS (cont.)

**STEP 74 Contractor Work Completed and Quality Verified**

- QA/QC construction duties to be assigned by District. The District Bridge Engineer must be informed when the work has been completed by contractor or department forces.
- Special inspection of repairs to be made by District or consultant.

**STEP 76 Update BMS2 data**

- Responsibility for this activity to be assigned by the District.

**STEP 80 Develop Work Order and Work Plan**

- Bridge Maintenance Coordinator to work with Bridge Unit to create Work Order for the Department Forces

**STEP 82 County notified of new work through SAP**

- Notification to SAP to be generated in BMS2

**STEP 84 Department Force Work Completed and Quality verified**

- QA/QC construction duties to be assigned by District
- Special inspection of repairs to be made by District or consultant to verify satisfactory completion of the critical or high priority work and to update BMS2.

**STEP 86 Update BMS2 Data**

- Responsibility for this activity to be assigned by the District.

**Communication and tracking**

- During a bridge emergency, good communication is vital to teamwork and ultimate success of mission.
    - Use e-mail and other forms of electronic communication to expedite transfer of critical information between team members.
    - Communication of work status to Assistant Chief Bridge Engineer - Inspection is vital in order that technical assistance and oversight may be made expeditiously.
- The District is responsible to track the progress of POA in BMS2.
- The District Bridge Engineer will be responsible to provide timely status updates to the Assistant Chief Bridge Engineer - Inspection and Chief Bridge Engineer.
- The Assistant Chief Bridge Engineer - Inspection or the Chief Bridge Engineer is responsible for required reports to Executive Staff.

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Figure IP 2.14.3-2

### 2.14.4 POA Data in BMS2

The Maintenance page of BMS3, as well as the Proposed Maintenance Screen in BMS2 have data fields to record the limited data items to document and track POAs for Critical and High Priority maintenance activities as well as critical findings for tunnels. Guidance for the data items is provided in the BMS2 Coding Manual, Publication 100A. The POA must contain sufficient detail to outline responsible party, estimated start and completion dates of the necessary repair to address the item. The Districts shall document their bridge POAs by updating BMS fields in the IM screen. The POA information contained in the IM section shall serve as PennDOT's official POA for the maintenance item.

Bridges with critical and/or high priority maintenance items or tunnels with critical findings will require the maintenance items to have complete information for the POA in BMS2 before the inspection can be accepted. These fields include IM14b – POA Date, IM14c – Mitigation Date (if mitigated from a P0 to a P1), IM15a – Notes, IM15c – Bridge Approver, and IM15d – Maintenance Approver. The specific POA information and details shall be entered in BMS field IM15a. POA information and data contained in IM15a shall be in the form of "#" notes in the field and clearly present how the item was initially planned to be addressed, and subsequent actions taken to repair the item through completion. Refer to Publication 100A for more information regarding the coding of BMS Item IM15a.

### 2.14.5 Status Reports for POA Activities

The District is responsible to ensure that the BMS2 data items relating to the POA and status of corrective activities are maintained and up-to-date. The Bridge Inspection Section will monitor this data for compliance with Department policies.

Upon completion/mitigation of Priority 0 items, the District Bridge Engineer is to notify the Assistant Chief Bridge Engineer - Inspection via e-mail that these critical tasks are complete. This notification will also serve as documentation for completion of needed activities identified in the Bridge Problem Report. Additionally, IM14a – Completion Date for the maintenance item should be completed in BMS2 or BMS3.

For bridges or problems of special interest, the Assistant Chief Bridge Engineer - Inspection may request more frequent status reports of work activities/completion.

## 2.15 RETAINING WALLS

The purpose of the retaining wall safety inspection program is to verify each retaining wall's inventory data, to determine its physical condition and maintenance needs, and to record the same in the Department's BMS2. The inspection typically consists of an examination and recording of signs of damage, deterioration, movement, and if in water, evidence of scour.

To achieve a minimum 100 year service life for retaining walls, the walls should be inspected/monitored on a regular basis as described in IP 2.15.3 to identify and address specialized issues and receive routine preventative maintenance, preservation, and rehabilitation. That is, both maintenance and inspection are vital to ensuring the longevity and performance of retaining walls.

The inspection of retaining walls is required for walls that have their own S-number. Wingwalls of a bridge are included with the inspection of the bridge.

### 2.15.1 Types of Retaining Walls

Retaining walls typically have a material type of steel, concrete, timber, masonry or stone. Many structural configurations of retaining walls exist such as cantilever, gabion, Mechanically Stabilized Earth (MSE), tie-back, and concrete modular walls, etc.

#### 2.15.1.1 MECHANICALLY STABILIZED EARTH WALLS

Mechanically Stabilized Earth (MSE) retaining wall systems have three major components:

- Soil reinforcement (mesh or strip) – The soil reinforcement is described by the type of material used, the soil reinforcement geometry, and the connection method to the precast facing panels.
- Backfill - The backfill used within the reinforced zone is granular to meet stress transfer, durability, and drainage requirements.
- Precast facing elements - Facing elements are provided to retain fill material at the face. Typical facing elements include precast concrete panels with or without architectural treatments, extruded metal sections or timber.

In addition to the three major components, several other components are typical to MSE retaining walls:

- Leveling pads - Note: Leveling pads are non-structural footings used at the base of the wall as an aid in construction.
- Barriers - Barriers supported by moment slabs are common on top of MSE retaining walls to protect traffic traveling on a parallel roadway.
- Expansion joints - Vertical slip joints are required for expansion for long lengths of walls.

Neither reinforcement nor backfill will be able to be inspected; therefore, a close visual inspection of the facing panels and drainage facilities is required to provide information on all three of the major components. This includes visual inspection of the roadway surface (i.e., pavement) above the MSE wall for tension cracking. Inspection of the leveling pads, if visible, can provide information on scour, erosion or settlement. Inspection of the barriers can also provide important information regarding movement of the MSE wall.

### 2.15.2 Types of Retaining Wall Inspections

All retaining wall inspections shall be coded in BMS2 Item 7A03 as T-Retaining Wall. The specific type of inspection for the retaining wall should be indicated in BMS2 in the Inspection Applet Item IW01. The following are four inspection types, all of which include close visual and hands-on examination of retaining walls. A brief description of each of these is given below:

<u>INITIAL INVENTORY</u> (BMS2 Item IW01 = F) – This type of inspection provides for the collection of a retaining wall's inventory data for entry into BMS2. For mechanically stabilized earth walls with a length in excess of 100 feet and with at least 20 feet of exposed height, a three-dimensional survey* must also be completed. Refer to Publication 100A, BMS2 Coding Manual, Section 2.4 for creating new structures. BMS2 screen VW must be completed.

<u>IN-DEPTH</u> (BMS Item IW01 = D) – A close visual and hands-on examination of retaining walls and their drainage systems. Use of down-hole cameras or visual inspection of larger pipes is required for the drainage system. All items included on the BMS3 Maintenance, Notes, and Walls pages must be completed. For mechanically stabilized earth walls with a length in excess of 100 feet and with at least 20 feet of exposed height, a three-dimensional survey* must also be completed. Existing inventory data is to be updated.

<u>ROUTINE</u> (BMS2 Item IW01 = R) – A close visual and hands-on examination of retaining walls and their drainage systems <u>without traffic control.</u> Those portions which cannot be accessed safely from beyond the edge of pavement are viewed using binoculars and/or a digital camera. All items included on the BMS3 Maintenance, Notes, and Walls pages must be completed. Existing inventory data is to be updated.

<u>SPECIAL INSPECTION</u> (BMS2 Item IW01 = P) – A close visual and hands-on examination of retaining walls and their drainage systems after a significant occurrence such as a vehicular collision or extreme weather event where heavy prolonged rains or flooding may have occurred. All items included on the BMS3 Maintenance, Notes, and Walls pages must be completed. Existing inventory data is to be updated.

* The three-dimensional survey should be completed in accordance with the guidelines in the Surveying and Mapping Manual, Publication 122M, Part A Chapter 3 and Chapter 6.7. The Photogrammetry and Survey Section and Multiple Districts have LiDAR scanners with three-dimensional survey capability available for use in the Engineering Districts.

### 2.15.3 Inspection Intervals and Typical Cycles

The inspection interval of retaining walls is listed below in Table IP 2.15.3-1. Wingwalls of a bridge are to be inspected with the bridge on the interval defined in Table IP 2.3.2.4-1 or Table IP 2.3.2.4-2.

**Table IP 2.15.3-1 – Retaining Wall Inspection Intervals**

| Inspection Type (IW01) | Type of Inspection | Interval (years) | Comments |
|---|---|---|---|
| F | Initial | N/A | Inspection required within 6 months of construction completion or before opening road to traffic, whichever is less |
| D | In-Depth | 10* | For Structural Evaluation (BMS2 item 4A09) ≤ 4. For MSE Walls only: Three-dimensional survey to be completed for walls ≥100' long and ≥20' in height |
| | | 15* | For Structural Evaluation (BMS2 item 4A09) > 4. For MSE Walls only: Three-dimensional survey to be completed for walls ≥100' long and ≥20' in height |
| R | Routine | 5 | |
| P | Special | As required | Performed after a significant occurrence such as a vehicular collision, extreme weather, or indication of wall movement. |

* A three-dimensional survey can be requested for MSE Walls at any time when movement of the wall is suspected.

Inspection types, In-Depth, Routine, and Special are performed subsequent to the initial inventory inspection and involve varying levels of effort.

### 2.15.4 Field Inspection Procedures

Many of the techniques from the bridge inspection are also applicable to retaining wall inspections. Establishing a baseline condition for retaining walls is crucial for effective future inspections.

- Inspect exposed wall faces, barriers and moment slabs, footings and joints for: arching, spalling, movement of joints, corrosion of members, locations of entrapped water/improper drainage, evidence of impact, condition of riprap, and/or indications of scour.
- Inspect wall for movement, rotation or settlement.
- Inspect crest of sloping backfill for evidence of soil stress or failure as an indication of settlement or wall movement.
- Inspect drainage facilities in the wall and in the proximity of the wall (above and below the wall) to ensure proper functioning of drainage.

#### 2.15.4.1 MECHANICALLY STABILIZED EARTH WALL FIELD INSPECTION PROCEDURES

The critical factors affecting the long-term performance of MSE walls are: corrosion of the soil reinforcement, improper drainage, improper backfill material and compaction, freezing of entrapped water, and movement of the entire MSE Mass (global stability). Recommendations for inspection and maintenance of MSE retaining walls and their drainage systems are provided in IE 4.3.5.7.2.1I.

In some of the first generation MSE walls, there is a latent defect with the method of connection between soil reinforcement and precast facing panel. The soil reinforcement wires were attached to the panel using collets held in place by steel button heads welded to the end of the soil reinforcement (See Figure IP 2.15.4.1-1). The weld of the button head to the soil reinforcement has failed in some locations. The cold formed button head details were found to develop micro-cracks which contributed to the failure of the button head. The inventory inspection must determine from the shop drawings whether this type of connection was used. State whether or not the button head connection method was used in BMS2 item VW32.

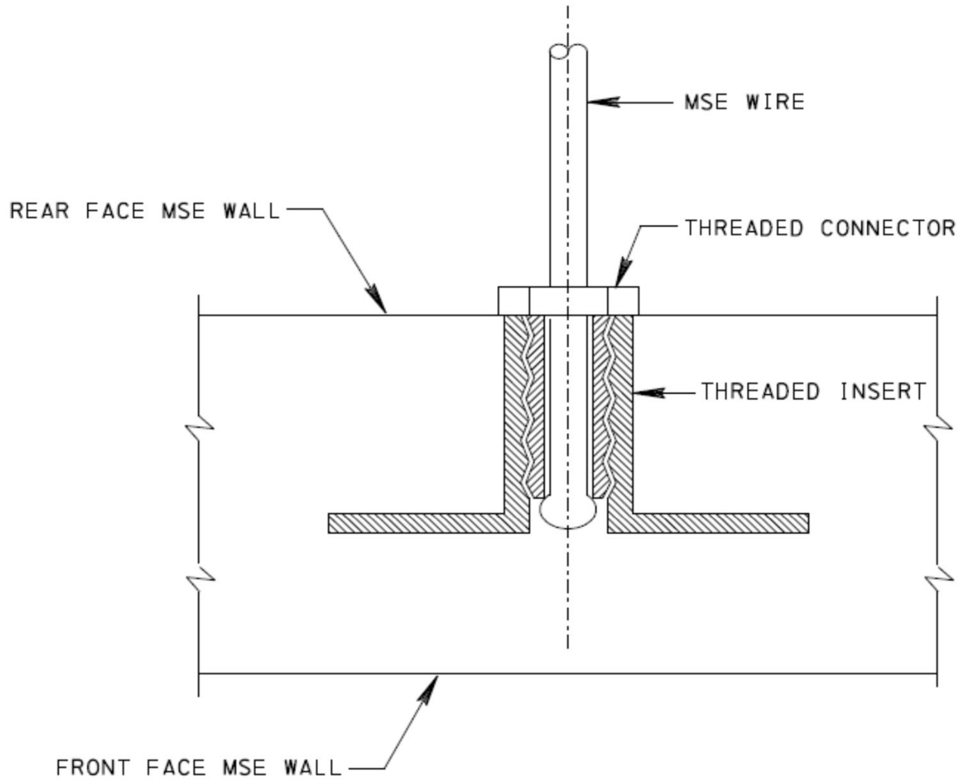


**Figure IP 2.15.4.1-1**

### 2.15.5 Determination of Maintenance Responsibility

Retaining walls which are inventoried in BMS2 and have a regular inspection schedule, shall be maintained by the owner as specified in BMS2. For retaining walls which are found in need of repair, but are not inventoried in BMS2, the following procedures shall be followed to determine who has the responsibility for maintenance.

PennDOT's guidelines have historically reflected that walls on the top side of a highway (which hold abutting property up) are the abutting land owner's responsibility, while walls on the bottom side (which hold the highway up) are PennDOT's responsibility. This should be considered a good first step in determining responsibility but is not totally supported by the law, as determining responsibility for walls along or within highway ROW requires case-by-case analysis. Ultimate responsibility depends on the location of the wall, who constructed it, its function, who has maintained it and other relevant matters.

The initial step in determining responsibility should begin with the District ROW unit to determine if the wall is within PennDOT limits. This may require additional coordination or assistance from the Bureau of Design and Delivery's Right of Way, Utilities and Grade Crossing Division. Once this is known, requests for determination of maintenance responsibilities of retaining walls should be made to the Deputy Chief Council of the Real Property Division. These requests shall include the following:

- Municipality in which the wall is located.
- All highway plans.
- Information about when and how the SR was taken over as a State Highway, if applicable.
- Confirmation/evidence of who constructed the wall.
- A statement on the function of the wall.

- Evidence of who, if anybody, has maintained the wall. This would include any District maintenance activity related to the wall and any evidence showing the local government or others have maintained the wall.
- Any indication of why the issue has arisen. Is the wall failing?
- Any other relevant facts.

If it is determined that the wall is PennDOT's responsibility, the repairs will need to be completed by PennDOT. The wall will also need to be inventoried in BMS2 and inspected on a regular cycle. If it is not PennDOT's responsibility, a letter shall be sent to the property owner as a notification of the issues that need to be addressed, following the format found in Publication 23 for encroachment. Since the OCC review process can take some time, documented communication with the property owner can begin before a final determination is made in order to make the property owner aware of the situation and see if any repairs are planned. Additionally, OCC has confirmed that inspecting the walls does not affect liability, and it is recommended to look into the safety of these walls to determine what needs to be repaired or addressed.

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## 3.1 GENERAL

Highway bridges shall be load rated in accordance with The AASHTO Manual for Bridge Evaluation (MBE), as supplemented by this Manual. Load Ratings shall consider the structural conditions, material properties, live loads, and traffic conditions at the bridge site.

Load Ratings shall be performed using either the LFR, ASR, LRFD or LRFR method.

## 3.2 LOADS

### 3.2.1 Dead Loads

The bridge analysis should include only those dead loads that the bridge currently carries. Typically, future dead loads such as future wearing surface used in design are not to be included in the analysis and rating for safety inspections. The analysis/rating from the design computations for a new or rehabbed bridge (including future loads) may be utilized for NBIS when the bridge conditions have not changed from design assumptions and the resulting conservative ratings are acceptable for posting and permit vehicles. However, when re-analysis of the bridge is required, only those current dead loads on the bridge are to be included.

Dead loads are to be fully documented in the analysis computations, including the listing of iron or steel shape properties and concrete, steel and/or other materials' weight computations. Member shapes not shown on plans shall be measured in the field and identified using steel shapes catalogs corresponding to the date of construction.

### 3.2.2 Live Loads

#### 3.2.2.1 NBIS REQUIREMENTS

NBIS requires all States to load rate their highway bridges for two purposes:

- To provide a uniform measure of the live load capacity of the nation's bridges in the NBI for planning and programming purposes.
- To ensure bridges are properly posted for the legal load configurations used in individual States

1. **Live Load Rating for planning and programming purposes (NBI):** Historically, the AASHTO HS20 loading has served as the only live load needed for NBI planning purposes, primarily because it had been the design load for most bridges using either Load Factor Design (LFD) or Allowable Stress Design (ASD) methodologies. However, with the adoption of the Load and Resistance Factor Design (LRFD) method by AASHTO in 2000, the HL-93 is a better measure, especially for the LRFD bridges. In Pennsylvania, the PHL-93 loading has been accepted by FHWA as Pennsylvania's design vehicle.

   Because the number of LRFD bridges is still a limited portion of the in-service bridge inventory, FHWA has determined it was not cost-effective to require the re-rating of the ASD and LFD bridges for the new HL-93 live loading at this time.

2. **The State's legal load(s) for bridge load posting evaluations**: The legal load vehicles are used to determine the need to post weight restrictions on the bridge for public safety. Inventory and Operating level ratings are required for all legal load configurations (Pennsylvania Bridge Posting Vehicles). Note: For LFD and ASD ratings, the HS20 loading serves dual purposes for both planning and posting and should be designated as the NBI rating.

#### 3.2.2.2 PA BRIDGE POSTING VEHICLES

The following vehicle configurations represent the legal load vehicles in PA and shall be used to establish the need for a bridge restriction under §4902(a) of the PA Vehicle Code. All highway bridges in PA, regardless of the analysis or rating method used, are to be rated for these bridge posting vehicles as part of the bridge safety inspection program. If the bridge's Safe Load Capacity (SLC) for any of the posting vehicles is less than their legal weight, the bridge is to be posted. SLC is discussed in IP 4.3.2. Posting Policy is discussed in IP 4.

- **AASHTO HS20 vehicle**
  A. Vehicle Load Type
  - Design vehicle.
  - Standard lane load as per Figure M6B.6.2-2.

  B. Vehicle Geometrics
  - The axle weights and spacings are shown in Fig IP 3.2.2.2-1.
  - For the HS20 truck width and transverse wheel location, see Figure M 6B.6.2-1.

  C. Posting Considerations
  - Can be used to determine weight restrictions applicable only to Combination Vehicles.
  - Combination vehicles have a practical minimum weight of 10 tons.

- **AASHTO H20 vehicle**
  A. Vehicle Load Type
  - Design vehicle.
  - Standard lane load as per Figure M6B.6.2-2.

  B. Vehicle Geometrics
  - The axle weights and spacings are shown in Fig IP 3.2.2.2-1.
  - Width of the H20 is the same as the HS20 truck.
  - Transverse wheel location is the same as the HS20 truck.

  C. Posting Considerations
  - If the bridge's SLC for the H vehicle is 19 tons or greater, the posting shall be governed by the lesser of the ML80 SLC rating or the TK527 SLC rating.
  - If the bridge's SLC for the H vehicle is less than 19 tons and governs the rating, contact the BIS for further analysis options and alternate guidance can be approved.

- **ML80 vehicle**
  A. Vehicle Load Type
  - The ML80 loading is representative of the axle weights allowed by the Vehicle Code. It is not a notional load. All axles shall be considered when determining force effects.

  B. Vehicle Geometrics
  - The axle weights and spacings are shown in Fig IP 3.2.2.2-1.
  - Width of the ML80 is the same as the HS20 truck.
  - Transverse wheel location is the same as the HS20 truck.

  C. Computing Live Load Effects
  - When computing the ML80 Rating Factor (RF), use the axle spacings and weights for the posting vehicle shown in Figure IP 3.2.2.2-1.
  - For the posting vehicle, 3% was added to the ML80 maximum registered weight to account for the tolerance allowed by the Vehicle Code for the portable scales used in truck weight enforcement efforts. The axle weights shown in Fig IP 3.2.2.2-1 have this scale tolerance included.

  D. Determining ML80 rating (in Tons)
  - Use the maximum registered weight of 73,280 lbs. (36.64 Tons) for the weight (W) of ML80 truck. The 3% scale tolerance used to compute the live load effect (L) is included only in computing RF.
  - Substituting that maximum registered ML 80 weight:
    For ML80 $RT = (RF)*W$ or $RT = (RF)*(36.64)$.
  - Use RT to determine the IR, OR and SLC for the bridge.

- **TK527 vehicle**

A. Vehicle Load Type
- The TK527 loading is representative of the axle loads allowed by the Vehicle Code. It is not a notional load. All axles shall be considered when determining force effects.
- The TK527 vehicle represents a series of 5 to 7 axle trucks that were adopted as PA legal loads through Act 37 of 2001. The live load effects of various configurations and axle loadings were studied to determine a configuration to envelop the entire group. The seven-axle motor vehicle with a maximum gross weight of 80,000 lbs. produces moments and shears in excess of the five and six axle vehicles allowed under that law and was selected as the bridge posting vehicle to represent the series.

B. Vehicle Geometrics
- The axle weights and spacings are shown in Fig IP 3.2.2.2-1.
- Width of the TK527 is the same as the HS20 truck.
- Transverse wheel location is the same as the HS20 truck.

C. Computing the Live Load Effects
- When computing the TK527 Rating Factor (RF), use the axle spacings and weights for the posting vehicle shown in Figure IP 3.2.2.2-1.
- For the posting vehicle, 3% was added to the TK527 maximum registered weight to account for the tolerance allowed by the Vehicle Code for the portable scales used in truck weight enforcement efforts. The axle weights shown in Fig IP 3.2.2.2-1 have this scale tolerance included.

D. Determining the TK527 rating (in Tons)
- Use the maximum registered weight of 80,000 lbs. (40 Tons) for the weight (W) of the TK527 truck. The 3% scale tolerance used to compute the live load effect (L) is included only in computing RF.
- Substituting that maximum registered TK527 weight:
  For TK527 RT = (RF)W or RT = (RF)*(40).
- Use RT to determine the IR, OR and SLC for the bridge.

COMPARISON OF LIVE LOAD EFFECT OF PA BRIDGE POSTING VEHICLES: For information only, a comparison of the bending moment effects of the bridge posting vehicles for various span lengths is illustrated in Figure IP 3.2.2.2-2. This figure is not an acceptable substitute for a rigorous analysis. The P82 Permit Load is shown for information only.

A table of simple span live load moments and shears for each of the posting vehicles for various span lengths are shown in Appendix IP 03-A.

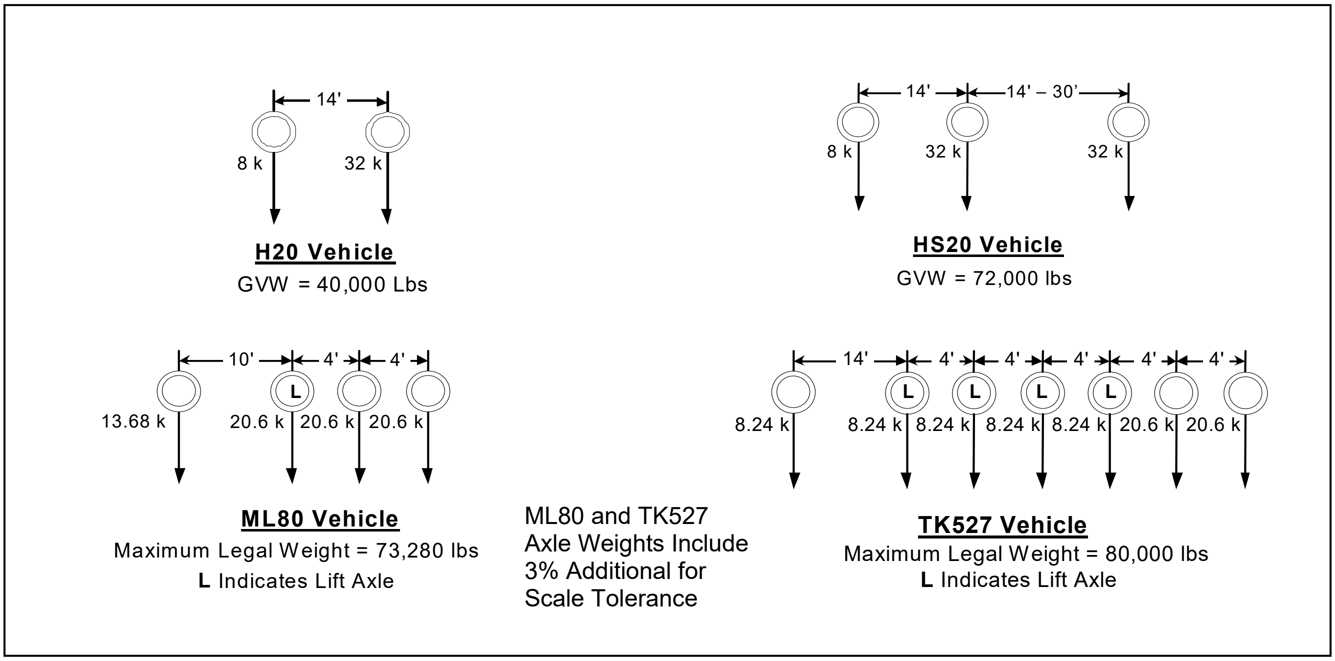


Figure IP 3.2.2.2-1 Bridge Posting Vehicles ─ Axle Configuration and Axle Weights

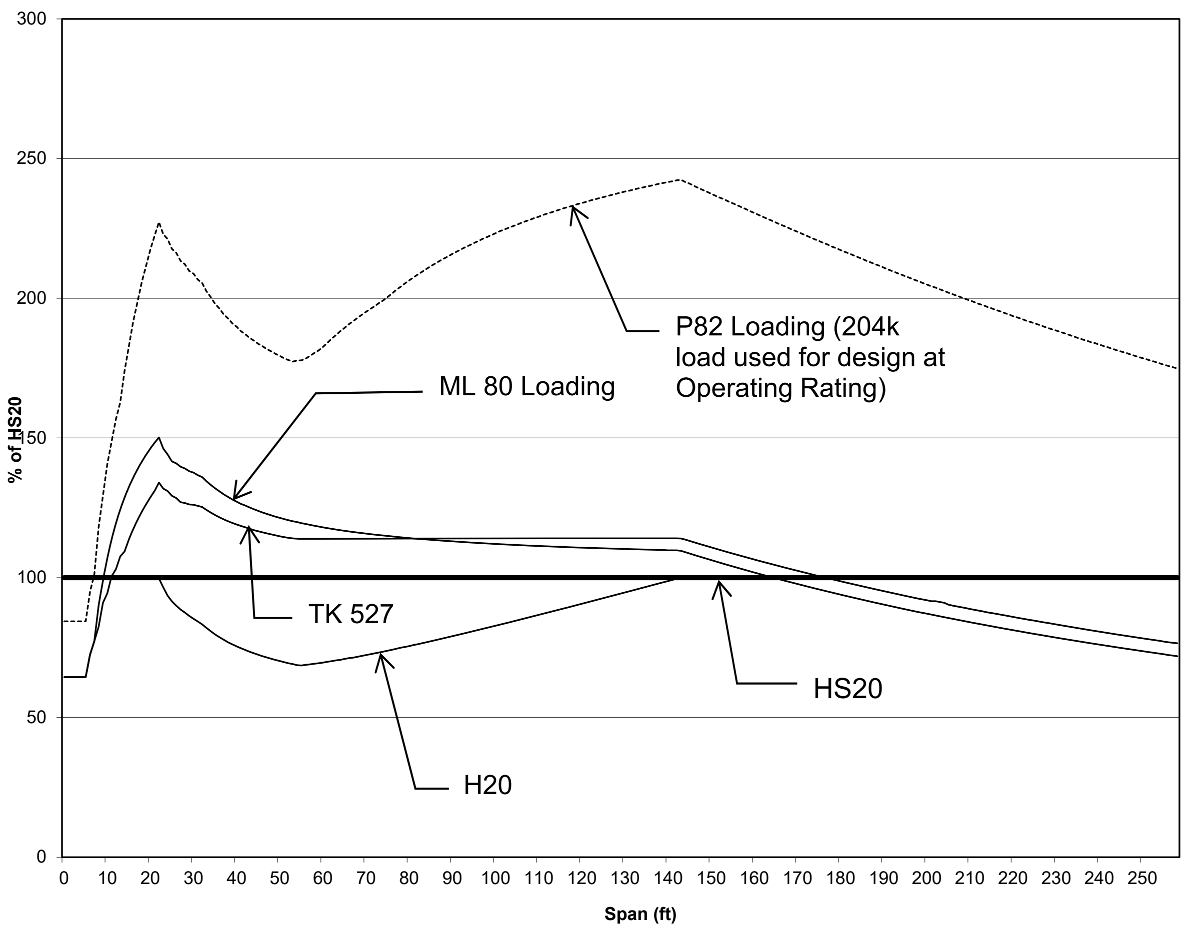


Figure IP 3.2.2.2-2 Comparison of Bridge Posting Vehicle Simple Span Bending Moments

#### **3.2.2.3** AASHTO TYPICAL LEGAL LOADS

AASHTO identifies three typical legal loads that may be used for posting evaluations. These are the Type 3, Type 3S2 and Type 3-3 Vehicles. Because the live load effect produced by PA's posting vehicles is greater than the live load effects produced by these three AASHTO legal loads, the AASHTO legal loads are not used for rating or posting evaluations in PA. The axle weights and configurations for the three AASHTO legal loads are shown in Appendix M D6A.

#### **3.2.2.4** AASHTO SPECIAL HAULING VEHICLES

FHWA Memorandum HIBT-10 dated November 2013 identified four AASHTO Special Hauling Vehicles (SHVs) that are to be included in rating and posting evaluations. These are the SU4, SU5, SU6, and SU7 vehicles. The department conducted a parametric study in 2016 that verified PA's posting vehicles envelope the SHVs for force effects and load posting. Therefore, the AASHTO SHVs are not used for rating or posting evaluations in PA. The axle weights and configurations for the four AASHTO SHVs are shown in Appendix M D6A.

#### **3.2.2.5** FHWA FAST ACT EMERGENCY VEHICLES

FHWA Memorandum HIBS-1 dated November 2016 identified two Emergency Vehicles (EVs) that are to be included in rating and posting evaluations. These are the EV2 and EV3 vehicles. The EV2 and EV3 were developed by FHWA to encompass the effects of the family of emergency vehicles covered by the Fixing America's Surface Transportation (FAST) Act. The department conducted a parametric study in 2019 that verified PA's posting vehicles envelope the EVs for force effects and the specific conditions outlined for load posting. Therefore, the EVs are not required to be considered in posting evaluations in PA. Load rating evaluations for these vehicle types, however, shall be performed during the bridge's next rating update to be in compliance with FHWA's Memo HIBS-1. The inclusion of EV2 and EV3 in the load rating analysis shall be applicable to all bridges with the exception of those rated using Engineering Judgment. For bridges rated using Engineering Judgment, CONSPAN bridges or similar structures located on or within reasonable access to the Interstate System, defined as within one-road-mile from access to and from the Interstate System, contact the Bridge Inspection Section for guidance on EV2 and EV3 load rating analysis. Atypical bridge decks (those less than 8" thick or with conditions affecting their capacity) shall also be evaluated for these vehicles. The axle weights and configurations for the two EVs are shown in the FHWA Memo HIBS-1. The memo also provides guidance on application of multiple presence and live load factors.

### **3.2.3 Impact Loads**

Impact loads shall be accounted for in the analysis and determined in accordance with AASHTO.

## **3.3 DISTRIBUTION OF LIVE LOADS ON LONGITUDINAL MEMBERS**

Three methods are acceptable for distributing live load laterally to longitudinal members: Lever Rule, Simplified Line Girder, or Refined Analysis. Other methods of lateral live load distribution for longitudinal members must be approved by the Assistant Chief Bridge Engineer - Inspection.

### **3.3.1 Lever Rule**

The lever rule determines the portion of the live load distributed to a beam line by assuming the deck acts as a rigid beam and summing the moments of wheel loads about one support (beam line) to find the reaction at the other support (beam line). The lever rule is used to determine the live load lateral distribution to longitudinal members in the following situations:

1. To each girder or truss of a two or three-girder or two or three-truss bridge.
2. To exterior beams or stringers of beam-slab bridges.
3. To interior beams or stringers of beam-slab bridges with geometric properties that fall outside the range of applicability for using the Simplified Line Girder Method.

The lever rule can be used in other situations (box beams w/ S > 18', wood beams, steel grid deck on steel beams, etc.). See PD Table 4.6.2.2.2b-1 and PD Table 4.6.2.2.3a-1 for more information.

When the lever rule is used to distribute live load, a reduction in load intensity for multiple lanes of live load may be applied to truss or girder bridges with two main longitudinal members. See the appropriate sections of AASHTO for LFD methodology (SD 3.12) or LRFD methodology (AD 3.6.1.1.2). The reduction is only to be used when 3 or more lanes of traffic are loading a particular member.

### 3.3.2 Simplified Line Girder (AASHTO Distribution Factor)

#### 3.3.2.1 GENERAL APPLICATION

The Simplified Line Girder is an empirical method used to distribute the vehicular live load laterally to the longitudinal girders. The Department allows either set of simplified line girder live load distribution factors that AASHTO developed for its two main design methodologies to be used for bridge analysis and rating:

- LFD Distribution Factors (or S-Over Factors) See IP 3.3.2.2.
- LRFD Distribution Factors See IP 3.3.2.3.

To use the AASHTO simplified line girder analysis distribution factors, the bridge must meet the specific applicability requirements in the appropriate AASHTO design specification and the following general constraints from AD 4.6.2.2.1 (as modified by PD 4.6.2.2.1):

- Deck width is constant.
- Four or more beams in the cross-section.
- Beams are reasonably parallel and have approximately the same stiffness (see IP 3.3.3.3).
- Roadway part of deck overhang does not exceed 3.0' from centerline of exterior beam.
- Structural members are not horizontally curved. (See as specified in AD 4.6.1.2.1 and PD 4.6.1.2.1).
- Skew angle is greater than 70 degrees (see IP 3.3.3.1).
- Cross-section is consistent with cross-sections shown in AD Table 4.6.2.2.1-1.
- When there is significant deterioration, the simplified line girder analysis is no longer an acceptable method.

Exceptions to these general constraints are addressed in IP 3.3.2.3.

#### 3.3.2.2 AASHTO LFD DISTRIBUTION FACTORS (S-OVER FACTORS)

The rules of applicability and formulas for the determination of the AASHTO LFD Distribution Factors (S-Over Factors) are outlined in SD 3.23.

A reduction in load intensity to account for multiple lanes of live load shall not be used in conjunction with the AASHTO LFD Distribution Factors. The reduction factors of SD 3.12 do not apply in this situation.

Previous Department policy specified that live load distribution factors for prestressed or reinforced concrete adjacent box beam bridges should follow the guidance provided in the 1983 AASHTO instead of SD 3.23.4. Central Office Bureau of Bridge, Bridge Inspection Section performed a review of the two methods in 2022 and determined that there is not a significant difference with the results provided by the two methods. The guidance, provided with this publication update, outlined in SD 3.23.4 shall be used for live load distribution factors for all concrete adjacent box beam bridges.

#### 3.3.2.3 AASHTO LRFD DISTRIBUTION FACTORS

The rules of applicability and formulas for the determination of the AASHTO LRFD Distribution Factors are outlined in AD 4.6.2.2 and PD 4.6.2.2.

A reduction in load intensity to account for multiple lanes of live load shall not be used in conjunction with the AASHTO LRFD Distribution Factors. The multiple presence factors of AD 3.6.1.1.2 do not apply in this situation.

### 3.3.3 Bridges with Special Girder Geometry

A refined analysis may be required for bridges with girders that are skewed, curved, or variably spaced.

#### 3.3.3.1 SKEWED BRIDGES

Skewed bridges are defined as structures having their highway centerline (or a parallel thereto) intersect a line parallel to the major axis of substructure units at an angle (θ) other than 90° (normal). For the definition of skew angle, refer to DM4 PP 3.2.2.

For bridges analyzed using the simplified girder analysis methodologies, the distribution of live load bending moment for skewed bridges is sufficiently in agreement with the AASHTO distribution factors developed for normal bridges and no adjustment is needed. However, the shear forces in a skewed bridge are attracted to the stiffer obtuse corner of the bridge and may significantly increase the vertical shear and girder reactions over the values calculated using the simplified girder distribution factors for normal bridges.

Shear skew adjustment factors for the simplified girder distribution factors are available to use on girders with skew angles less than 90º, but greater than 30º (see DM4 PD 4.6.2.2.3c). A more refined analysis is required for skewed bridges outside of the limits of applicability listed in DM4.

No skew adjustment factor for bending moment is required for ratings using simplified girder analysis or lever rule.

#### 3.3.3.2 CURVED BRIDGES

Bridges with horizontally curved members meeting the requirements of AD 4.6.1.2 and PD 4.6.1.2 are to be analyzed using a refined method to accurately model actual conditions.

#### 3.3.3.3 SPLAYED (VARIABLY SPACED) BEAM BRIDGES

In cases where straight beams are not parallel and the spacing varies along their length, the Live Load distribution to the beams, for non-refined analysis methods, is subject to interpretation. Generally, the average beam spacing or beam spacing at the centroid of contributing deck area, or some other weighted average may be used at the discretion of the rating engineer. See AD C4.6.2.2.1 for additional information. All assumptions made here are to be documented with the analysis. The beam spacing used shall be between the narrowest and the widest beam spacing present. A refined analysis may be warranted to accurately model actual conditions.

### 3.3.4 Refined Method of Analysis

The refined method of analysis for determining live load distributions on longitudinal members shall be done in accordance with AASHTO and PennDOT DM4. Programs used for this analysis method shall be Department approved (see IP 3.8).

### 3.3.5 Temporary Measures Present

In some instances, temporary measures (e.g., barrier, guide rail, etc.) are placed on a bridge to restrict live load from reaching portions of the bridge to direct traffic away from deteriorating portions of the structure. The load rating may consider the temporary measure for placement of the live load only if it is sufficient to physically restrict traffic. If the measure is not able to physically restrict traffic, then the load rating analysis must consider live load effects as if the temporary measure did not exist.

#### 3.3.5.1 RATING PROTECTED PORTIONS OF STRUCTURE

The protected portions of the structure must still rate for all remaining permanent loads per one of the following cases:

1. Protected beam to carry self-weight and tributary weight of temporary measure.
2. Protected beam to carry self-weight only. Temporary measure carried by adjacent beam(s).

If the protected beam (or beams) is not sufficient to sustain its own self weight, it must be removed. All remaining dead load, including the temporary measure, shall be included in analysis of structure that remains in place.

**3.3.5.2** PLACEMENT OF LIVE LOAD

For standard lane widths, the truck shall be positioned transversely for maximum load effect, but the center of any wheel need not be closer than 2’ from the edge of the temporary measure.

The actual lane widths shall be taken into consideration to produce the maximum load effect. Substandard lane widths may cause the center of the wheel line to be placed closer than 2’ from the edge of the temporary measure.

For narrow bridges not posted as a single lane or one truck at a time, two trucks should be positioned on the deck. Center the truck in the available lane space, which may cause the center of the wheel line to be placed closer than 2’ from the edge of the temporary measure.

**3.3.5.3** DISTRIBUTION FACTORS WITH TEMPORARY MEASURES

When temporary measures are utilized, approximate distribution factors may no longer apply to the remaining beams which should then be analyzed by lever rule or refined methods. If the remaining beams can still be shown to fall within the range of applicability for simplified line girder distribution factors, the interior beam closest to the temporary measure should be analyzed as an exterior beam.

## 3.4 DISTRIBUTION OF LIVE LOADS ON TRANSVERSE MEMBERS

If the bridge deck is directly supported by the transverse members only, the portion of the live load distributed to the transverse members is determined in accordance with SD 3.23.3.2 or AD 4.6.2.2.2f.

For transverse members not meeting the applicability requirements of the foregoing sections or for transverse members supporting a deck with longitudinal stringers, the Lever Rule or Refined Analysis should be used to distribute the live load.

Other methods of lateral live load distribution for transverse members must be approved by the Chief Bridge Engineer.

### 3.4.1 Lever Rule

The lever rule determines how the live load is distributed to a transverse member by assuming the deck acts as a rigid beam and summing the moments of wheel loads about one support (transverse member) to find the reaction at the other support (beam line). The lever rule is used to determine the live load lateral distribution to transverse members in the following situations:

3. Through stringers to end and interior floorbeams for deck floor systems with longitudinal members supported by floorbeams.
4. To interior and end floorbeams for deck floor systems with the deck supported directly by the floorbeams and with geometric properties that fall outside the range of applicability for using the Simplified Distribution Factors.

When the lever rule is used to distribute live load, a reduction in load intensity for multiple lanes of live load may be applied to the transverse members. See the appropriate sections of AASHTO for LFD methodology (SD 3.12) or LRFD methodology (AD 3.6.1.1.2).

### 3.4.2 Refined Method of Analysis

The refined method of analysis for determining live load distributions on transverse members shall be done in accordance with AASHTO and PennDOT DM4. Programs used for this analysis method shall be Department approved (see IP 3.8).

### 3.4.3 Distribution Factors for Transverse Members

#### 3.4.3.1 GENERAL APPLICATION

The use of live load distribution factors for transverse members is applicable only in bridge decks that are directly supported by floorbeams only (no stringers in the deck system).

Distribution Factors are an empirical method that uses an estimation of the relative stiffness of the deck to the floorbeams to distribute the vehicular live load distribution laterally to the transverse members. The Department allows either set of live load distribution factors that AASHTO developed for its two main design methodologies to be used for bridge analysis and rating:

- LFD Distribution Factors See IP 3.4.3.2.
- LRFD Distribution Factors See IP 3.4.3.3.

The live load distribution on transverse members not meeting the applicability requirements for using LFD or LRFD Distribution Factors shall be determined using the Lever Rule or by a Refined Analysis.

#### 3.4.3.2 AASHTO LFD DISTRIBUTION FACTORS (S OVER FACTORS)

The rules of applicability and formulas for the determination of the AASHTO LFD Distribution Factors (S-Over Factors) are outlined in SD 3.23.

A reduction in load intensity to account for multiple lanes of live load shall not be used in conjunction with the AASHTO LFD Distribution Factors. The reduction factors of SD 3.12 do not apply in this situation.

#### 3.4.3.3 AASHTO LRFD DISTRIBUTION FACTORS

The rules of applicability and formulas for the determination of the AASHTO LRFD Distribution Factors are outlined in AD 4.6.2.2.2f. At the discretion of the rating engineer, the LRFD Distribution Factors may be used in an LFD analysis and rating.

A reduction in load intensity to account for multiple lanes of live load shall not be used in conjunction with the AASHTO LRFD Distribution Factors. The multiple presence of live load factors of AD 3.6.1.1.2 do not apply in this situation.

### 3.4.4 Guidance for Closely Spaced Floorbeams

For determining the moment and shear in interior floorbeams spaced between 9.5' and 15.5', the lane load will control the HS loading. This lane load is to be applied as the distributed load (640 lbs. per linear foot of load lane) plus a concentrated load (26,000 lbs.). In this case, the 26 kip concentrated load normally used for checking shear on longitudinal beams is to be applied to calculate HS live load moment and shear for the transverse beam. The BAR7 Program uses this procedure to determine the applied forces on floorbeams.

### 3.4.5 Cross Girders

Cross girders (or cross tie girders) are transverse members that span from one column or substructure unit to another and transfer the superstructure loads from several girders to the substructure. Cross girders are used where the use of traditional wall piers or multi-column bents would otherwise obstruct highways or railroads beneath the structure. Typically, cross girders are steel members, although some pre/post tensioned concrete members have been used.

Although cross girders are considered to be part of the substructure, they function in a similar manner as floor beams on trusses and are to be considered as main members carrying live load. Cross girders are to be load rated using the bridge posting vehicles. Live load and dynamic load allowance/impact shall be calculated in the same manner as it is for transverse floorbeams. If the safe load capacity of the cross girder is not sufficient to safely carry the legal loads, the bridge is to be posted for a weight restriction. Because they are NSTMs, cross girders should be inspected carefully.

## 3.5 ANALYSIS OF MULTI-SPAN PRESTRESSED CONCRETE BEAM BRIDGES

For multi-span prestressed concrete girder bridges without a deck joint over a pier, the details of the beams and deck reinforcement must be reviewed to determine the ability of the superstructure to act in a continuous manner for those spans. The rating engineer must carefully review the original design computations and design/shop drawings to ascertain the level of continuity that the structure can achieve. If full continuity for superimposed dead loads and live loads cannot be realized because reinforcement area or details are inadequate, the bridge should be analyzed as a series of simply supported spans.

## 3.6 LIVE LOAD CAPACITY RATING METHODS

### 3.6.1 Live Load Rating Methods for Bridge Posting Evaluations

The following methods are acceptable, within the applicability limits of AASHTO and this Manual, to determine the live load capacity of bridges:

- Allowable Stress Method
- Load Factor Method
- Load and Resistance Factor Rating Method (LRFR)
- Load and Resistance Factor Design Method (LRFD)
- Engineering Judgment
- Load Testing (use of this method requires prior approval of the Assistant Chief Bridge Engineer - Inspection)
- Assigned Ratings

The rating engineer may select the most appropriate of the approved rating methods for the posting evaluation. LRFR ratings may be used as warranted which shall be determined in conversation with the Bridge Inspection Section. Other rating methods (not listed above) must also be approved by the Assistant Chief Bridge Engineer - Inspection.

For the rating of a single bridge component, one method shall be used to determine the live load capacity for all of the bridge posting vehicles. More than one method of rating may be used on a bridge to evaluate different components (example: ASD would be used for truss members while the rating engineer may elect to use LFD for the floor system.)

The ratings of the various bridge posting vehicles used for the bridge posting evaluations are to be entered in the appropriate subfields in BMS2 Items IR10 Inventory Rating and IR11 Operating Ratings. See the BMS2 Coding Manual (Pub 100A) for more detailed instructions. The rating method used for the controlling member's posting evaluation is to be recorded in BMS2 Item IR06.

#### **3.6.1.1** ENGINEERING JUDGMENT

Engineering judgment alone shall not be used to determine the live load capacity of a bridge component where sufficient structural information is known.

A bridge rating based upon engineering judgment should consider, but is not limited to, the following, factors:

- Type of bridge
  - Stone masonry arch bridges and metal arch culverts are considered higher risk bridges
  - Concrete beam bridges and concrete arches are considered lower risk
- Condition of the load carrying components
- Section loss and location of the section loss
- Year built

- Material properties of members
- Redundancy of load path
- Traffic characteristics
  - Number and size of trucks
  - Loading
  - Projected traffic
- Performance of bridge under current traffic
  - Evidence of distress
  - Evidence of excessive movement under load
- Bridge restrictions (Past, current, and proposed)

When using engineering judgment for the main live-load carrying members/components, ratings are to be determined for all of the bridge posting vehicles at the Inventory and Operating Rating levels for the live load carrying component of the bridge. These vehicle ratings are needed for the posting evaluation.

One reasonable approach to determine the ratings for the various vehicles is:

1. Establish the Safe Load Capacity for the posting vehicle that represents the most critical loading (shear or moment).
2. Multiply the critical vehicle's rating factor for SLC by the ratio of the live load effect of the critical vehicle to the live load effect of the other posting vehicle. For example, if the ML80 is the critical posting vehicle, to determine the HS rating from the bending moment live load effect:
   *Rating Factor for* $SLC_{HS}$ = *Rating Factor for* $SLC_{ML80}$ * $(LLM_{ML8}0/LLM_{HS})$
   *where LLM = Live Load Moment and* $SLC_{HS} = RF \ x\ W$
3. To determine the OR and IR from the SLC for each vehicle, use appropriate ratios as determined by the engineer. Suggested ratios are as follows:
   $OR = SLC * 1/\ f$

   $IR = 60\% * OR$
   *where f = Safe Load Capacity Reduction Factor (see IP 4.3.2)*

For bridges posted for weight restrictions based solely on the condition of components other than the main load carrying members, the restrictions may be less than the Inventory Ratings, but not greater than the Safe Load Capacity for the main load carrying members (see IP 4.4.4).

BRIDGES WITHOUT PLANS: In PA, the vast majority of bridges either have sufficient design drawings or the properties of the members can be verified through field measurements to support an engineering analysis. However, for many older concrete or masonry structures (including slabs, beams, and arches), the structural components either cannot be measured (arches) or critical details (e.g., reinforcement details) are not known with sufficient confidence to evaluate through computations. For these structures, a rating based on engineering judgment by a qualified engineer familiar with the bridge and the factors listed above may be appropriate. A suggested method for applying engineering judgment alone to determine the ratings of such bridges without plans is contained in Appendix IP 03-B.

**3.6.1.2** ASSIGNED LOAD RATINGS

In accordance with FHWA's Assigned Load Ratings Memo dated September 29, 2011, certain bridges currently in service with benign condition deterioration, designed and checked by modern methods for modern bridge loadings, and with no changes to dead loads and State legal and routine permit vehicular loads since the design was completed may adequately have their load carrying capacity calculated. A separate analysis would not be required. FHWA has determined that inventory and operating ratings may be assigned based on the design loading, provided the following conditions are met:

1. The bridge was designed and checked using either the AASHTO Load and Resistance Factor Design (LRFD) or Load Factor Design (LFD) methods to at least PHL-93 or HS-20 live loads, respectively; and
2. The bridge was built in accordance with the design plans or shop drawings; and
3. No changes to the loading or structure condition have occurred that could reduce the inventory rating below the design load; and

4. An evaluation has been completed and documented, determining that the force effects from State legal loads or permit loads do not exceed those from the design load (this will be true for all bridges designed by PennDOT standards for the PHL-93 vehicle); and
5. The checked design calculations and relevant computer input/output information must be accessible and referenced or included in the individual bridge records.

A summary of the assigned load rating, which demonstrates these five conditions are met, is to be included in the bridge file and approved by the individual charged with the overall responsibility for load rating bridges (Assistant District Bridge Engineer-Inspection), or by an individual meeting 23 CFR 650.309(d) qualifications and delegated, in writing, this approval authority. If any of these conditions cannot be met for a bridge at any point during its service life, load ratings cannot be assigned and must be determined by other methods defined in IP 3.6.1.

If complete design files have not been retained for existing bridges, design plans that clearly identify the loading as at least PHL-93 or HS-20 and bear the stamp of a licensed professional engineer may be used by the individual responsible for load rating under 23 CFR 650.309(d) as the basis for an assigned load rating. The approval needs to be documented as the basis for the assigned rating and become part of the official bridge records. This information demonstrates satisfaction of conditions (1) and (5) above. Conditions (2), (3), and (4) still need to be met.

For State bridges designed using LRFD, load ratings based upon LF or LRFD methodology must be performed for APRAS purposes.

#### **3.6.1.3** APPLICABILITY OF ANALYSIS METHODS

Ensure the load analysis software/methodology is suitable to address the given structure or structure elements. Utilize advanced or refined methods of analysis, as appropriate, for complex structures, complex components of structures, and/or complex conditions of structures or components of structures.

#### **3.6.1.4** EFFECTS OF DETERIORATION, DAMAGE, AND OTHER DEFECTS

Ensure element conditions such as deterioration, impact damage, or other defects are appropriately assessed and modeled in the load rating analysis. Consider that deteriorated, damaged, or otherwise defective elements or sections of a structure may behave differently than the structure as originally designed and therefore different failure modes may govern the load capacity. For example, a severely corroded girder web that has become disconnected to a flange may have member properties and structural function that are much different than what is obtained using a simplified approach.

### 3.6.2 Live Load Capacity Rating for the NBI

FHWA requires that live load capacity ratings are to be performed for either the HS20 or the HL-93/PHL-93 loadings. BMS2 Item IR05 is used to designate which live load vehicle is reported to FHWA in the annual NBI report. The NBI load ratings at both IR and OR rating levels are to be reported in BMS Items IR10 and IR11 for tons and IR20 and IR21 for rating factors. Accordingly, for the NBI rating (IR05 = 1), provide the rating for the appropriate combination of design methodology and live loading configuration as shown in Table IP 3.6.2-1.

Table IP 3.6.2-1 below summarizes the above requirements for the live load configuration for the NBI rating.

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<table>
<tr><th colspan="5">Table IP 3.6.2-1 Live Load Configuration to be Used for NBI Rating</th></tr>
<tr><th>Design Method For Superstructure</th><th>Bridge Placed in Service</th><th>Rating Purpose</th><th>NBI Rating</th><th>Rating Method</th></tr>
<tr><td rowspan="6">LRFD</td><td rowspan="3">On or After 2011</td><td>Design rating or reconstruction</td><td>PHL-93 (Rating Factors)</td><td>LRFD</td></tr>
<tr><td>Re-rating due to loading or structure condition change</td><td>PHL-93 (Rating Factors)</td><td>LRFD or LRFR</td></tr>
<tr><td>Design Rating for Superstructure Replacement</td><td>PHL-93 (Rating Factors)</td><td>LRFD</td></tr>
<tr><td rowspan="3">Before 2011</td><td>Design rating</td><td>PHL-93 (Rating Factors) or HS20 (Tons)</td><td>LRFD or LFD</td></tr>
<tr><td>Re-rating due to loading or structure condition change</td><td>PHL-93 (Rating Factors) or HS20 (Tons)</td><td>LFR, LRFD, or LRFR</td></tr>
<tr><td>Design Rating for Superstructure Replacement</td><td>PHL-93 (Rating Factors) or HS20 (Tons)</td><td>LRFD</td></tr>
<tr><td>ASD, LFD, or Other</td><td>Any time</td><td>Design rating or Re-rating</td><td>HS 20 (Tons)</td><td>LFR or ASR</td></tr>
</table>

### 3.6.3 Allowable Concrete Tensile Stresses for Prestressed Concrete Beams

It has been Department policy to limit the design tensile stresses in concrete of the beam to a maximum value of $3\sqrt{f'c}$ to avoid potential long-term problems related to fatigue of the strands. This same value should be used for bridge ratings using the LFR method.

However, if the ratings using $3\sqrt{f'c}$ would necessitate a bridge weight restriction, the allowable concrete tension may be raised to a value of $6\sqrt{f'c}$ if a fatigue check of the beams results in adequate remaining fatigue life.

## 3.7 MATERIAL TESTING, STRENGTH OF MATERIALS, AND INSTRUMENTATION FOR ANALYSIS

### 3.7.1 Non-Destructive Testing

Non-Destructive Testing (NDT) is a critical part of bridge inspection and evaluation. NDT is used to supplement the visual inspection by providing information regarding the condition of bridge components that is not detectable by a visual inspection alone. NDT is a generic name given to repeatable processes applied to components or structures to determine the condition of the structure's material without compromising structural integrity. To ensure accurate NDT results and resulting programming decisions, properly trained individuals should carefully perform the tests. The various NDT methods and their capabilities for detecting defects in different materials are discussed in M 5.2.

### 3.7.2 Strength of Materials

The strength of materials for analysis shall be determined using the following hierarchy:

- From the as-built plans.
- From the design plans.
- From specifications at the time the structure was built.

If the material strength/properties are still not known with confidence, the material strengths for steel, concrete and timber may be determined from tables and/or testing (see IP 3.7.2.1, IP 3.7.2.2, and IP 3.7.2.3).

#### 3.7.2.1 STEEL MATERIAL PROPERTIES

The material properties of steel may be determined using the "Date-Built" tables in M6B.5.2.1. If further information or confidence in the material properties is required, the steel may be tested in accordance with M5.3 through M5.6. Material samples removed from the structure shall be documented and removal performed so that the structural integrity of the bridge or its components is not compromised.

#### 3.7.2.2 CONCRETE MATERIAL PROPERTIES

If the original specifications for a bridge are not available, the material properties of concrete may be assumed from the values in Table IP 3.7.2.2-1. If further information or confidence in the material properties is required, the concrete may be tested in accordance with M5.3 through M5.6. Material samples removed from the structure shall be documented and removal performed so that the structural integrity of the bridge or its components is not compromised.

**Table IP 3.7.2.2-1 Specified Concrete Strengths for Department Bridges**

| Date Range | 28-Day Design Strength (psi) | | | | Deck Slab Concrete Strength (psi) | | |
|---|---|---|---|---|---|---|---|
| | Class AAA | Class AA | Class A | Class B | Design Strength | Design (fc) | 28 Day (f'c) |
| 1990-Current | 4500 | 3500 | 3000 | | 4000 | | 4500 |
| 1984-1989 | 4500 | 3500 | 3000 | | 4000 | | 4500 |
| 1968-1983 | 4500 | 3500 | 3000 | | 4500 | | 4500 |
| 1963-1967 | | 3500 | 3000 | 2500 | 3500 | 1000 | 3500 |
| 1950-1962 | | 3000 | 3000 | 2500 | 3000 | 1000 | 3000 |
| 1949 & Earlier | | | 3000 | 2200 | 3000 | 1000 | 3000 |

#### 3.7.2.3 TIMBER MATERIAL PROPERTIES

The material properties of timber are to be determined in accordance with SD Chapter 13 and M 6B.5.2.7 or through testing in accordance with M 5.3 through M 5.6.

### 3.7.3 Instrumentation

Instrumentation of a bridge may be necessary if the structure cannot be accurately modeled by analysis, the structural response to live load is in question, or as a last means to avoid a weight restriction on a significant structure. Non-destructive load testing shall be in accordance with IE 08.

## 3.8 BRIDGE RATING SOFTWARE

Use Department developed or Department approved software for the live load rating of bridges. See DM4, PP 1.4.7 for a list of PennDOT LRFD and LFD engineering programs. See Accepted Commercially Available or Consultant Developed Software on PennDOT's website:
https://docs.penndot.pa.gov/Public/Bureaus/Bridge/AcceptedSoftware/Bridge-Accepted-Software.pdf

## 3.9 LOAD RATING APPROVAL AND DOCUMENTATION

Analyzing and load rating bridges based on the most current field conditions is an essential component to ensure public safety. The results of the load rating analysis are used to determine whether or not a bridge is able to support legal loads. Due to the criticality of this matter, all load ratings must be performed by, or under the direct supervision of, a registered Professional Engineer (PE) in Pennsylvania and shall be signed and sealed by that PE. For a load rating analysis performed by Department personnel, the District Bridge Engineer or the Assistant District Bridge Engineer for Inspection is to sign and seal the load rating analysis. Sealing the load rating indicates that the rating has been reviewed and the items on the Load Rating Quality Control Verification Checklist (Appendix IP 06-B) are satisfied.

Proper documentation of the load rating analysis assists in the review of the load rating and facilitates future load ratings, if required. An integral part of the load rating is a load rating summary page which summarizes the results of the load rating, rating method and assumptions. All load rating analysis shall have a load rating summary page as part of the bridge file. The load rating summary information shall be referenced by inspectors during inspections to identify conditions that are different than accounted for in the load rating analysis. See IP 8.3.2 for proper load rating analysis documentation. A sample load rating summary form is in Appendix IP 03-C.

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## 4.1 GENERAL COMMENTS ON BRIDGE RESTRICTIONS

Size and weight restrictions on vehicles are sometimes necessary to ensure public safety and to safeguard our bridge infrastructure. Because bridges are a critical link for transportation in PA, bridge restrictions must be prudently established to maintain an adequate level of safety without unduly restricting the movement of goods and services (especially emergency services).

There are three basic types of restrictions placed on vehicles crossing over or under bridges:
- Weight restrictions based upon the condition of the bridge.
- Weight restrictions based upon traffic conditions.
- Vertical clearance restrictions.

The needs for these restrictions are ascertained through the bridge safety inspection program. Every effort must be made to minimize both additional restrictions and their resultant economic impact without compromising either public safety or full compliance with NBIS/NTIS. It is the responsibility of the District Bridge Engineer or the bridge owner to ensure that proper bridge restriction signs are installed and maintained for this critical public safety need.

## 4.2 STATUTES AND REGULATIONS REGARDING BRIDGE RESTRICTIONS

The below listed statutes and regulations are important to the issues of bridge restrictions.

- **NBIS (National Bridge Inspection Standards)**
  §650.313 of this federal regulation requires that bridges are to be load rated and if the State legal loads exceed the loads allowed under the Operating Rating, the bridge must be posted for a restriction.

- **PA Statute Act 44 of 1988**
  Act 44 amended the PA Administrative Code of 1929 to enlarge the powers and duties of the Department. In part, Act 44 established the Department's responsibilities for the safe posting of locally owned bridges. Act 44 authorizes and requires the Department to inspect (which includes load rating) and post locally owned NBIS bridges as needed. See IP 1.6.3 for further information on Act 44.

- **TITLE 75 VEHICLES (PA Vehicle Code)**
  Chapter 49 Size, Weight and Load authorizes the Department and local bridge owners to place restrictions on the vehicles using bridges and highways. Two subsections are pertinent to establishing these bridge weight or size restrictions:
  - § 4902(a) Restrictions based upon the condition of the bridge.
  - § 4902(b) Restrictions based upon traffic conditions.

- **TITLE 67 TRANSPORTATION**
  Title 67 sets Department regulations governing transportation issues. Chapters relevant to load posting of bridges include:
  - Chapter 191 Authorization to Use Bridges Posted Due to Condition of Bridge.
  - Chapter 212 Official Traffic-Control Devices.

  An Internet version of TITLE 67 can be found at the website http://www.pacode.com/.

- **NTIS (National Tunnel Inspection Standards)**

§650.513 of the NTIS federal regulation requires that tunnels are to be load rated and if the State legal loads exceed the loads allowed under the Operating Rating, the tunnel must be posted for a restriction. (A copy of the NTIS is in Appendix IP 01-C.)

## 4.3 BRIDGE POSTING EVALUATIONS

### 4.3.1 Requirements for Bridge Posting Evaluations

Each highway bridge is to be evaluated for its ability to safely carry each of the bridge posting vehicles as part of each bridge safety inspection. This posting evaluation is performed once the current bridge conditions are known and the Inventory Ratings (IRs) and Operating Ratings (ORs) for each of the posting vehicles have been determined (or verified from previous inspections). Any bridge that cannot carry the posting vehicles at its safe load capacity must be posted for the appropriate bridge restriction. See IP 3.2.2 for a description of the bridge posting vehicles that represent the various PA legal load configurations.

The posting evaluation must include a recommendation about the need for a bridge restriction that would be governed under **PA Vehicle Code § 4902(a) Restrictions based on the condition of the bridge.** Recommended bridge restrictions may consist of a posted bridge weight limit, a "One Truck at a Time" restriction, or a combination of both. Closure of a bridge is considered to be the most severe weight posting. In special circumstances, other vehicle restrictions may be imposed.

The bridge posting evaluation is the justification for imposing a § 4902(a) restriction and is to be maintained as a part of the permanent bridge record. A licensed professional engineer must prepare the bridge posting evaluation.

If a new restriction or a revision to a previous restriction is needed, the posting evaluation must be sufficiently detailed to fulfill the requirements of TITLE 67 Chapter 212 Official Traffic-Control Devices. Generally, the bridge posting evaluation focuses on the information in the §212 Appendix, item 2 (18), Structural analysis, but other factors pertinent to the load posting (e.g., ADT, ADTT) are to be added as appropriate.

**4.3.1.1** ELEMENTS OF A BRIDGE POSTING EVALUATION:

1. Summary recommendation addressing the need for a bridge restriction based on its condition
    a. If a weight restriction is not needed, state it as so.
        *EXAMPLE:*
        *The Safe Load Capacity (SLC) of the bridge is greater than the live load effects of the bridge posting vehicles and there is no need for a bridge weight restriction under §4902(a) of the PA Vehicle Code.*

    b. If a new restriction or revision to an existing restriction is needed:
        - State proposed restriction(s)

          *EXAMPLE (for format only):*
          - *The following restrictions are to be placed on the bridge under §4902(a) of the PA Vehicle Code:*
            *Bridge Weight Limit 25 Tons Except Combinations 32 Tons*
            *And Bridge Limited To One Truck At A Time*

        - State reason(s) for the restriction
          *NOTE: This reason is intended to succinctly inform the owner and public of the critical safety need for the bridge restriction. Accordingly, this section should not be a detailed description of bridge conditions and ratings. It should be simple and direct language similar to the reasons listed in BMS2 Item VP06 Reason for Posting/Closing the Bridge.*

          *EXAMPLES OF REASONS FOR RESTRICTION:*
          - *This bridge restriction is necessary because the main bridge members (floor beams) are deficient and cannot carry the legal loads safely.*

          - *This bridge restriction is necessary because the main bridge members (girders) are deficient and cannot carry legal loads safely.*

2. Summary of bridge ratings for each bridge posting vehicle.
    a. List IR, OR, and SLC for each posting vehicle and appropriate load effects (e.g., moment, shear, axial compression) for controlling superstructure member(s). This is generally done in table format. If "One Truck At A Time" restriction is proposed, list ratings for both normal traffic and for "One Truck" loading.

b. Identify controlling member(s). Note if member is non-redundant and identify section and/or page number of bridge analysis and rating computations for reference.

c. Provide additional information for special conditions:
   - If the capacity of the bridge is limited by the condition an element not considered in the superstructure analysis (e.g., deck, pier cap), provide additional IR/OR/SLC for that portion of the structure with justification and/or supporting computations.
   - If "One Truck at a Time" restriction is proposed, provide additional information as required by IP 4.4.3 to demonstrate that site conditions are appropriate for that kind ofrestriction.
   - If the bridge is being restricted as the result of a PUC Order, include the basis for the order (e.g., bridge analysis and rating by the Department) and a copy of the Order.

Identify the date of inspection for which the bridge conditions used in the rating were developed. If the conditions and bridge ratings have not changed since the previous posting evaluation, state it as so.

### 4.3.2 Safe Load Capacity for Bridges

The rating engineer must determine the Safe Load Capacity (SLC) for each of the bridge posting vehicles at the controlling sections/members of the bridge for the various load effects (such as moment, shear, axial compression, etc.). The SLC may be determined by engineering calculations or procedures or by engineering judgment. Bridge restrictions should be based upon the SLC.

It should be understood that the SLC is not a single number and may involve different members or sections for each of the load effects and the various live load configurations. However, for simplicity in this article, the SLC will be discussed as if it were a single value.

SLC AND OPERATING RATING: By definition (M 6.3.2), the maximum SLC for a bridge cannot exceed its Operating Rating (OR). The SLC is determined by modifying the OR by using the Safe Load Capacity Reduction Factor, which accounts for the condition of the bridge as shown in the equation below.

$$SLC = f * OR$$

Where: $f$ is the Safe Load Capacity Reduction Factor.

A Safe Load Capacity Reduction Factor less than 1.0 should be used when the substructure or superstructure has a condition rating of 4 or less, as shown in Table IP 4.3.2-1. When determining the SLC for the H, EV2, and EV3 vehicles, $f$ should not be less than 1.0.

**Table IP 4.3.2-1: Safe Load Capacity Reduction Factors**

| ADTT > 500 | Superstructure/Culvert or Substructure | | |
|---|---|---|---|
| Condition Rating | > 5 | 4 | < 3 |
| $f$ | 1.0 | 0.80 | 0.80 |

| ADTT < 500 | Superstructure/Culvert or Substructure | | |
|---|---|---|---|
| Condition Rating | ≥ 5 | 4 | ≤ 3 |
| $f$ | 1.0 | 0.90 | 0.80 |

Aside from the establishing the reductions due to the condition ratings, the owner or District may elect to establish a particular Safe Load Capacity Reduction Factor for various reasons, including: probability of overloads, degree of load path redundancy, presence of fracture-prone details, etc. For Department bridges other than those with a Condition Rating of 4 or less, the posting evaluation must include justification for a SLC less than 100% of the OR.

The rating engineer may use some judgment when choosing the SLC factor to apply to Superstructure ratings, when the Substructure rating is controlling the SLC, if the Substructure condition has been determined not to affect the load carrying capacity of the bridge. An explanation for the SLC factor used shall be provided in the load rating summary for these situations.

SLC AND INVENTORY RATING: Because the Inventory Rating (IR) of a bridge represents a live load that can safely utilize an existing structure for an indefinite period of time (by definition M 6B.2.1), the SLC of a bridge must be equal to or greater than its IR at the section being evaluated. When the SLC is based upon the structural rating, a bridge cannot be restricted under § 4902(a) of the Vehicle Code at a level less than its IR (see TITLE 67 §212 Appendix). However, the SLC and restriction may be limited by the poor condition of other components (see IP 4.4.4).

## 4.4 WEIGHT RESTRICTIONS BASED UPON THE CONDITION OF THE BRIDGE

Weight restrictions based on the condition of the bridge are subject to enforcement under § 4902(a) of the Vehicle Code.

### 4.4.1 Bridge Restrictions – Types of Weight Postings

In PA, the weight restrictions on bridges and highways are established for two basic types of postings:

- **All vehicles**
- **All vehicles except Combination vehicles**
  - Combination vehicles are more commonly known as semi-tractor trailers.
  - This posting is generally higher than postings for H20, ML80, and TK527 because of the better distribution of loads in the HS vehicle.

For bridges restricted under § 4902(a) based on the condition, the bridge weight restriction for each of these two basic types of postings is established using the following rating vehicles:

- **All vehicles**
  - Bridge posting vehicles: H20, ML80, TK527, and HS20 (see IP 3.2.2).
- **Combination vehicles:**
  - Represented by the bridge posting vehicle HS20.

### 4.4.2 Bridge Restrictions – Vehicle Weight Limit

Weight restrictions for vehicles using the bridges shall be established within the following limits:

- **Postings for all vehicles**
  - Using the controlling load case from the following bridge posting vehicles: H20, ML80, TK527, and HS20.
  - Minimum Posted Weight Limit: 3 Tons.
  - Maximum Posted Weight Limit: 36 Tons if ML80 vehicle controls.
  - Maximum Posted Weight Limit: 40 Tons if TK527 vehicle controls and ML80 rating is greater than 36 Tons.

- **Postings for Combination vehicles using the "Except Combinations" posting sign:**
  - Using the bridge posting vehicle HS20.
  - Minimum Posted Weight Limit: 10 Tons (practical minimum weight of Combination vehicle).
  - Maximum Posted Weight Limit: 40 Tons (for Rating ≥ 36 Tons).
  - For example: ML-80 Rating = 33 Tons, HS20 Rating = 37 Tons: Post "33 Tons Except Combinations 40 Tons"
  - Reason: The lighter HS20 force effect envelopes the other legal combination vehicles that go up to 40 Tons. If the posting for combinations were set at 37 Tons, it would unnecessarily restrict legal vehicles which weigh up to 40 Tons.

- **Lowest Legal posting**
  - The lowest legal posting is 3 tons (H rating). If a bridge cannot safely sustain the minimum 3 Ton posting, the bridge must be closed.

- **Other considerations for weight postings**
  - **Weight limit values on signs** The MUTCD allows the posting signs (R12-1 and R12-5A) to present the weight limit in pounds or in tons (including fractions of tons). For example, a 12 ½ ton weight limit sign is legal and enforceable. However, the Department recommends that the

weight limits be established only in integer values of tons because:

- The fractions of tons may be harder to read and the possibility that the truck driver may misread the sign is real and a threat to public safety.
- The fractional tonnage implies a level of precision in the bridge capacity determination and in the actual vehicle weight that may not exist.

– **Weight limits established by engineering judgment** When performing a bridge posting evaluation using engineering judgment (without an analysis), the precision of the bridge's live load capacity (IR, OR, and SLC) does not support a fractional tonnage value for the weight restriction signs. The use of 5-ton increments in the weight restrictions is recommended for "engineering judgment" postings, especially for higher weight limits.

– **Special vehicles** When a bridge posting evaluation may result in low weight restrictions, the need for special vehicles such as school busses and ambulances to use the bridge on a relatively low frequency should be considered. Although the weights of the actual vehicles are the best level of information for the rating, a typical school bus is approximately the same as a 17 Ton H truck and a typical box-type ambulance is about a 6 Ton H vehicle.

### 4.4.3 Bridge Restrictions – One Truck at a Time

Bridges are analyzed and rated for the general live load consideration that the traffic is not controlled and there will be more than one truck on a bridge at time. If the bridge's SLC for the multi-presence of live load case is less than Operating Rating for the bridge posting vehicles, the need for a vehicle weight restriction can be avoided or minimized if the District Traffic Engineer can demonstrate that the loading case of only a single vehicle on the bridge is appropriate for the site.

Traffic characteristics, site conditions, and the bridge configuration must be considered to warrant the use of the "One Truck" restriction. If it has been determined that the "One Truck At A Time" restriction is appropriate for the site and that the required signing will be installed, the SLC for the bridge may be analyzed and rated for each posting vehicle on the assumption that only a single vehicle will be on the bridge. The "One Truck" bridge restriction may be used alone or in conjunction with posted weight limits.

A bridge is a candidate for the application of a "One Truck At A Time" restriction if the following conditions are met:

1. The bridge roadway width is eighteen (18) feet or less (limiting it to one directional traffic), Or
2. The probability of having two fully loaded trucks on the bridge at the same time is minimal because all the following criteria are met:
   a. The total length of the structure does not exceed two hundred (200) feet unless a traffic study determines that the 200 feet limitation can be safely exceeded.
   b. The Average Daily Truck Traffic (ADTT) does not exceed 200, unless a traffic study including a truck classification count indicates that the truck traffic limit of 200 can be safely exceeded.
   c. There is adequate sight distance in both directions to provide the necessary driver reaction time.
   d. There is adequate space to stop the approaching vehicle safely.
   e. Advanced signing can be properly placed.

Information to demonstrate that the above conditions are met for the use of the "One Truck At A Time" is to be included in the posting evaluation. The District Traffic Engineer is to sign Page 4 of the Bridge Load Posting Recommendation Form (see Appendix IP 04-A) verifying that the conditions of Item 2 above are satisfied.

### 4.4.4 Bridge Restrictions Based Upon the Condition of Other Components

Typically, only the main live load-carrying superstructure members are analyzed and rated to determine the need for a bridge restriction. However, when the engineer has determined that the condition and/or structural makeup of other bridge elements, including, but not limited to, the deck, pier cap, arch spandrel walls, cross-frames and diaphragms, substructure units, etc. have compromised the bridge's safety to carry live loads, the bridge restriction should be based on the controlling component.

For bridges posted for weight restrictions based solely on the condition of components other than the main live load carrying members, the restrictions may be less than the IRs, but not greater than the SLC for the main live load carrying members. The reason for such postings must be clearly identified in the posting evaluation and BMS2 Item VP06 (see IP 3.6.1.1) The posting evaluation is to identify the IR, OR, and SLC for the main live load carrying members.

### 4.4.5 Bridge Restrictions – Exemptions for Certain Vehicles

- **Use of "Except Local Deliveries" Sign for Bridges Restricted due to Condition**
  While V.C. §4902(a) allows the owner to exempt certain vehicles (e.g., School busses, emergency vehicles…) from the posted weight restrictions, it is an unsafe practice to do this by adding the sign "Except Local Deliveries" (sign number R5-2-3) to the bridge restriction signs. The bridge cannot determine the purpose of the vehicle, only its load effect. The R5-2-3 sign is not to be used for bridges restricted due to condition.

- **Use of Special Hauling Permit to Exceed Posted Weight Limits**
  Chapter 191 of the PA Code has a provision to allow owners to issue special hauling permits to allow an individual vehicle to exceed the posted bridge weight limit. Such a permit is usually issued for a limited number of crossings after an analysis of that individual vehicle indicates it can safely use the bridge. This type of permit is used sparingly and is generally limited to critical emergency vehicles or snow removal equipment.

  For Department bridges, all vehicles, including snow plows and emergency vehicles, that exceed the bridge weight limit imposed under V.C. §4902(a) are required to have a permit issued in accordance with Chapter 191 of the PA Code. Each year before October, the District Bridge Engineer should advise the individual County managers of posted bridges in their area so they may adjust equipment assignments or obtain the necessary permits. Local bridge owners are strongly urged to proactively follow a similar policy for their bridges, especially ones that may be vital to emergency services.

### 4.4.6 Bridges on Routes Posted Due to Highway Conditions

Weight limit restrictions placed on a route solely due to highway condition (e.g., pavement condition) do not prevent all heavy vehicles from using that roadway or bridges located on it. For example, heavy trucks may receive a hauling permit or local delivery trucks exempted that allow trucks to use a bridge on a route posted for a weight limit due to the condition of the highway. Accordingly, all bridges with inadequate capacity to carry the legal loads must be posted separately and in addition to the highway posting.

### 4.4.7 Bridge Restriction Signing

#### 4.4.7.1 VEHICLE CODE REQUIREMENTS FOR SIGNING

The PA Vehicle Code § 4902(e) requires that the Commonwealth and local authorities have restriction signs erected and maintained both at the bridge site and at locations in advance of the bridge.

The Vehicle Code also requires that an advance informational sign must also be placed at the intersection nearest each end of the restricted bridge to allow drivers to avoid the restricted bridge. Additional advance informational signs at other intersections may be considered where the alternate route closest to the bridge is not the desired detour. Signing must be done in accordance with the Manual on Uniform Traffic Control Devices (MUTCD) and the Department's Handbook of Approved Signs, Publication 236.

#### 4.4.7.2 EXAMPLES OF BRIDGE RESTRICTION SIGNING

Figure IP 4.4.7-1 and Figure IP 4.4.7-2 are examples of proper signing at the bridge for restrictions established under Vehicle Code §4902(a) Restrictions Based on the Condition of the Bridge and are shown here for information only. Refer to the MUTCD and Publication 236 for further information on their configurations, use, and placement.

#### 4.4.7.3 VERIFICATION OF BRIDGE RESTRICTION SIGNING

VERIFICATION DURING INSPECTIONS: The bridge safety inspectors are to verify the existence and condition of bridge restriction signing during each inspection in BMS3 to ensure that this critical safety warning is in place. Any signing deficiencies are critical deficiencies and should be noted on the BMS3 Maintenance page and/or appropriate action to rectify taken immediately (Maintenance Priority “0”).

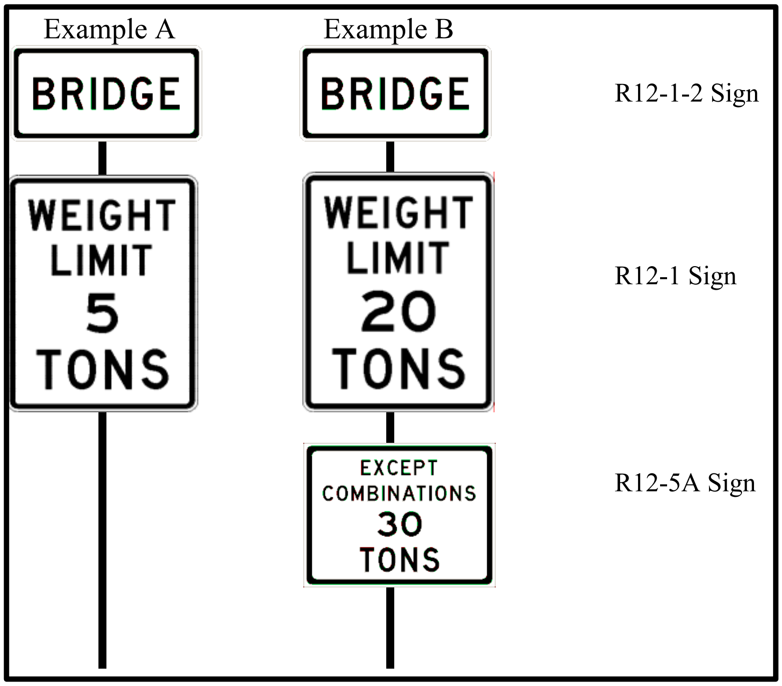


**Figure IP 4.4.7-1 Examples of Signing for § 4902(a) Restrictions with Weight Limits**

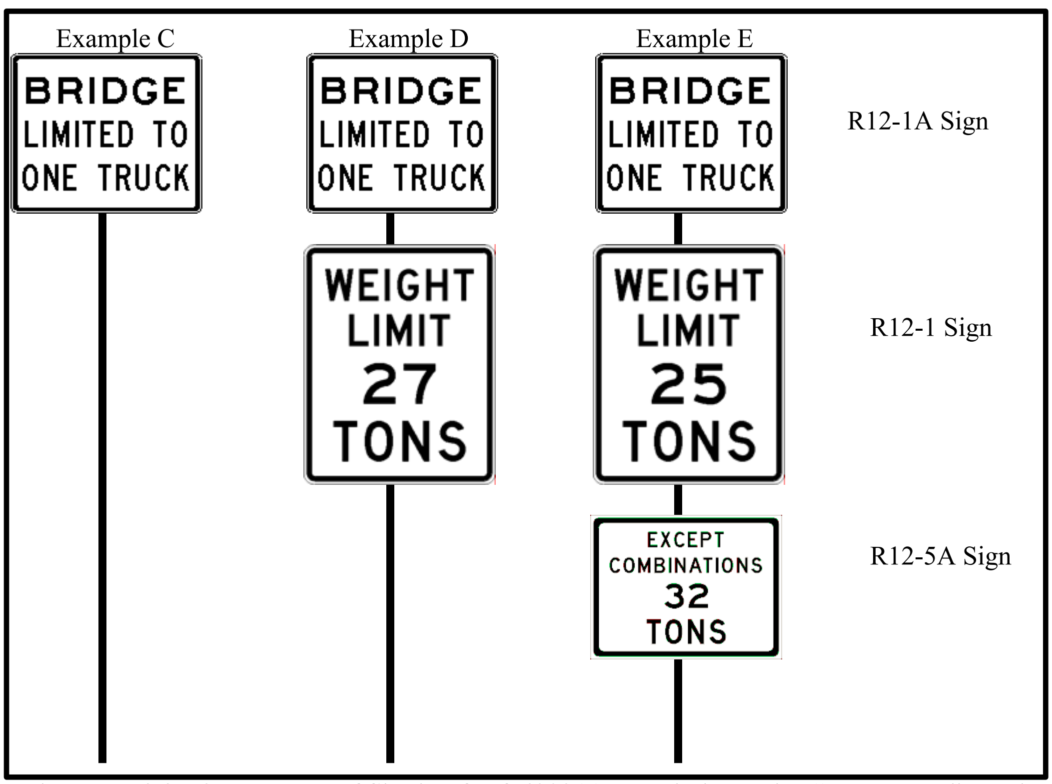


**Figure IP 4.4.7-2 Examples of Signing for § 4902(a) Restrictions with “One Truck” Posting**

## 4.5 WEIGHT RESTRICTIONS BASED UPON TRAFFIC CONDITIONS

Weight restrictions on bridges may be established based solely upon traffic conditions. Such restrictions are imposed under §4902(b) of the Vehicle Code and must not be related to the bridge condition. Warrants for such postings must be established in accordance with Chapter 212 of TITLE 67.

Allowing exemptions for traffic conditions postings to non-overload vehicles via the use of the "Except Local Deliveries" sign does not violate any structural safety concerns.

## 4.6 PROCEDURES FOR POSTING RESTRICTIONS ON DEPARTMENT BRIDGES

### 4.6.1 Applicability

These procedures apply to all first-time weight restrictions, lowering of existing weight restrictions, permanent bridge closures, "Bridge Limited to One Truck" restrictions, and removal of weight restrictions on PennDOT owned bridges equal to and greater than 8'.

For emergency situations, the District is to take actions it deems necessary for public safety. The formal posting authorization process should be completed shortly thereafter.

For bridges under PUC jurisdiction, the posting should be authorized by the Department at the appropriate authority level prior to the PUC proceedings. See the Grade Crossing Manual, Publication 371, for additional guidance on bridges under PUC jurisdiction. Publication 371 includes an example letter in Appendix A when posting a bridge under PUC jurisdiction.

### 4.6.2 Posting Approval Authority

The approval authority for bridge postings is based on the type of route carried by the bridge:

- Case 1: Interstate System Bridges – Secretary of Transportation.
- Case 2: Numbered Traffic Route (1-999) Bridges or National Highway System (NHS) Bridges – Chief Bridge Engineer.
- Case 3: Non-NHS Bridges and Other Bridges - District Executive.

For posting purposes only, bridges on ramps are to be considered part of the higher-level system to which they connect. For example, a bridge carrying an exit ramp connecting an Interstate to a local road would be treated as an Interstate structure.

### 4.6.3 Posting Approval Procedure

The District or their consultant is to inspect, analyze, and rate the bridges to develop the appropriate posting evaluation and recommendation. The District is also to review the impact of the proposed posting on emergency services and commerce.

The District is to compile the findings on the Bridge Load Posting Recommendation Form (see Appendix IP 04-A) and maintain the form in the bridge inspection file. The Bridge Load Posting Recommendation Form is to be signed by the District Bridge Engineer. The completed form shall reflect the actual inventory and operating ratings of the bridges; however, for bridges where the posting is based upon the use of a Safe Load Capacity Reduction Factor less than 1.0, comments and assumptions shall be noted on the Bridge Load Posting Recommendation Form.

**For Case 1 or 2 (see IP 4.6.2)**, the District is to submit a posting authorization request to the Assistant Chief Bridge Engineer - Inspection for review and recommendation to upper management (see Appendix IP 04-B for examples).

The posting authorization request letter must state that the bridge is to be restricted under §4902(a) of the PA Vehicle Code. Also, the reason for the restriction must be stated (see IP 4.3.1.1 for examples).

The posting authorization request letter is to include the following attachments:

- Completed Bridge Load Posting Recommendation Form.
- Load Rating Summary Form.
- Site Maps: Portion of State Map and portion of Type 10 County Map.

The Assistant Chief Bridge Engineer - Inspection is to review the District's data and forward a recommendation to the Director or the Secretary, as appropriate. The signed posting authorization letter will be sent to the District Executive with copies to the Program Center, and to the Right of Way, Utilities and Grade Crossing Division in the Bureau of Design and Delivery for bridges under PUC jurisdiction and must be saved in the bridge inspection file.

**For Case 3 (see IP 4.6.2)**, the District Executive is to review and approve the recommendation of the District Bridge Engineer. (See Appendix IP 04-B for a sample Posting Authorization Request Letter). Copies of the posting approval letter are to be sent to:

- Chief Bridge Engineer
- Assistant Chief Bridge Engineer - Inspection
- Grade Crossing Unit in the Utilities and Right-of-Way Section for bridges under PUC jurisdiction
- Director of the Center for Program Development and Management

The District is to maintain a copy of the bridge posting approval letter in the bridge file. For Districts with a District Executive who is not a Professional Engineer, the Assistant District Executive for Design (a Professional Engineer) must review and sign the posting authorization.

The Bridge Inspection Section (BIS) will perform a quality assurance review on a minimum of 50% of all bridge postings approved by the Districts. See IP 06 for additional information on quality measures for safety inspections.

### 4.6.4 Implementation of Posting

After the need for a posting is identified, the District shall complete the implementation of the posting in the field as soon as possible but in no more than 30 calendar days. The need for a posting is identified by the date the load rating set was sealed. Sign description and placement information shall be entered into BMS2 as soon as the fieldwork is complete. Coding changes within the appropriate screens must be entered at the same time.

The District is to ensure that elected and public officials, and emergency services personnel are informed of all posting actions.

Public information efforts are to be initiated by the Districts in consultation with the Central Press Office. Public announcements of postings shall provide reasons in words readily understood by the public.

### 4.6.5 Posting Documentation for Police

The District shall give full cooperation to both State and local police in the enforcement of the bridge weight restriction. Documentation stating the reason for the bridge weight restriction is to be submitted to the Pennsylvania State Police (PSP) for their use in enforcing the restrictions.

This posting documentation is to be in the form of a letter on Department letterhead from the District Executive to the PSP barracks having jurisdiction over the area where the restricted bridge is located. A new posting documentation letter is to be sent to PSP whenever bridge restrictions are initially placed, existing restrictions revised, or removed. The letter is to include the following:

- Bridge location in subject area.
  - Include: County, SR________over_______, Segment______Offset______.
- Bridge weight restriction.
  - Cite applicable section of the Vehicle Code (e.g., §4902 (a)).
  - Use format for bridge restriction as per IP 4.3.1.1.
  - Note if this is an initial restriction on the bridge or a change to an existing restriction.
- Reason for bridge weight restriction.

   - See IP 4.3.1.1 for format.
   - Emphasize public safety.
   - Do not include summary of bridge ratings or other bridge inspection related information.
- For bridges that are to be posted by a PUC order, include a statement that the PUC order is based on an engineering study performed by either the railroad or PennDOT.
- Provide the following two authorizing signatures.
   - District Bridge Engineer's signature and PE Stamp.
   - District Executive's signature.
   - If District Executive is not a Professional Engineer, then Assistant District Executive for Design signature.
- In addition to internal distribution, provide a copy of the posting letter to:
   - Assistant Chief Bridge Engineer - Inspection.
   - PUC, for bridges under PUC jurisdiction.
   - County Maintenance Manager for installation and maintenance of signage.
   - A copy of this posting letter with original signatures and PE Stamp shall be maintained in the District Bridge Inspection File.

### 4.6.6 Other Considerations for Posting Evaluations

Alternatives to postings, including, repair, temporary shoring, temporary/permanent strengthening, etc. should be considered for all bridges, especially those that seriously impact emergency services and/or commerce. For Interstate bridges, the District must provide a summary of alternatives studied with the Bridge Posting Recommendation Data Sheets.

Bridges on the Tandem Trailer Truck Network or on the access roads to that network, identified in BMS2 Items 6C16 and 6C14 as TTTN and ATTT respectively, should be given additional consideration if not on a Numbered Traffic Route.

A temporary measure placed on top of the bridge may be considered in the load rating only if the measure is sufficient enough to physically restrict traffic from reaching the deteriorated portion of the structure. If a temporary measure is in place, at a minimum the bridge posting shall be restricted (Item VP02 = R), resulting in a 12-month maximum inspection interval.

If the temporary measure is not capable of physically restricting traffic, the bridge shall be posted for the weight restriction or closed based on the results of an analysis which does not limit traffic placement on the bridge.

When bridges are to be closed, analyze and note whether pedestrian use of the bridge will be allowed. If not allowed, adequately barricade the bridge to prevent pedestrian use.

## 4.7 PROCEDURES FOR POSTING RESTRICTIONS ON LOCALLY-OWNED BRIDGES

### 4.7.1 Applicability

These procedures apply to all first-time weight restrictions, changes to existing weight restrictions, permanent bridge closures, "Bridge Limited to One Truck" restrictions, and removal of weight restrictions on locally-owned bridges.

For emergency situations, the local bridge owner or their consultant is to take actions they deem necessary for public safety. The formal posting authorization process should be completed shortly thereafter.

For bridges under PUC jurisdiction, the posting should be authorized by the local bridge owner and the District notified prior to the PUC proceedings.

### 4.7.2 Posting Approval Authority

The bridge owner has responsibility and approval authority for the posting of its bridges to ensure public safety.

The Department is to ensure that all local bridge restrictions are in accordance with Department standards to avoid improper postings. This responsibility and authority to ensure proper posting was assigned to the Department through Act 44 of 1988 (see IP 1.6.3). The District, acting on behalf of the Department, must work closely with the local bridge owner and/or their consultant to ensure proposed bridge restrictions are proper before they are put in place.

If the local bridge owner fails to implement or maintain the proper bridge restrictions, the Department is obliged to act on behalf of public safety to correct the deficiency. The District is to immediately inform the bridge owner, in writing, of the need to correct the posting deficiency immediately with a copy of the letter sent to the Chief Bridge Engineer. Facsimile notification is acceptable. The District will coordinate further needed measures with the Assistant Chief Bridge Engineer - Inspection. In the event of an emergency, the District may take necessary actions immediately. The District is to identify and invoice the local bridge owner for all non-reimbursed costs incurred by the Department.

### 4.7.3 Posting Approval Procedure

The local bridge owner or their consultant is to inspect, analyze, and rate the bridges to develop the appropriate posting evaluation and recommendation. The local bridge owner or their Consultant is also to review the impact of the proposed posting on emergency services and commerce.

The local bridge owner or their consultant is to compile its findings on the Bridge Load Posting Recommendation Form, (see Appendix IP 04-A) and maintain the signed form in the bridge inspection file. The Bridge Load Posting Recommendation Form is to be signed by a Professional Engineer.

**Local Bridge Postings are to be submitted to the District Bridge Engineer for a QC review prior to final approval by the Local Bridge Owner to:**

- Check for compliance with Federal and State Regulations and Policy.
- Facilitate reimbursement of costs for first-time postings on NBIS bridges.
- Coordinate with Districts any postings on locally-owned bridges carrying State Routes.
- Facilitate coordination with PUC for postings on bridges under their jurisdiction.

The District and local bridge owner are to maintain a copy of the bridge posting approval letter in the bridge file.

### 4.7.4 Implementation of Posting

After the need for the posting is identified, the local bridge owner shall proceed immediately with the implementation of the posting in the field within 30 calendar days. The need for a posting is identified by the date the load rating set was sealed. Sign description and placement information shall be entered into BMS2 as soon as the fieldwork is complete. Coding changes within the appropriate screens must be entered at the same time. Any required local ordinance or statute must be implemented to ensure proper enforcement and adjudication of violations.

The local bridge owner is to ensure that elected and public officials, and emergency services personnel are informed of all posting actions.

Public announcements of postings shall provide reasons in words readily understood by the public.

### 4.7.5 Posting Documentation for Police

The local bridge owner shall give full cooperation to both State and local police in the enforcement of the bridge weight restriction. Documentation stating the reason for the bridge weight restriction is to be submitted to the Pennsylvania State Police (PSP) for their use in enforcing the restrictions.

This posting documentation is to be in the form of a letter on official letterhead from the local bridge owner to the PSP barracks having jurisdiction over the area where the restricted bridge is located. A new posting documentation letter is to be sent to PSP whenever bridge restrictions are initially placed, existing restrictions revised, or removed. The letter is to include the following:

- Bridge location in subject area.
    - Include: County, SR________over_______, Segment______Offset______.
- Bridge weight restriction.
    - Cite applicable section of the Vehicle Code (e.g., §4902 (a)).
    - Use format for bridge restriction as per IP 4.3.1.1.
    - Note if this is an initial restriction on the bridge or a change to an existing restriction.
- Reason for bridge weight restriction.
    - See IP 4.3.1.1 for format.
    - Emphasize public safety.
    - Do not include summary of bridge ratings or other bridge inspection related information.
- For bridges that are to be posted by a PUC order, include a statement that the PUC order is based on an engineering study performed by either the railroad or local bridge owner.
- Provide the following two authorizing signatures.
    - Professional Engineer responsible for the restriction.
        - This may be a staff engineer from the local owner or from the Consultant.
        - PE stamp is to be affixed to the posting letter.
    - Local bridge owner's signature.
- In addition to internal distribution, provide a copy of the posting letter to:
    - Local owner bridge inspection file.
        - This copy should have the original signatures and PE stamp.
        - Maintain in file for enforcement purposes.
    - District Bridge Engineer.
    - PUC, for bridges under PUC jurisdiction.
    - Local bridge owner's maintenance forces for installation and maintenance of signage.
    - A copy of this letter with original signatures and PE Stamp shall be maintained in the local bridge owner's Inspection File.

### 4.7.6 Other Considerations for Posting Evaluations

Alternatives to postings, including repair, temporary shoring, temporary/permanent strengthening, etc., should be considered for all bridges, especially those that seriously impact emergency services and/or commerce.

Local Bridges carrying State Routes should be given additional consideration and postings coordinated with the District.

While the number of lane live loads on a bridge for analysis is normally the same as for design, using only the delineated lanes may be allowed by the Assistant Chief Bridge Engineer - Inspection to minimize or avoid a weight restriction.

When bridges are to be closed, analyze and note whether pedestrian use of the bridge will be allowed. If not allowed, adequately barricade the bridge to prevent pedestrian use.

## 4.8 VERTICAL CLEARANCE RESTRICTIONS

The minimum vertical clearance for all bridges in PA is to be recorded in the bridge inspection report and in BMS2. The minimum vertical clearance shall take into account the structure, and any signs, utilities, or other appurtenances attached to the bridge. This information is to be verified during each routine inspection of the bridge. If changes in vertical clearance are noted, the vertical clearance needs to be measured, recorded and if necessary vertical clearance posting signs need to be installed or revised. Any signing deficiencies are to be noted in the IM screen form and/or appropriate action to rectify taken immediately (Maintenance Priority "0").

Sketches of the bridge clearance envelopes are to be maintained in the bridge inspection files at the District.

The clearance envelope should show the actual vertical clearance at critical points such as edge of pavement, center of highway, outside edge of shoulder, and at changes in the bottom elevation of the bridge. Bridges such as rigid frame structures or arch structures may require special detailing.

### 4.8.1 Maximum Legal Height of Vehicles

Height restrictions for vehicles are subject to enforcement under §4922(a) of the Vehicle Code. The maximum legal height established by §4922(a) is 13'-6". Vehicles that exceed the maximum legal height are not authorized to move on highways without a written permit authorizing such movement (see IP 10) subject to §4961 of the Vehicle Code.

### 4.8.2 Posted Vertical Clearances and BMS2 Recordation

The MUTCD states "the Low Clearance (W12-2) sign shall be used to warn road users of clearances less than 12 inches above the statutory maximum vehicle height." Because the maximum legal vehicle height is 13'-6", bridges or structures over highways should be posted for a size restriction when the vertical clearance is less than 14'-6". Accordingly, all bridges with actual clearances under 14'-6" shall be posted with advance vertical clearance signs (W12-2). In addition, the W12-2A Clearance sign should be placed on the structure at the critical clearance whenever possible. If signs are not in place or are incorrect, it should be noted in the inspection report as a critical deficiency. The actual vertical clearance is to be rounded down to the next lower whole inch and recorded in BMS2 Items 6C20 and 6C21.

Inspectors should identify all possible locations where signs may be required and/or needed due to site conditions. For bridges over State routes, the inspection review engineer shall work with the District's Traffic Unit to determine if a sign is warranted. If it is determined that a sign is not warranted, a memo stating the decision should be placed in the inspection file and all pertinent inspection data, IM Screen maintenance items, comments, etc. should be revised accordingly. For bridges not over State routes, the signing for vertical clearance is the bridge owner's responsibility to determine if signs are required and coordinate with the roadway owner as necessary. The local consultant or in-house staff has the responsibility to make a proper recommendation.

The posted vertical clearance generally includes a 3" buffer from the minimum measured vertical clearance to allow safe passage of vehicles beneath the structure accounting for some vertical curvature in the roadway profile, frost action, and the possibility that the vehicle is bouncing. Additional buffer for the vertical clearance may be warranted when sag or crest vertical curves at the bridge reduce the effective opening for longer vehicles.

For overhead bridges (such as arches) where the vertical clearances vary significantly across a cross section of the highway, posting the bridge for vertical clearance at more than one point (i.e., at centerline and at edges of pavement) may be warranted.

Overhead bridges with posted vertical clearances shall have an advance sign (W12-2) placed at the nearest intersection or a wide point in the road to facilitate a vehicle detour or turnaround.

However, because non-route specific annual permits for loads up to 14'-2" height are legally allowed to travel on Pennsylvania roads without a review of the route, bridges under that vertical clearance may be struck by a driver obeying an issued permit. Advance signing at the nearest intersection or wide point in the road is also warranted for this case.

Because APRAS includes the 3" buffer for vehicles in excess of 13'-6" internally as part of its review process, the measured vertical clearances are to be entered in BMS2 Items 6C20 - 6C23.

Vertical clearance postings do not require approval beyond the District Bridge Engineer. The vertical clearance posting is to be noted in the inspection report, recorded in BMS2 and implemented by the District.

PennDOT Publication 236M, 'Handbook of Approved Signs', indicates justification and placement for the W12-2 and W12-2A signs.

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## 5.1 GENERAL

Pennsylvania's Bridge Management System 2 (BMS2) is a powerful management tool that records and stores bridge inventory and inspection data for Pennsylvania's bridges and enables Department managers to make key decisions concerning bridge inspection, maintenance, preservation, rehabilitation and replacement. The integrity of the BMS2 is directly related to the quality, accuracy and uniformity of the bridge inventory and condition data obtained through field inspections. BMS2 helps ensure public safety and allows PA to comply with the federally mandated National Bridge Inspection Program and National Tunnel Inspection Program.

BMS2 is composed of inventory and inspection items collected for various structure types including bridges, tunnels, culverts, sign structures, retaining walls and high mast light poles. These data items are explained in detail in PennDOT's BMS2 Coding Manual, Publication 100A. The system also has the ability to produce a wide range of reports including standard monthly statistics reports, standard menu driven reports, and customized user-generated reports.

BMS2 integrates and exchanges key data elements with RMS, MPMS and SAP/PM. BMS2 also stores bridge data that supports APRAS.

BMS2 supports the federally mandated National Bridge Inspection Program and National Tunnel Inspection Program which enables Pennsylvania to receive its Federal allocation of funds.

## 5.2 BMS2 DATA

Pennsylvania's BMS2 contains data information separated onto various screens. The screens are separated into inventory and inspection screens. Inventory screens generally contain information about the components that make up the bridge (i.e. type, size, location). The Inspection screens document the findings a bridge inspector would report while inspecting the bridge. All data required by the Federal Highway Administration is included in these screens plus additional data deemed necessary by the Department. Publication 100A provides detailed descriptions and coding for each data item.

Data that resides in BMS2 can come from any of three sources: direct data entry via keyboard or BMS3, such as bridge condition ratings; data that is generated through system calculations, such as maintenance deficiency rating; and finally, data that is imported from other Department Management Systems, such as average daily traffic.

BMS2 also exports bridge data to other Department Management Systems. The exchange of data between Department systems occurs via automatic batch processes at either daily or weekly intervals depending on data type.

BMS2 currently integrates with the Multimodal Project Management System (MPMS), the Roadway Management System (RMS), the Automated Permit Rating and Analysis System (APRAS), and the Geographic Information System (GIS) as indicated in Table IP 5.2-1.

**Table IP 5.2-1 Integration of Data in BMS2**

| System | Data to BMS2 | Data from BMS2 |
|---|---|---|
| RMS | Traffic Volumes, Network Data, and Functional Classification | Bridge Location and Select Bridge Data |
| MPMS | Program and Budget Data | Select Bridge Data |
| APRAS | None | Bridge Clearance and Load Data |
| GIS | None | Select Bridge Data |
| SAP/PM | Bridge Maintenance Data | Proposed Maintenance |

## 5.3 BMS2 REPORTING

### 5.3.1 Reporting Within BMS2

A wide range of reporting capabilities has been included in BMS2 in order to access and fully utilize the extensive amount of data it contains. BMS2 has the ability to produce standard, menu driven reports; customized, user-generated reports; and automatic monthly bridge statistics reports.

Automatic monthly bridge statistics reports serve to report, document, and monitor the number, condition, type, and ownership of all bridges in BMS2. These reports also serve as a basis to track trends or patterns that may be developing over time. For example, a comparison of monthly reports could be used to determine whether bridge maintenance needs have increased or decreased over the last five years on a Statewide basis or within specific areas of the State. Department managers would then have a basis to consider changes to bridge maintenance program funding levels.

### 5.3.2 Crystal Reports

Crystal Reports is a report writer program that provides a user-friendly Graphical User Interface (GUI) environment to enable users to utilize previously defined reports or to create new reports. Frequently used report definitions can be "published" to a PennDOT web server where the data results can be refreshed and viewed by any authorized BMS2 user without having Crystal Reports software installed on their local workstation. To create new reports, Crystal Reports software must be on the user's desktop. The PennDOT web server can only be accessed by users with the Internet Explorer browser connected to the PennDOT network. BMS2 Crystal Reports accesses data from the DB2 version of the BMS2 database also used for APRAS.

Important features of Crystal Reports and the web server include:

1. Reports can be represented as charts or graphs as well as textual data.
2. Reports are rendered in color and can include graphical elements such as the PennDOT logo.
3. The web server interface provides a search capability and a hierarchical tree structure that allows you to jump directly to a particular report section or page.
4. In addition to hard copy prints, reports can be partially or fully exported to a number of standard desktop software formats such as Adobe, Excel, Word and plain text.
5. Reports can be "parameterized," allowing the user to easily filter the report to narrow the results. For example, reports can be filtered by District so that only the information for the selected district is contained within the report. By providing the capability to enter parameters, a single report can serve the needs of all District users.

The District Bridge Engineers are responsible to see that bridge inspection information is not inadvertently released through this reporting tool. Access to BMS2 data using Crystal Reports and the PennDOT web server is only available to PennDOT users and not outside agencies.

The Bridge Inspection Section (BIS) is responsible for authorizing user access to BMS2 and BMS2 data through the PennDOT web server for Department users. Users requesting access to BMS2 and BMS2 data through the PennDOT web server must follow the instruction in Publication 100A and submit to the BMS2 manager in the BIS. A user may have READER or PUBLISHER access. The form is available in Publication 100A.

1. A READER is a user who:
    a. Can access published reports on the PennDOT web server through their Internet Explorer browser and can adjust parameters to customize the report and specify output format (e.g., hardcopy print, Excel Worksheet)
    b. Must have access to PennDOT network
    c. Needs no Crystal Reports software on desktop to run published reports.
    d. Needs little or no formal training.

2. A PUBLISHER is a user who has the same capabilities as a READER and:
    a. Can add reports to the PennDOT web server for other users to run.
    b. Must have a copy of the Crystal Reports Developer software on desktop.
    c. Must have more advanced training in Crystal Reports.

The initial folder structure on the PennDOT web server provides a repository for Statewide reports and locations for district-specific reports. An individual user can run queries using the report definitions in the Statewide or District folders but cannot modify them without first copying to their folder.

## 5.4 DATA ENTRY & MAINTENANCE

The following organizations have the responsibility to maintain current information in BMS2 for the bridges in their jurisdiction:

Districts – All bridges on or over State Routes plus all bridges on or over local routes owned by counties, municipalities or other parties.

DCNR – All bridges owned by the Department of Conservation and Natural Resources.

PTC – All bridges on or over the Pennsylvania Turnpike.

BIS – All other bridges in the State owned by other State or federal agencies.

The table in Appendix L of Publication 100A provides the BMS2/APRAS RACF Security Authorization Profile for all BMS2 users. It indicates the appropriate BMS2 and APRAS capabilities for the full range of system users. Authorization for functions shown in parentheses (   ) will be granted if requested and if job duties include those functions. For persons or capabilities not shown on the Security Authorization Profile, additional justification must be submitted.

All inspectors with BMS access are responsible for data entry pertaining to their qualifications, current contact information, and reporting of any adverse actions taken against them or their PE license. All inspectors with BMS access shall self-certify with their credentials related to bridge inspection on the User Preferences Screen in BMS2. See Publication 100A, Section 2.17 for additional information. Inspectors must also report to the Bridge Inspection Section when they change employers as their account should remain with them throughout their career.

### 5.4.1 BMS2 Access

BMS2 access will be controlled by the Department's ECMS and the table in the preceding section concerning Authorized BMS2/APRAS Users. A "Request for BMS2/APRAS Access" form must be completed and sent to the BIS, Attention: BMS2 Manager. This form can be found in Publication 100A. The BMS2 Manager will approve correctly completed forms and forward them to the ECMS for entry. This process is only applicable to Department employees.

Access for non-Department users is controlled by the administration of their organization. Please refer to Publication 100A for more information.

### 5.4.2 BMS2 Deletions

Bridge records should not be deleted from BMS2. The BIS will retain records of bridges removed from inventory on ECS.

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## 6.1 QUALITY MEASURES

The bridge inspection process is the foundation of the entire bridge management operation and the bridge management system. Information obtained during the inspection will be used for determining needed maintenance and repairs, for prioritizing rehabilitations and replacements, for allocating resources, and for evaluating and improving design for new bridges. The accuracy and consistency of the inspection and documentation is vital because not only does it impact programming and funding appropriations but also it affects public safety. Therefore, the Department addresses this need with extensive formalized quality control and quality assurance procedures. Reference MBE Section 1.4 for additional guidance.

## 6.2 QUALITY CONTROL

Quality Control (QC) is the enforcement, by a supervisor, of procedures that are intended to maintain the quality of a product or service at or above a specified level. QC of the inspection of highway bridges is a daily operational function performed in each organization performing the safety inspections, including consultants, owners, and District Bridge Units. A set of effective QC procedures will provide for uniformity of inspection and recording methods and will ensure quality reports. To ensure Statewide uniformity and consistency the Department shall provide basic inspection training and mandatory biannual refresher courses (every 5 years for tunnel refreshers) (See IP 07, Training and Certification Program).

Each bridge safety inspection organization (e.g. Department District Bridge Units, engineering consultant firms, or bridge owner's staff) is to have internal quality control procedures in place to assure that the public safety is maintained on the bridge and that the inspections are performed in accordance with NBIS/NTIS and Department standards. An effective quality control program for safety inspection shall address the following areas:

- Organization and Staffing.
- Review of Field Inspections.
- Office File Review.
- Bridge Maintenance/Rehabilitation/Replacement Needs.
- Annual Meeting with Bridge Inspection Staff.

A record of QC efforts (e.g. a QC Logbook) shall be maintained by the inspection organization.

### 6.2.1 Inspection Organization and Staffing

An effective QC program begins with assuring that an adequate, qualified and properly equipped staff is in place to address the primary functions of a bridge inspection program:

- Field Inspections and Final Report
- Bridge Analysis, Rating and Posting Evaluations
- Maintenance and Improvement Needs

Responsibilities of PennDOT personnel including the District Bridge Engineer and the Assistant District Bridge Engineer (ADBE) – Inspection are outlined within the following sections. Each district may distribute the work differently to achieve the same desired results. Therefore, when a responsibility is assigned to one of these individuals within this chapter, it indicates the ultimate responsibility resides with that individual, however, the task may be assigned to a delegate.

The District Bridge Engineer is to maintain a roster and organization chart of the staff addressing these primary functions. The District Bridge Engineer is to ensure that the staff meets NBIS/NTIS and Department requirements for certification, training, and experience. The staffing complement must be sufficient and properly equipped to ensure that inspections are performed in a timely manner and in compliance with NBIS/NTIS and Department requirements.

The District Bridge Engineer is to ensure that engineering consultants, bridge owners and Districts have the proper staff for the bridges assigned to them, see IP 2.1.

### 6.2.2 QC Review of Field Inspections and Final Reports

Quality Control reviews are to be performed by personnel other than the individual who completed the original report or calculations. The required and recommended procedures and responsibilities for Quality Control review of field inspections and acceptance of final reports are detailed below.

**Bridge Inspection Team Leader**

REQUIRED ACTIONS

- Upon completion of each field inspection, the Team Leader is to review the data and notes entered into BMS3, and then upload the data to BMS2 where it will be stored in "Submitted" status. As part of this review, team leaders are to ensure that all items listed in the Inspection Report Quality Control Verification Checklist in Appendix IP 06-A are addressed prior to submittal of the report.
  - For consultants, this will include an intermediate step for independent office review by the consultant's Project Manager or other inspection team leader.
- Upload inspection data to BMS2 and provide written notification to the owner within 30 days (90 days for tunnels).
  - For Department owned bridges, direct the notification to the Department's ADBE – Inspection.
  - For non-Department owned bridges, direct the notification to the owner of record and copy the Department's ADBE - Inspection.
- For Department personnel, upon the discovery of any critical or high priority structural maintenance activities, immediately contact the ADBE – Inspection.
- For consultant personnel, upon the discovery of any critical or high priority structural maintenance activities, immediately inform the contract Project Manager. The consultant shall contact the bridge owner within 24 hours as required by the standard scope of work for bridge safety inspections using the procedures described in the company Quality Control Plan.
  - Contact with the owner shall be made by a Professional Engineer.
  - For Department owned and non-Department owned bridges, inform the Department's ADBE – Inspection.
- Make special note of the existence of any condition ratings of 3 or less for Deck, Superstructure, Substructure, Channel and Culvert (not applicable for tunnels). For consultant personnel, inform ADBE – Inspection and/or bridge owner of the existence of these conditions by written notification (time frame for notification not to exceed 2 weeks). An expedited inspection report is preferred. Copy the ADBE – Inspection on all notifications concerning non-Department owned bridges. Bridges (or structures) with these condition ratings must be reviewed by the District Bridge Engineer.
- Make special note of any elements or systems with condition state of 4.

**Assistant District Bridge Engineer (ADBE) - Inspection (or delegate)**

REQUIRED ACTIONS

- Review BMS2 in "Submitted" status for accuracy and completeness.
  - All bridges in good condition, i.e., deck, superstructure, substructure, and culvert condition rating of 5 or greater, must be reviewed and accepted by a Certified Bridge Safety Inspector. The CBSI must be independent from the Team Leader and Team Member(s) of the inspection.
  - All bridges in poor condition, i.e., deck, superstructure, substructure, or culvert condition rating of 4 or less, must be reviewed by a CBSI that is a Professional Engineer before the report can be "Accepted" in BMS2. The Professional Engineer reviewer must be independent from the individual responsible for the safety inspection report and be Department staff.
- When review is complete, approve the report by placing it into "Approved" status in BMS2.
- Upon receipt of reports of high priority or critical deficiencies, immediately notify the District Bridge Engineer.

RECOMMENDED ACTIONS

- Once every three months, select a few bridges that were inspected during the previous quarter, and visit sites.
  - Reviews should focus on bridges in poor condition and bridges with high priority maintenance needs.
  - Using BMS2 Coding Manual, rate the bridge for condition and appraisal ratings.
  - Compare results with previous inspection reports.
  - Review field observations with the Bridge Inspection Team Leaders.
  - Enter comments and site locations in a QC log book.

- Review adequacy of reference materials and inspection tools with each inspection team.

**District Bridge Engineer**
REQUIRED ACTIONS

- Review and approve all critical and high priority maintenance needs, component condition rating (i.e., deck, superstructure, substructure or culvert) of 3 or less, and all elements/systems in condition state 4 before final acceptance of the inspection report in BMS2. This includes ensuring applicable fields on the Proposed Maintenance screen in BMS2 have been completed (see Section IP 2.14.4).
- Ensure bridges identified with a condition rating of a 3 or less have appropriately coded critical or high priority maintenance items to improve the condition rating of the structure.
- Develop Plan of Action for each critical and high priority maintenance item. (See IP 6.2.4.1)

RECOMMENDED ACTIONS

- Conduct annual meeting with all bridge inspection staff, including consulting firms. (See IP 6.2.5)

**Assistant District Executive (ADE) - Design**
RECOMMENDED ACTIONS

- Once or twice (preferred) per year conduct meeting with ADE - Maintenance, District Bridge Engineer, ADBE – Inspection and Bridge Maintenance Coordinator
- Review recommendations for maintenance and improvements and completion of high priority and critical needs (see IP 6.2.4.2).

### 6.2.3 QC Review of Office File

The bridge files in the office should be reviewed to ensure that the information needed for bridge inspection is readily available. All documentation of inventory and inspection information should be kept in an orderly and retrievable manner. A sample plan for the Districts is suggested below. This can be modified for other organizations:

**ADBE – Inspection (or delegate)**
RECOMMENDED ACTIONS

- Review the files for approximately 10% of the bridges inspected the previous month for completeness and accuracy.
- Every three months, review posted bridge lists and review the files for 10% of these bridges, which were inspected within the previous quarter to see that the file documentation is sufficient and agrees with the posting, and the rating is current with latest inspection findings.
- Review 25% of NSTM bridge files to ensure information needed for an NSTM inspection is available one month before the upcoming inspection.

**District Bridge Engineer (or delegate)**
RECOMMENDED ACTIONS

- Annually review the list of posted bridges to determine repair or replacement options
- Annually review Fatigue and Fracture Inspection Plan for District's NSTM Bridges to develop rehabilitation/replacement strategies.

### 6.2.4 QC of Bridge Maintenance/Rehabilitation/Replacement Needs

#### 6.2.4.1 QC OF CRITICAL OR HIGH PRIORITY MAINTENANCE NEEDS

**District Bridge Engineer (or delegate) Responsibilities**

- Respond immediately to all reported Priority 0 or 1 maintenance needs by first ensuring that public safety is protected and then by developing a Plan of Action to correct or repair the critical deficiency (see IP 2.14).

#### 6.2.4.2 QC OF SCHEDULED BRIDGE MAINTENANCE (PRIORITY 2 THRU 5), REHABILITATION AND REPLACEMENT NEEDS

The determination of bridge needs (maintenance, rehabilitation, and replacement) by the inspection organization should be reviewed annually. A sample plan for the Districts is suggested below. This can be modified for other organizations:

ADE-Design:

- Review with the District Bridge Engineer the procedures to be used in the event of a bridge emergency for reporting and coordinating repairs.
- Review with the District Bridge Engineer the procedures for selection of candidates for bridge maintenance program and rehabilitation/replacement programs. Review accomplishments and identify concerns.
- Review how large differences in bridge inspection condition/appraisal ratings or posting recommendations from the previous inspection are handled by the ADBE - Inspection.
- Review completion of high priority or critical maintenance needs annually.

### 6.2.5 Annual Meeting with Bridge Inspection Staff

An annual meeting of field inspection staff with the District Bridge Engineer, ADBE – Inspection, and Ratings Engineer is recommended to ensure that the entire team is aware of the latest developments in safety inspection. Additional meetings should be considered if significant issues or concerns arise. The following suggestion is made for the Districts and may be modified by other organizations.

District Bridge Engineer

- Once a year review all Q/C comments and observations with entire bridge inspection staff including local inspection coordinator.
- This review may be scheduled following a session of the Refresher Course for Bridge Safety Inspectors that one or more of the inspectors have attended to apprise remaining staff of the latest developments and the Department's current emphasis.
- This review should be separate from the Statewide QA program's district close-out meeting.

### 6.2.6 Samples of Good Inspection Practices

Inspection teams shall review inspection files prior to field work and take copies of inspection files to the field. Team shall complete all condition and appraisal ratings and review other items for correctness if directed by the supervisor on the BMS3 Report and D-491 or BMS2 printout.

Bridge inspection teams should be rotated so that a team does not inspect the same bridge on consecutive routine bridge inspections. Consecutive inspections by the same team could lead to complacency because of too much familiarity with the structure.

The Bridge Inspector's Supervisor shall review each report for completeness and uniformity. Load ratings shall be computed for any bridge which has changed due to section loss or recent repair, etc. or whose ratings were never computed.

## 6.3 PENNSYLVANIA STATEWIDE BRIDGE AND TUNNEL SAFETY INSPECTION QUALITY ASSURANCE PROGRAMS

Quality assurance (QA) is the verification or measurement of the level of quality of a sample product or service. The Statewide Bridge Safety Inspection QA Program is administered by the Bridge Inspection Section (BIS) in the Bureau of Bridge in conjunction with its Bridge Safety Inspection QA Consultant. The Statewide Tunnel Safety Inspection QA Program is administered solely by the BIS.

The purpose of the Statewide Bridge and Tunnel Safety Inspection QA Programs is to measure the accuracy and consistency of Pennsylvania's bridge and tunnel safety inspections. The findings from these programs are used to enhance or emphasize training needs in the State's bridge inspection training courses and to address any Statewide bridge or tunnel inspection anomalies.

The Department's Statewide Bridge and Tunnel Safety Inspection Quality Assurance Programs are outlined in Publication 240, Bridge and Tunnel Safety Inspection Quality Assurance Manual.

### 6.3.1 Statewide Bridge Safety Inspection Quality Assurance Program

The Statewide Bridge Safety Inspection Quality Assurance Program consists of independently re-inspecting a selection of NBIS bridges in each of the 11 Districts and some owned by other agencies (including PTC, DCNR, DRPA, DJBTC) on a cycle basis (1 cycle is the equivalent of a year).

Results of the QA inspections in the form of a draft District Summary Report are reviewed with the inspectors in each District or other agency during a Close-Out Meeting. The Close-Out Meeting is an important part of the QA process because it encourages communication between the QA reviewers and the individual inspectors. Findings from the QA inspections are discussed. The results of these meetings are used to emphasize training requirements, improve inspection techniques, improve the quality of the data in BMS2, and initiate needed changes to inspection and coding manuals and Department rating programs.

If the results of the QA inspections discover a common issue which requires clarification to all bridge inspectors Statewide, the issue will be addressed via a link on the BMS2 homepage. Inspectors shall check this link frequently for the latest information.

The results of the QA Close-Out Meeting are incorporated into the report and the final District Summary Report is distributed to the District or other owner.

When all District Summary Reports are finalized and distributed to the respective Districts, the Statewide Cycle Summary Report is compiled, and a copy is distributed to each District.

The Statewide Cycle Summary Report is a compilation of all the Districts' QA results. This compilation gives an indication of Statewide trends in bridge inspection. Any consistent problems are identified and corrected through the following means:

- Revisions to Procedures and Manuals
- Bridge Inspection Basic and Refresher Training Courses
- Other Bridge Inspection Related Courses

### 6.3.2 Statewide Tunnel Safety Inspection Quality Assurance Program

The Statewide Tunnel Safety Inspection Quality Assurance Program has the same goals as the Bridge QA Program but has different processes and methods. The Tunnel QA Program is outlined in detail in Publication 240, Part B – Tunnel Quality Assurance.

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## 7.1 GENERAL

The Department provides several bridge and tunnel inspection training courses for its safety inspectors and supervisors. These courses are briefly discussed in the following section. For additional information including prerequisites and learning outcomes, check the Training Calendar in ECMS (at https://www.ecms.penndot.pa.gov/ECMS/).

The District Bridge Engineer is responsible to see that all Department personnel performing safety inspections are trained and qualified at the appropriate level. The District Bridge Engineer is to ensure that all bridge owner or consultant staffs performing NBIS/NTIS safety inspections in their Districts are qualified at the appropriate level. The Assistant Chief Bridge Engineer - Inspection is responsible to ensure those consultants and other State or federal agency personnel performing NBIS/NTIS safety inspections on Statewide open-end agreements or other agency bridges are qualified at the appropriate level.

The Department does not typically provide any training classes for safety inspection of tunnels. The tunnel inspection training course, NHI 130110 is available through FHWA.

## 7.2 REQUIRED BRIDGE SAFETY INSPECTION COURSES

### 7.2.1 Safety Inspection of In-Service Bridges (NHI Course #130055)

PennDOT offers the NHI Safety Inspection of In-Service Bridges (Basic Course) multiple times a year and the course is intended to provide first-time certification of bridge safety inspectors and to impart the basic knowledge and skills necessary to accurately report on bridges for Statewide uniformity. The course includes ten days of comprehensive training based on the FHWA training guide to satisfy the requirements outlined in the NBIS. Completion of this course fulfills the training requirements of the NBIS (23 CFR 650.309) for an FHWA-approved comprehensive course. Inspectors who register for the basic course offered through Pennsylvania are automatically enrolled in an additional four days of training in the PennDOT Bridge Inspection Practices and Procedures course which covers PennDOT specific inspection practices and procedures. This additional training is required to inspect bridges within Pennsylvania.

### 7.2.2 Bridge Inspection Refresher Training (NHI Course #130053)

PennDOT offers NHI's Bridge Inspection Refresher Training course. The Bridge Inspection Refresher Training course is a three-day training which is intended to update the certification for a bridge safety inspector. This training provides a review of inspection and reporting procedures as well as information on special topics. The completion of this course fulfills the training requirements of the NBIS (23 CFR 650.30) for an FHWA-approved refresher training course. Inspectors are required to complete this 18-hours of training over each 60-month period. Refer to Section IP 7.4.1.2 for guidance on the 60-month window.

### 7.2.3 PennDOT Bridge Inspection Practices and Procedures

PennDOT develops and offers the PennDOT Bridge Inspection Practices and Procedures course. This course is four-days and specifically focuses on policy and procedures for inspecting bridges in Pennsylvania, above and beyond the minimum requirements set forth by the NBIS. PennDOT collects nearly 1,000 data points per structure compared to FHWA's SNBI field list of just over 100. In addition to the extra coding field review, the course covers inspection intervals and a basic overview of PennDOT's Bridge Management System, which houses the data collected by inspectors. Prior to enrolling in this class, bridge inspectors must have successfully completed a comprehensive bridge inspection training course (#130055 or #130056). Completion of this course along with the comprehensive course is needed to fully satisfy PennDOT's training requirements for a Certified Bridge Safety Inspector (CBSI).

### 7.2.4 Safety Inspection of In-Service Bridges for Professional Engineers (NHI Course #130056)

This course is specifically designed for prospective bridge inspectors who possess a valid and current Professional Engineering license. This course was adapted from the Basic Course (See Section 7.2.1) and has been reduced to five-days. Unlike the Basic Course where the Practice and Procedures is already incorporated into the 14-day schedule, this course does not include that information. Students attending this course must also schedule to complete an additional four days of training in the PennDOT Bridge Inspection Practices and Procedures course which covers PennDOT specific practices and procedures. Completion of this course fulfills the training requirements of the NBIS (23 CFR 650.309) for an FHWA-approved comprehensive course.

#### 7.2.5 PennDOT Annual Inspection Workshop

With the transition to the use of NHI's Bridge Inspection Refresher Training course to provide FHWA-approved refresher training that meets the requirements set forth in the 2022 NBIS, PennDOT has elected to develop and require bridge inspectors to attend an annual virtual workshop that allows PennDOT to convey hot topics to all inspectors on a more frequent basis than every 60-months. PennDOT will not be submitting this training for FHWA approval. Therefore it will not be counted towards the 18-hours of required refresher training in a 60-month window. This training course will be offered on-demand for inspectors, and they will be required to complete the modules and subsequent exams within the calendar year they are published. The course is estimated to be 4 to 8 hours in length and be updated annually. Similar to the previously mentioned training courses, this class will be available through the ECMS training page and will typically be available from January through April each calendar year.

## 7.3 OTHER TRAINING COURSES OFFERED BY PENNDOT

#### 7.3.1 Non-Redundant Steel Tension Member Inspection Techniques for Steel Bridges (NHI #130078)

The Non-Redundant Steel Tension Member Inspection Techniques for Steel Bridges is an NHI course (#130078). The Department offers this course periodically as needed for commonwealth employees and consultants. The training covers the concept of a Non-Redundant Steel Tension Member (NSTM), NSTM identification, failure mechanics, fatigue and an overview of Non-Destructive Testing (NDT) methods. The course also includes a hands-on workshop for popular nondestructive evaluation equipment and a case study of an inspection plan for a bridge containing NSTM's. Team leaders completing inspections on bridges with NSTM's must have completed this course. The course is approximately four (4) days in length.

#### 7.3.2 Bridge Scour Evaluation

The Bridge Scour Evaluation course is a course developed by PennDOT and is intended to provide additional instruction on the evaluation and coding of scour for PA bridges. The course objective is to improve the ability of bridge inspectors and supervisors to better recognize, understand, evaluate and report actual and potential scour conditions base on PennDOT procedures outlined in Pub 100A, BMS2 Coding Manual. This training is not mandatory; however it is strongly encouraged for team leaders. The training will be offered on an as-needed basis. The course is approximately two (2) days in length.

#### 7.3.3 Load Rating Analysis of Highway Bridges

The Load Rating Analysis of Highway Bridges course presents details on when and how to perform a load analysis on highway bridges to conform to Department requirements and to ensure safe load posting. It also instructs the participants where to find information required to perform a rating analysis. This course is in the process of being reviewed and this section will be updated upon completion of the course documents.

#### 7.3.4 Other Bridge Inspection Related Courses

Other bridge inspection related training courses may be offered from time to time as they become available or as warranted. Refer to the ECMS Training Website for a full listing.

## 7.4 BRIDGE SAFETY TRAINING REQUIREMENTS FOR BRIDGE INSPECTORS IN PENNSYLVANIA

#### 7.4.1 Bridge Inspectors

##### 7.4.1.1 Group 1 Requirements – Complete Initial Bridge Inspection Training

At a minimum, all bridge inspectors must complete comprehensive bridge inspection training. There are two options to meet these requirements.

Option 1 – Attend an NHI Basic Course (#130055) offered by PennDOT; this will include the 10-day basic course in addition to PennDOT's Practices and Procedures course (a total of 14-days).

Option 2 – Attend an NHI Basic Course for PE's (#130056) hosted by PennDOT or another vendor **OR** attend an NHI Basic Course (#130055) by someone other than PennDOT, **AND** attend PennDOT's Practices and Procedures Course.

#### 7.4.1.2 Group 2 Requirements – Complete Bridge Inspection Refresher Training

Successfully complete PennDOT's Annual Bridge Inspection Workshop and associated exam,

**AND**

Attend an NHI Bridge Inspection Refresher Training Course every five (5) years to satisfy the 18-hour in 60-month training requirement from the NBIS.

The first 60-month window for a bridge inspector to complete the 18-hours of required training starts at whichever date is of the latter using the following criteria:

a. The month of the last day of the bridge inspector's basic course (as described in Section 7.4.1.1 OR
b. The month of last day of the bridge inspection refresher training course taken after June 6, 2022

Subsequent 60-month windows begin the month following the completion of the required refresher training. For example, if a student completes the required refresher training in October 2024, the next refresher training window ends in October 2029.

There will no-longer be a grace period for inspectors to wait for training courses to be held in their local area. The 1st day of the 61st month, the inspector will no longer be qualified to inspect bridges. To regain their status within 24-months of expiration, an inspector must pass an approved refresher training and complete the annual PennDOT workshop for the given year. If the inspector wishes to recertify after 84 months (60 initial +24 grace), they must complete the NHI Basic Course or equivalent (#130055 or #130056).

### 7.4.2 Team Leaders

Bridge inspection team leaders must meet the requirements of 23 CFR 650.309(b) and the minimum requirements for a bridge inspector outlined in IP Section 7.4.1. Team leaders who have not completed a comprehensive bridge inspection training course and were previously grandfathered as of January 2005 will no longer be qualified as a team leader as of June 6, 2024. These individuals must complete a comprehensive bridge inspection training course (NHI #130055 or #130056) to continue to function as a Team Leader (i.e. complete a comprehensive course).

Team Leaders completing inspections of portions or components of bridges containing NSTMs shall have successfully completed the NSTM Training (#130078) prior to the NSTM inspection. Note the course certificate may have a different name from past offerings, but it must show the course number of #130078 to be considered for meeting this requirement.

### 7.4.3 Divers

Underwater Divers completing bridge inspections within Pennsylvania must comply with Groups 1 and 2 Requirements as covered in Sections 7.4.1.1 and 7.4.1.2 in addition to completing NHI's #130091 – Underwater Bridge Repair, Rehabilitation and Countermeasures. Divers who were completing underwater bridge inspections prior to June 6, 2022 are exempt from the requirement to have successfully completed the NHI #130091 course.

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## 8.1 PURPOSE OF INSPECTION RECORDS AND FILES

Owners are to maintain complete, accurate, and up-to-date records for each of their bridges. These records are needed to:

- Establish an inventory of infrastructure assets
- Document the condition and functionality of infrastructure, including the need and justification for bridge restrictions, for public safety
- Identify improvement and maintenance needs for planning and programming
- Document improvements and maintenance repairs performed
- Meet documentation requirements for work performed using Federal and State funding
- Provide available information in a timely manner for safety inspections

The bridge inspection file is an integral part of an effective bridge inspection and management system. The information in the bridge inspection file is kept current through bridge inspections scheduled at regular intervals. As bridge inspection files are updated, the existing information is archived and retained to establish a history for each bridge.

The inventory data is required by the NBIS under Section §650.315 and by the NTIS under Section §650.515 which states the minimum interval after the inspection for inventory data to be submitted to the State or Federal agency. PennDOT requires that BMS3 reports be submitted to BMS2 for the Department's approval within 10 days of the completion of the field inspection for bridges and within 30 days for tunnels.

## 8.2 INSPECTION ORGANIZATION UNIT FILE

The Districts and individual bridge owners are to maintain a general file of their organization for bridge safety inspection. This general organization file should contain:

- List of bridges and structures*
- List of posted bridges* with date of most recent signing verification
- List of NSTM bridges*
- List of bridges with special features and/or conditions that necessitate special or more frequent inspections*
- List of bridges that require underwater inspection*
- List of bridges to be inspected during/after high water events*
- Contact list for key staff during bridge emergencies
- Inspection organization
    - Organization Chart
    - Staffing with inspection certification credentials indicated
    - Internal inspection QC plan
    - List of inspection equipment
    - Availability of bridge design and inspection reference material
    - Results of QA reviews

* May be generated from BMS2 data

## 8.3 INDIVIDUAL STRUCTURE INSPECTION FILE CONTENTS

The inspection file for each bridge/structure shall consist of a wide variety of information from several sources to ensure sufficient information is readily available for safety inspections and overall bridge management. Because sources for most of the bridge information is more short-lived than the bridge structure itself, it is important that the inspection file becomes the final repository from which information on the bridge's design, construction and maintenance can be retrieved to better evaluate current conditions.

In the past, many components of the structure inspection file resided in hard copy format only. In the digital world we now live in, all new records shall be filed in BMS2 under the documents screen. By placing all components of the inspection file in one central location, the data is easily accessible for future use. For structures in which the entire inspection file contents are not included in BMS2, Districts must upload a pdf of the inspection file index (see below) to BMS2.

INSPECTION RECORD FILE INDEX: Because portions of the Inspection File may be stored in more than one physical location, an index of the information available is critical to enable the inspector to quickly access information needed to evaluate a structure. A good index for each bridge should identify the types of records available, their format, storage location, date of record, etc.

DISTRICT FILES FOR LOCAL BRIDGES: For local bridges, the Districts are to maintain on BMS2, at a minimum, a copy of the last inspection, the current bridge rating and posting evaluation, and documentation of posting verification.

### 8.3.1 Field Inspection Records

These records are typically generated through the routine safety inspection program activities and include for each individual structure as applicable:

- Inspection Reports (all inspection types, see IP 2.3)
- Fatigue and Fracture (F&F) Plan (pdf file and fillable form version)
- Fatigue Analysis (pdf file)
- Rocker Bearing Movement Spreadsheet (Excel file)
- Element Level Quantities
- Complex Bridge Inspection Procedures
- Underwater Inspection Procedures
- Tunnel-Specific Inspection Procedures
- Documentation and results of functional systems testing

All bridge inspection reports shall include the following items:

- Cover Page with photo, “Not for Public Record” stamp (see IP 1.8.3), and basic bridge identifying info (see the top of PennDOT’s Inspection Report Generator Cover page, in BMS2, for minimum fields)
- Inspection Summary*
- Table of Contents/Bookmarks
- Location Map
- BMS3 Report
- Photographs**

*Inspection Summary shall include at a minimum the information provided in the PennDOT Inspection Report Generator available in BMS2: bridge description, bridge type, inspection schedule, 2A02 Inspection Notes, bridge posting recommendation summary, inspection team, load rating summary, and maintenance priorities.
**Photographs to include at a minimum: approaches, elevation views, upstream and downstream channel (or crossing feature), general condition of each component, deficiencies, posting restrictions, and other important features. See IP 2.2.8 for additional guidance.

The following items should also be included in bridge inspection reports as applicable:

- Defect Sketches
- Rocker Bearing Movement Spreadsheet (Appendix IE 04-B)
- NCABB Inspection Forms (Appendix IE-04-C)
- Corrugated Metal Culvert Inspection Forms (Appendix IE 04-E)
- Clearance Sheet for Features Intersected
- F&F Plan with Signed Cover Page (Appendix IP 02-H)
- Uncoated Weathering Steel Forms (Appendix IP 02-I)
- Stone Masonry Inspection Forms (Pub. 100A, Appendix G)
- Scour Plan of Action (for locally owned bridges)
- Waterway Sketch
- Waterway Contour Map (for underwater inspections only)
- Completed Scour Templates at each subunit and structure opening
- Priority 0/1 Notification Correspondence
- Other Correspondence
- Destructive and Non-Destructive Test Results
- Load Rating Analysis (only if analysis updated as part of the inspection report)
- Truss Inspection Findings Summary (Appendix IE 04-A)

### 8.3.2 Load Rating Analysis

The Load Rating Analysis is part of the safety inspection of a bridge but is treated separately here to ensure the proper attention is given. The Load Rating Analysis shall include the following documentation:

- Load Rating Summary Form, including Pennsylvania P.E. seal and signature
- Supporting calculations including any hand-calculations and/or spreadsheets
- Sketches showing member cross sections and section loss, if applicable
- Computer output and input files
- References to appropriate design, rehab or shop drawings
- Engineering Judgment Documentation, if applicable

A sample Load Rating Summary Form is provided in Appendix IP 03-C. A template version of this form can be found in the Forms and Templates link on the BMS2 homepage. The condition of the bridge and date of the inspection that the rating is based upon should be documented with the above. In addition, the summary information should be coded in BMS2 on the Load Rating screen. Critical information such as controlling member identification and associated section loss and section remaining data should be recorded on the load rating summary form and in BMS2 field IR19- Notes for all load types so bridge inspectors are aware of the critical section and its properties used in the analysis.

A sample assigned load rating approval form is provided in Appendix IP 03-D. A template version of this form can be found in the Forms and Templates link on the BMS2 homepage. A summary of the assigned load rating must be included in the bridge file as indicated in IP 3.6.1.2 for all bridges which have ratings assigned from the design.

### 8.3.3 Posting Evaluation

The Posting Evaluation records are part of the safety inspection of a bridge, but are treated separately here to ensure the proper attention is given to these critical records which include:

- Posting Evaluation
- Posting Recommendation Data Sheets
- Posting Approval Letter
- Related Correspondence

### 8.3.4 Design-Related Information

Information generated during the design of the bridge that should be incorporated into the permanent inspection file includes:

- Design plans for original construction or rehabilitation
- Design Computations
- Design Exception Approval letters (Used in Rating Appraisal Items)
- Foundation Report
- Surveys
- Special Provisions
- ROW Plans

### 8.3.5 Waterway and Scour-Related Reports

Reports that assist in evaluating the waterway opening and the bridge's resistance to scour include:

- Hydrology and Hydraulics Reports
- Observed Scour Assessment Report
- Scour depth computations (may be part of H&H or stand-alone calculations)
- Scour Plan of Action

### 8.3.6 Construction and Maintenance Records

Records regarding construction and maintenance to be part of the bridge inspection file may include:

- As-Built drawings
- Shop Drawings
- Material Certifications
- Technical Specifications
- Pile Hammer Approvals and Pile Driving Records
- Field Change Orders
- Jacking and/or Demolition Schemes
- Documentation of latent defects
- Maintenance Work Orders, Sketches
- Repair Records
- Construction Photos

### 8.3.7 Miscellaneous Documentation

Other documents that may be maintained as part of the inspection file include:

- Bridge Problem Reports
- PUC Documents
- Occupancy Agreement and/or Permit
- Miscellaneous Bridge-Related Correspondence
- Cost Estimates for Improvements
- Reports of Field and/or Laboratory Testing (including equipment type, manufacturer, serial number, calibration certificate, and operator)
- Meeting Minutes of Critical Deficiency Meetings with Owners
- Plans of Action for Critical Deficiencies

### 8.3.8 Field Preparation Documentation

Preparation requirements for the field phase of an inspection vary greatly. Variations may be due to structure type, site accessibility, traffic volume, or channel conditions. Documenting field preparation requirements can reduce budgets by maximizing mobilization efficiency. The following areas of preparation, where applicable, are to be documented for each bridge.

- <u>Required Tools and Equipment</u>
  Identify any specialized tool or piece of equipment necessary that is not ordinarily carried by the bridge inspector. Example tools might be extendable ladders, special non-destructive testing equipment, power tools, lights, special safety equipment, special underwater tools or diving gear.
- <u>Special Services</u>
  Record any special services that are required. Example services might be traffic control, structure cleaning operations, inspection access such as structure rigging, an under-bridge inspection crane, or special working platforms such as a barge.
- <u>Scheduling</u>
  Document specific scheduling needs for non-routine inspections. This includes manpower needs for larger structures that require an extended duration inspection effort with multiple inspectors, bridges subject to seasonal flooding conditions, NSTM bridges where special services are required, and underwater bridge inspections.
- <u>Site Condition Considerations</u>
  Identify unique site conditions that require more than routine preparation. Unique site conditions include railroad property right-of-way restrictions, navigable waterway restrictions, high voltage transmission lines, unusually heavy vegetation, mud, pollution, insect or animal droppings, unusually high water level or unique traffic safety procedures.

For tunnels being inspected in accordance with the NTIS, additional field preparation documentation is required as outlined in Section IP 8.3.9 below.

### 8.3.9 Tunnel Specific Inspection Procedures Documentation

In accordance with the NTIS, every tunnel shall have a written document outlining the tunnel specific inspection procedures which shall be saved in the tunnel file for future inspectors' reference. NTIS specifies this document shall take into account the design assumptions and tunnel complexity and also identify the following: tunnel structural elements and functional systems to be inspected; methods of inspection to be used; inspection interval for each inspection type; and inspection equipment, access equipment, and traffic coordination needed.

In order to create uniformity for all PA tunnels, PennDOT has developed guidance on the development of the tunnel inspection procedures document which includes ten major headings and a summary of the intended contents as detailed in the paragraphs below. If any heading does not apply to the given tunnel (e.g., specialized training requirements may not be applicable for a non-complex tunnel) the heading shall still be used, but a brief explanation of why it does not pertain should be included. Additional headings may be added at the end of the document as needed. For additional guidance, see Section 4.5 of the TOMIE Manual.

1. Introduction – The introduction shall include general background information about the tunnel including the tunnel name, location, length, construction type, design assumptions, rehabilitation information, number of tubes, owner, classification (complex or non-complex), target date for inspection, unique features, established survey control (panel points/stationing), and any other key details.
2. References – This section shall include a list of all references the inspectors would use to perform the inspection. At a minimum, this would include Publication 238, Publication 100A, NTIS, TOMIE, and SNTI. Other references may also include the FHWA Tunnel Load Rating Manual, MUTCD, NFPA, NEC and other codes applicable to the functional systems present within the tunnel.
3. Specialized Training Requirements – The NTIS shall be referenced here for minimum requirements of team leaders. If additional qualifications or personnel are warranted for complex tunnels, they should be included in this section. For example, an owner may desire to have a team leader oversee each discipline during the inspection and an overall team leader to oversee and coordinate the whole inspection. In addition, this is a good place to call out qualifications of specialty contractors if needed for systems testing for complex tunnels.
4. Inspection Types and Intervals – Include a brief description of the scope of each type of inspection and the interval associated with each. Per the NTIS, all inspection types must be mentioned in this document including routine, in-depth, special and damage.
5. Access/MPT – Means and methods associated with access of all portions of the tunnel shall be spelled out herein. Provide contact information for key personnel which may include owner, maintenance coordinator, local emergency room, fire department, or other stakeholders (railroad, etc.). Entry and exit points of support spaces, cross passageways and other inspectable spaces shall be specified along with rules and restrictions such as who to contact if the locations are locked or if inspectors must access during day or night hours only. Traffic control requirements and configurations shall also be called out along with who is responsible for the maintenance and protection of traffic (owner or inspection team). This section may also include detour information (if required), access equipment needs and locations of parking/staging areas.
6. Health and Safety – While PennDOT's Publication 238 provides a detailed description of typical hazards for field inspection personnel, this section allows for a detailed analysis of any additional hazards specific to the tunnel location. If there is a confined space present within the tunnel which must be entered as part of the inspection, this section shall specify if it is permit-required or not and provide a detailed description of the space so the team leader can develop an appropriate confined space entry plan.
7. Elements Present – List all the elements present within the tunnel and a description of each element including a description of its location. If there are any elements present within the tunnel which the owner wishes to track, but is not included in the SNTI, an agency defined element shall be called out along with guidance on condition state coding of that element. This section may be supplemented with labeled section views or photographs to identify the location of elements present.
8. Functional Systems Testing – Define the critical systems present, types of testing required, testing procedures, interval for each test and who shall perform the testing (tunnel maintenance, specialty contractor, other). Include the location of testing documentation (BMS2, tunnel operations center, other) and how the team leader shall go about obtaining the data. This section shall also define the interval for direct observation of systems testing by the team leader. Define any functional systems or utilities suspended over the roadway and provide means of inspection of the structural components.

9. Inspection Best Practices/Techniques – This section shall include inspection best practices and techniques to be utilized including physical methods and any advanced techniques required (NDT). If inspection includes a "sampling" of an area, define the expectation for sampling (i.e., sound 25% of surface area of tiles) and whether or not the sampling varies with inspection type or interval. Identify requirements for data collection (sketches, field forms, etc.) for uniformity in data collection from one inspection to another for comparison. Identify the locations of minimum vertical and horizontal clear for measurements to be field verified and note how often these values shall be collected. Identify whether or not a load rating analysis is required for the structure and the posting status.
10. Reporting – This section allows the tunnel owner to set guidelines for inspection reports for uniformity based on preferences and may include a sample table of contents, list of required appendices, guidance on photo expectations, excel templates etc. At a minimum, this section shall remind inspectors of the importance of providing sufficient justification of the assigned condition state coding of all elements and systems documented within the report.

## 8.4 FILE MAINTENANCE

### 8.4.1 Record Retention Period

In accordance with OA Management Directive 210.5 and the approved Records Retention and Disposition Schedule the following is a list of guidelines for records retention:

1. Retention of paper bridge files shall be 7 years. After 7 years the files are to be sent to the State Record Center for storage.
2. Retention of electronic and microform bridge files shall be 100 years.

For Department bridges that are turned back, given or sold to local municipalities or private/public organizations, all bridge inspection file information shall be given to its new owner. The District needs only a file with contents similar to other local bridges. A record of the ownership transfer shall be maintained in the bridge file.

### 8.4.2 Retention Method

1. The agency (District and Central Office) must submit all electronic inspection files to ECS for storage. This can be done using the Documents link in BMS2.
2. Electronic files and microform storage management is at the discretion of the agency (District or Central Office).

## 8.5 DEPARTMENT FORMS FOR INVENTORY AND INSPECTION

### 8.5.1 Structure Inventory Forms for BMS2 – D-491 Series

The D-491 series of forms are a compilation of the inventory information contained within BMS2. The D-491 series are organized to show the bridge inventory inspection information as it appears on the BMS2 Screens. See Publication 100A, Appendix F for a blank version of the D-491 forms.

### 8.5.2 Field Inspection Pages for Bridges – BMS3

The Department developed the BMS3 field inspection pages to record condition/appraisal ratings of bridge components with narrative comments to support those ratings, element-level data and major improvement/maintenance needs in a uniform manner Statewide. The following make up the BMS3 pages:

- General
- Schedule
- Ratings
- Approach
- Deck
- Joints
- Superstructure

- Bearings
- Substructure
- Culvert
- Waterway
- Load Rating
- NSTM/Fatigue
- Elements
- Features
- Maintenance
- Notes
- Signs & Lights
- Walls
- Tunnels
- E-Docs

The BMS3 pages have limited space for comments and shall be supplemented when the narrative needs to be expanded to fully document the bridge conditions. Additional pages of lined or quad ruled paper are appropriate for this use. Sketches to describe and record conditions and/or tables of measurements are also most useful for documentation. New photos of the structure and defects (see IP 8.3.1) shall be included with dates and captions. To record NSTM/Fatigue inspection information, use PennDOT Form D-491 NSTM/Fatigue for BMS2 data and additional sheets for narrative.

### 8.5.3 Field Inspection Reports Using BMS3 Inspection Module

The inspection report format of the Department's BMS3 Inspection Module for electronic data collection of bridge safety inspections is patterned after the D-450 series of inspection forms and can be used as stand-alone or in conjunction with the D-491s and additional pages as per IP 8.4.2. A hardcopy of the BMS3 field inspection report is to be maintained in the bridge inspection file.

### 8.5.4 Field Inspection Pages for Retaining Walls and Sign Structures

The Department's BMS3 Inspection Module is to be used for the inventory and inspection of retaining walls and sign structures.

Similar to bridges, sketches to describe and record conditions and/or tables of measurements are most useful for documentation. Provide a sketch of the sign structure and roadway features showing all clearances. Roadway centerline, lane widths, shoulders, and guide rails are to be shown. Vertical clearances over roadway pavement and shoulder breaks are to be indicated. This data is to be field verified at each inspection.

Use the following referencing nomenclature and refer to Appendix IP 02-D, Figure 1:

- Near side / far side:

  The NEAR SIDE of the sign structure is as indicated in Appendix IP 02-D Figure 2, "Sign Structure Identification".

- Truss Referencing:

  The four (4) chord truss system will be referenced to as (LF) lower front; (UF) upper front; (LB) lower back; (UB) upper back. <u>The Panel Points will be numbered from left to right, while looking at the Near Side of the Frame.</u>

  The three (3) chord truss system should be referenced as noted above, except that the back <u>single</u> <u>chord</u> is referred to as <u>Mid-Chord</u>. Example: The interior diagonal from upper chord (U6) to Mid-Chord (M7) should be labeled U6-M7.

General photographs depicting the sign structure and showing the approach roadway and alignment, plus a side elevation view, as well as photographs of defects, must be included as part of the report. A photograph of the sign message (each sign) is also required. All photographs shall include a date and caption.

## 8.6 ELECTRONIC DOCUMENT MANAGEMENT SYSTEM

The PennDOT electronic document management system, Enterprise Content Services (ECS), is a group of computer software and hardware tools that electronically capture, store, and index paper documents such as engineering drawings, photos, sketches, analyses, and other documents and bridge inspection and load rating analysis file information. Users can take an existing business workflow that involves routing, distribution and approval and automate the whole process cutting down on the turn-around time and improving on overall efficiency. A benefit of ECS is reduced storage space, processing time, transmittal time, queue time, and administrative support. Bridge specific documents stored in ECS can be accessed through the Documents link in BMS2.

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## 9.1 GENERAL

Many types of standard tools, personal protection, traffic control, special equipment, safety and first aid, and special vehicles are available to aid in the inspection process. Proper advance planning is essential to determining the inspection equipment required to adequately inspect the bridge in an efficient manner. Most inspection organizations have limited workforce resources – having the right equipment for the right job the first time will ensure those resources are used wisely.

Answers to the following questions will determine the proper inspection equipment needed:

- What type of structure will be inspected?
- What type of inspection?
- What method of inspection?
- What kind of equipment is needed to access the remote portions of the bridge?
- When will the inspection be done?
- What feature(s) does the structure intersect?
- What hazards may be encountered (traffic, weather, wildlife, etc.)?
- How will team members communicate if large structures have them separated?

It is important that inspection teams are outfitted with the proper equipment to:

- Facilitate personal and public safety during inspection of the structure
- Perform an efficient and accurate inspection of the structure
- Perform the proper level of inspection intensity
- Correctly record the conditions of the structure

Special equipment required and method of inspection, once it has been determined for a structure, should become part of the bridge inspection file for future use.

## 9.2 INSPECTION TOOLS AND EQUIPMENT

### 9.2.1 Bridge Inspection Equipment

FHWA's Bridge Inspection Reference Manual (BIRM) provides additional information on inspection equipment in Section 2.4. For tunnel inspection equipment guidance, see the TOMIE Section 4.8. The information in these reference manuals will assist the Districts, bridge owners and consultants in properly preparing for field inspections. Inspectors should not be limited to the equipment on these lists as special circumstances may necessitate the use of non-standard tools.

The Department maintains a Statewide fleet of under-bridge inspection cranes for use on bridge inspections where access may otherwise be a problem (see IP 1.12).

### 9.2.2 Sign Structure Inspection Equipment

Most of the equipment needed for sign structure inspections is covered in the BIRM's standard tool list for bridge inspection.

The following equipment is needed or especially useful for sign structure inspections:

- Dye-penetrant testing kit (Must be used any time aluminum structural members are suspect, especially welds)
- 24-ounce ball-peen hammer
- Cans of cold galvanizing or zinc rich paint
- Binoculars (10 X 50)
- Magnet
- Dial-indicating Torque Wrench (0-300 in.-lb. range.)
- Pipe wrench
- Spanner wrench and 4 ft. pipe extension
- Wrenches (1/2" to 1- 3/4")
- Socket wrench and deep sockets (1/2" to 1- 3/4")

- Standard size nuts, washers, bolts and clips to replace missing components (to be provided by the Department).

## 9.3 INSPECTION SAFETY

Bridge inspection presents many hazards when working around the bridge site. The best way to avoid many of these hazards is to be well prepared for the conditions and be knowledgeable of the surrounding area. The following are some of the more common hazards to be aware of:

- Traffic – Many inspections are conducted while traffic is allowed to cross the bridge. Inspectors should always know in which lanes traffic is permitted to travel on the bridge, and inspectors should always stay within the confines of the traffic control pattern if provided. If no traffic control is available, inspectors should conduct their inspections from the shoulder of the roadway when inspecting the topside of the bridge and limit the number of times crossing the roadway to avoid oncoming traffic.
- Weather – Pennsylvania experiences extreme weather conditions in both the winter and summer. During the winter, inspectors should dress warmly and appropriately for the colder temperatures. Layers of clothing should be worn so that they may be taken off or added, depending on the temperature fluctuations. Inspectors should also be aware of icy conditions around the bridge site. Traversing through partially frozen streams and creeks presents treacherous footing conditions. The summer months in Pennsylvania may produce very hot and humid temperatures. Inspectors should drink plenty of fluids throughout the day to prevent dehydration, and sun screen or sun block should be applied to exposed skin to prevent sunburn.
- Wildlife – Pennsylvania is the home to many species of wildlife that may be harmful to the inspector when conducting bridge inspections. Snakes, such as copperheads and rattlesnakes, are poisonous and are present throughout most of Pennsylvania. Wasps, bees and hornets are also potential hazards in the field, especially for people who are allergic to bee stings. Ticks are another pest that lurk in tall grasses and brush. The deer tick carries the virus for Lyme disease and can cause serious health problems if undetected. Inspection personnel should check themselves for ticks and tick bites daily.
- Inspection Equipment – For some inspections it is necessary to use specialized equipment such as cranes, lifts, etc. to access bridge components. When operating inspection equipment, inspectors should obey all the manufacturer safety policies and procedures. Adequate training shall be provided to all personnel operating any type of inspection equipment. Operators should note where all potential obstructions (e.g., power lines) are located so they can be avoided.

Inspection personnel should know the potential hazards before they go out to each individual bridge site. In addition, they should also know where the nearest hospital is located and the phone numbers of emergency personnel in the area. Being well prepared for inspections will help inspection personnel to avoid potential dangers.

## 9.4 TRAFFIC CONTROL

Provide traffic control during the inspection as needed in compliance with the Department's Publication 213, Temporary Traffic Control Guidelines. Whenever possible, coordinate inspection effort when other work is planned at the site in order to minimize disruption of traffic.

Any anticipated lane closures should be coordinated in advance since the times for inspection may be restricted by the District Bridge Inspection Supervisor or District Traffic Unit due to anticipated traffic conditions.

For traffic control details differing from Publication 213, submit a sketch to the District for approval. These sites will be identified by the District.

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## 10.1 GENERAL

The efficient movement of goods and services within and through the Commonwealth necessitates that some vehicles may exceed the Size, Weight and Load provisions for vehicles specified in Chapter 49 of the Vehicle Code. In order to provide for some assurance of the safe movement of these large vehicles and to prevent damage to our highway and bridge infrastructure, it is the bridge owner's responsibility to review the vehicles and their possible effect upon the bridges on or over the intended route before approving its movement.

The regulations and procedures for issuing hauling permits for these oversize and overweight vehicles are promulgated in Chapter 179 Oversize and Overweight Loads and Vehicles of Title 67 TRANSPORTATION of the Pennsylvania Code. This section is not intended to be an in-depth review of those rather extensive and changing regulations and procedures, but more of a guide for the District bridge personnel and other bridge owners as they review the bridges on the proposed routes on these permit applications.

The Department's basic philosophy regarding oversize/overweight permit vehicles is to review every bridge on every route for every permit application. That means that the clearance of every bridge on the intended routes for permit vehicles will be checked to ensure that the vehicle will pass beneath the overhead bridge restriction. In addition, each bridge on the intended route will be structurally evaluated to ensure it can safely carry the load imposed by an overweight permit vehicle. If any bridge cannot safely accommodate the height or weight of a permit vehicle, that route is to be rejected.

The Department's Automated Permit Routing Analysis System (APRAS) (see IP 10.5) allows for the vast majority of these bridge checks to be made automatically using the bridge clearance envelope data and the bridge analysis program datasets stored in BMS2. The accuracy and safety of results of these reviews is heavily dependent upon the quality of bridge inspections and ratings.

In this Chapter, two generic terms will be used for brevity and are not intended to imply the more legalistic definitions used in the permit regulations.

- "Permit" will be used here to refer to the variety of hauling permits for oversize and overweight vehicles and not to utility occupancy related issues.
- "Vehicle" will be used here to represent both single-unit vehicles and combination (tractor-trailer) vehicles.

## 10.2 PERMIT CATEGORIES

The terminology in M 6B8 is not consistent with the PA permit regulations and Department procedures. Department terminology will be used in this Publication. For bridge review purposes, there are three basic categories of hauling permits: Single Trip Permits, Annual Permits, and Blanket Permits.

### 10.2.1 Single Trip Permits

Single Trip Permits are permits for the one-time movement of a non-divisible oversize and/or overweight vehicle on a specified route. Axle weights and spacings must be provided in addition to the physical dimensions of the vehicle.

Permit conditions may be imposed upon the permittee by the Department to reduce the load effect on the bridge and ensure the safety of the movement. The permit conditions are listed on the copy of the permit that must be carried in the vehicle. Permit conditions may include, but are not limited to:

- The permit vehicle being the only vehicle on the bridge
- Limiting travel to specific lanes
- Limiting vehicle to crawl speed across bridge(s)
- Temporary shoring of bridges

These permit conditions must be evaluated carefully to ensure they will achieve desired reductions on load effect to the bridge. In addition, the imposition of some of the conditions may have a negative impact on the overall traffic conditions that might preclude their use at particular sites. The Bridge Engineer should consult with the Traffic

Engineer for those cases. If a permit condition needed to ensure the structural safety of a bridge on the route cannot be effectively or safely put in place, the application for that route must be denied.

There are other specialized permits that are reviewed and processed for bridge structural safety issue reviews in a similar manner to single trip permits:

- Super Load – A vehicle having a GVW greater than 201,000 pounds, or a total length exceeding 160 feet, or a width greater than 16 feet. (Buildings being moved, drag lines, and other quarry equipment excluded).
- Permit to cross bridge with vehicle weighing more than the posted bridge restriction.

### 10.2.2 Annual and Seasonal Permits

Annual Permits are permits for individual overweight vehicles carrying specific types of cargos in specified vehicle configurations (including axle weights and spacings) on the applicant-specified routes and are granted for a period of up to 12 months. The applicant vehicle is checked for conformance to the specified configuration. The bridges on the permit route are checked through APRAS for a set of axle weights and spacings specified for that type of annual permit before the permit is issued. The Bridge Design and Technology Division (BDTD) determines the standard set of axle weights and spacings to be used for the structural check of the bridges. Once the annual permit is granted, the permittee may travel the approved route as many times as desired.

Since 1998, twenty-five overweight Annual Permit vehicles have been introduced to PA highways and bridges as a result of legislation. See Appendix IP 10-A for a list of Overweight Annual Permit Vehicles.

For the purposes of this manual, Seasonal Permits will be considered and treated as Annual Permits.

APRAS re-analyzes the bridges on the annual permit route on a regular basis to check for changed bridge conditions. The annual permit is revoked or re-routed by the Department if any bridge on the approved route fails one of these subsequent reviews.

Because the Department has less control over the number and movement of Annual Permit vehicles after the permit is issued than for single trip permits, the permit conditions (discussed in IP 10.2.1) used to reduce the load effect of overweight vehicles on bridges are not allowed to be used for Annual Permits. In addition, vehicles with an Annual Permit alone are not allowed to cross bridges posted at weight limits less than the vehicle GVW.

### 10.2.3 Blanket Permits

Blanket Permits are permits for individual overweight vehicles carrying specific types of cargo and with specified vehicle configurations (including axle weights and spacings) on a Department-designated network of routes, granted for a period of up to 12 months. BDTD determines the standard set of axle weights and spacings to be used for each of the Blanket Permit vehicles. The bridges on the designated Blanket Permit network are structurally reviewed on a regular basis through APRAS for a set of axle weights and spacings standard for that type of Blanket Permit vehicle. A list of bridges the Blanket Permit vehicles are not allowed to cross is issued with the permit. The applicant vehicle is checked for conformance to the specified configuration before the permit is issued. Once the Blanket Permit is granted, the permittee may travel the approved network as many times as desired.

Since 1994, four overweight Blanket Permit vehicles have been introduced to PA highways and bridges as a result of legislation. See Appendix IP 10-A for a list of Overweight Blanket Permit Vehicles.

APRAS re-analyzes the bridges on the designated Blanket Permit network on a periodic basis to check for changed bridge conditions. It is the responsibility of the Permit Office to send the responsible Motor Carrier the updated "Roadway Restriction List for Blanket Permits" attachment and any "Additional Route" attachment produced after Network analysis. It is the responsibility of the Motor Carrier to ensure a current copy of the "Roadway Restriction List for Blanket Permits" is attached to each Special Hauling Permit.

Because the Department has less control over the number and movement of Blanket Permit vehicles after the permit is issued than for single trip permits, the permit conditions (discussed in IP 10.2.1) used to reduce the load effect of overweight vehicles on bridges are not allowed to be used for Blanket Permits. In addition, Blanket Permit vehicles are not allowed to cross bridges posted at weight limits less than the vehicle GVW.

## 10.3 LOAD CAPACITY EVALUATION

The load capacity evaluation of bridges to carry commercial traffic is a critical safety issue and an important issue to industry. Two types of common commercial loads fall outside the envelope of the Bridge Posting Vehicle ratings performed for bridge safety inspection and are a special concern:

- Overweight vehicles – vehicles that exceed the applicable maximum gross weights specified in Chapter 49 Subsection C of the Vehicle Code.
- Vehicles with a GVW that exceed the bridge's posted weight restriction established under § 4902(a) of the PA Vehicle Code.

This section will discuss how the bridge analyses and the approval for the hauling permit process for these bridge overloads are accomplished.

### 10.3.1 Authorization for Overloads on Bridges

The District Bridge Engineer is authorized and responsible for the approval of overloads on the Department's bridges in the District.

For bridges on State Routes under the jurisdiction of the PUC, the responsibility and authorization for issuance of permits should be outlined in the PUC Order. Typically, it could be assumed that the responsibility for inspection and maintenance of the bridge superstructure includes the permitting authority. In the absence of such formal authorization, the Department should assume this authority under its obligation for the safety of the State highway users.

For other non-Department bridges, the bridge owner is responsible for the approval of overloads on its bridges.

For LOBSTORS (Locally-owned bridges on State Routes) or other non-Department bridges carrying State Routes, the owner may authorize the Department to review and issue heavy hauling permits for permit vehicles using the State Route to cross the bridge. This authorization should be in the form of a letter from the owner to the Department and is to be maintained in the District Bridge Unit files.

### 10.3.2 Maximum Permissible Load Effect on Bridges

The maximum permissible load effect of a permit vehicle on bridges after consideration of all permit conditions and pertinent analysis factors (e.g., LL distribution, impact, uplift, temporary shoring) is the SLC. All permit applications exceeding the SLC must be denied.

For bridges of special concern, and bridges with a superstructure and/or substructure condition rating less than or equal to 4, the SLC should be used as the upper bound of the load effect on a bridge.

### 10.3.3 Live Load Distribution for Permit Vehicles

The distribution of permit live load on the bridge is to be in conformance with provisions of this manual. All exceptions must be approved by the Assistant Chief Bridge Engineer - Inspection.

SINGLE LANE DISTRIBUTION: The application of the single lane load case for simplified girder distribution factors on a multi-lane bridge is allowed when:

- The permit vehicle has no more than two wheel lines and its gage distance is not less than the AASHTO specifications
- It can be assured by conditions in the permit that no other vehicle will be on the bridge at the same time as the permit vehicle.

DISTRIBUTION BASED UPON LATERAL POSITION OF VEHICLE: The distribution of live load for 2 girder bridges or trusses may be based upon a specific lateral position of the permit vehicle relative to the normal traffic lanes to reduce the live load effect if it can be assured by permit conditions that the vehicles will be in that position on the deck. Examples of special vehicle positions include, but are not limited to:

- Permit Vehicle to Straddle Centerline – used to minimize live loading to longitudinal truss or main girder of 2 girder system
- Permit Vehicle to Stay in Rightmost Lane – may be used to limit live load effect on transverse floor beam or tie-girder.

Note: Requiring the permit vehicle to travel adjacent to curb may not be feasible if the approach pavement does not extend to the curb line. Overweight permit vehicles are not permitted to travel on approach shoulders.

CRABBING: Some motor carriers have proposed the use of "crabbing" the permit vehicle to attempt to reduce the live load effect on a bridge. Crabbing involves steering the rear end of the trailer so that the front and rear ends of the trailer are in different lanes.

NON-STANDARD GAGE: When the gage distance between wheel lines of the permit vehicle are not in accordance with AASHTO standards, the distribution of live loads must be carefully considered. For gage distances less than the standard of 6'-0", the AASHTO simplified line girder distribution factors (See IP 3.3.2) are not applicable. Other examples of vehicle configurations where the simplified line girder distribution factors may not work include: When small trunnion axles are used in place of dual wheels or when there are more than 2 wheel lines. For these and other non-standard vehicles, the Districts may use the lever rule (see IP 3.4.1), a refined method of analysis (see IP 3.4.2) or may request guidance from the Assistant Chief Bridge Engineer - Inspection.

DUAL LANE TRANSPORTER: Dual Lane Transporters (DLT) are very large vehicles designed to spread the load out wide enough and long enough to meet bridge analysis load limits. They have multiple axles or trunnions side by side and usually take up more than one lane. They present an interesting challenge to bridge analysis in APRAS. For non DLT loads, the first analysis trial is usually with distribution factors based on two or more lanes loaded. Two lanes loaded with AASHTO type vehicles side by side performs similarly to a DLT. Therefore, analysis of a DLT, with AASHTO gauge and passing distances, can be performed by using half of the axle weight of the DLT axles with the distribution factor based on two or more lanes. For DLT's that have gauge and/or passing distances smaller than specified in AASHTO, an additional step is necessary. A lever rule analysis can be performed on varying girder spacings to determine the largest distribution factor for the bridges being analyzed. This can be used to calculate a multiplier that can be applied to the axle weights of the DLT. The multiplier provides a quick analysis method when importing the permit vehicle axle weights and spacing and bridges on the route from APRAS to stand alone ABAS.

### 10.3.4 Impact Load and Crawl Speed

In accordance with IP 3.2.3, impact loading must be included in the bridge evaluation of all permit vehicles.

A reduction in the impact factor to a minimum of 10% of the live load effect is allowed when permit conditions can assure that the vehicle crosses the bridge at crawl speed. For this purpose, crawl speed is defined as no more than 10 miles per hour with no acceleration or deceleration on the bridge. This last requirement normally necessitates stopping the vehicle well before the bridge and proceeding across the bridge at a steady rate. Use of an impact factor less than 10% must be approved by the Assistant Chief Bridge Engineer - Inspection.

### 10.3.5 Uplift Under Permit Loads

Uplift at the end spans of continuous bridges is to be avoided because of the potential for damage to bearings not designed for uplift. The potential for uplift is much greater under the typically heavy axles of the permit vehicle.

Uplift is considered to occur if the total (live load + dead load) reaction is less than 10% of the dead load reaction alone. If the uplift cannot be mitigated by hold-downs or added dead load, the permit should be rejected.

### 10.3.6 Temporary Shoring for Permit Loads

A permit applicant may propose to provide temporary shoring and/or repairs at their cost to a Department bridge to provide the additional strength to allow their vehicle to cross the bridge safely that would otherwise be rejected. The permit applicant may even propose a temporary bridge to bypass or to span over the Department bridge.

Any such proposal for temporary shoring, bridge, etc., must be submitted to the District Bridge Engineer for approval as a condition to the permit. The proposal must:

- Be prepared and signed by a professional engineer licensed in PA
- Have adequate sketches or plans and be supported by engineering computations
- Have acceptable details
- Contain all necessary permits for work in the waterway, etc.
- Agree to remove any temporary supports or material as required by the Department
- Provide materials acceptable to the Department
- Be acceptable to the District Bridge Engineer

The Department retains the right to deny a permit application proposing temporary shoring, bridges, etc., and to revoke any permit issued if the materials and or workmanship of the required temporary shoring, etc., is not satisfactory to the District Bridge Engineer.

### 10.3.7 Load Evaluations by Permit Applicant

Superload applicants that receive a denial notice, and have fully optimized their vehicle and route, may perform a load rating evaluation on a Department bridge by a more refined analysis method. The load rating analysis must be performed by a professional engineer licensed in PA and familiar with bridge analyses. Load effects on all bridge components, not just the superstructure, shall be considered in this analysis. Companies that intend to provide these services must be a registered business partner in ECMS, as stated in IE 8.21 and must submit a Quality Plan. The company will also be responsible for performing pre- and post-move Special Inspections as per IP 2.3.5 for the bridges on the route. The Department retains the right to deny the permit application if the evaluation is not acceptable to the District Bridge Engineer. The need for load rating bridge components other than the superstructure, and pre- and post-move inspections increases as the GVW of the permit vehicle increases. One or both may be waived by the District Bridge Engineer upon request.

## 10.4 POSTED BRIDGES AND PERMITS

As per IP 4.4, careful consideration of permit applications for overweight vehicles to use bridges restricted due to their condition (under §4902(a) of the Vehicle Code) must be made because of the potential for bridge failure.

Permit applications to cross bridges posted for reasons other than the condition of the bridge under V.C. §4902(b) should be reviewed in a similar fashion as unposted bridges and a special permit as per IP 10.4.2 is not required.

### 10.4.1 Bridges Limited to One Truck

When a bridge has a restriction of traffic limited to one truck at a time only without an accompanying weight restriction, an applicant can request a permit for an overweight permit vehicle.

### 10.4.2 Bridges with a Posted Weight Restriction

When a bridge has a weight restriction posted due to its condition, a special permit is required for a vehicle to exceed the posted weight limit. Paraphrasing from § 191 of TITLE 67 TRANSPORTATION of the PA Code:

- The posting authority may permit an over-posted-weight or over-posted-size vehicle or combination to use a bridge posted under 75 PA Consolidated Statutes § 4902(a) if it determines that:
  1. For all practical purposes, the vehicle or combination can reach its destination only via the posted bridge; and
  2. Analysis of the number of axles, axle weights, distance between axles, height, width and other data indicates that the vehicle or combination will not have a detrimental effect on the bridge.
- The permit may authorize a single trip, a limited number of trips during a 12-month period, or an unlimited number of trips during a period not to exceed three months.

Accordingly, the review and approval of such permits by the District Bridge Engineer for Department bridges is handled in much the same way as single trip permits, although a special application is required.

The posting authority may also authorize a special hauling permit when the vehicle or combination weight exceeds the posted bridge weight limit if the same two criteria listed above are met. For this case, the approved M-4902 Application/Permit to exceed Posted Weight or Size Limit must be on file with the posting authority or attached to the special hauling permit application.

## 10.5 AUTOMATED PERMIT ROUTING ANALYSIS SYSTEM (APRAS)

The APRAS system is designed for the electronic application and processing of special hauling permits by PennDOT users and its customers. APRAS utilizes client/server based and Internet technologies to provide for easy access to permitting information to Industry and the Department.

There are two ways to access the APRAS system, via the Department Network or via the Internet. Customers can apply for permits, search for and check the status of permit applications, and access additional support functions via the Internet or submit hardcopy applications to Department permitting staff who, in turn, access APRAS through their local PC Workstation on the LAN. PennDOT staff has the ability to enter permit applications, process applications, approve and deny applications, and manage the system.

Major components of APRAS of interest to the review of bridges include:

- **APRAS Database** contains all information required for permitting including permit applications, permits, routing data, permitting codes and restrictions, and other system management information. APRAS contains a representation of the RMS roadway network used for entering route information and for route analysis and generation. Selected bridge data from the BMS2 system for all structures carrying or over State Routes is in the APRAS database. It also contains the program logic representing the business rules to validate and process permits. The central database design ensures consistent, manageable application of APRAS business rules.

- **Route Prompting** is the mechanism by which permit route information is entered into APRAS. Route Prompting ensures that all routes on the application are based upon RMS roadway data and ensures their connectivity. It also provides the basic data structures which are used in the Route Analysis, tying them to RMS roadway data, and BMS2 bridge data (vertical and horizontal clearance envelope and live load capacity).

- **Route Analysis** ensures that a permit vehicle can travel safely over the route specified by comparing the vehicle dimensions and load data against the roadway and bridge information from RMS and BMS2. Route Analysis checks for roadway clearances and special restrictions on roadways (e.g., Posted and Bonded roads). It also identifies the bridges along the specified route and performs oversize and overweight checks for the bridge structures. The structural check of bridges is performed by the ABAS subsystem.

  Route Analysis is an important step in the approval process for a permit application. Once a permit has passed all administrative validation rules, it must then pass Route Analysis before it can be approved. If the vehicle load can safely pass Route Analysis, a permit can automatically be issued by APRAS. If problems in the route are encountered or if certain conditions require further review, the application is posted for manual review by Department permitting and bridge staff.

- **Route Generation**'s function is to find the optimal route for an oversize and/or overweight vehicle from a source point to a destination point, without traveling over any roads or bridges that cannot handle the load. This allows permit staff to choose an approved, system- generated alternative route for the permit vehicle.

- **External Systems - RMS, BMS2, SAP, CARATS, EngMgr:** APRAS depends on data from several Department systems for its validation of permit applications.

  - **RMS** is the source for all roadway information, except some information regarding canned routes, turn restrictions, etc.
  - **BMS2** is the source for all bridge related clearance and capacity information.
  - **CARATS** is the source for vehicle registrations.
  - **EngMgr** is the Engineering Dataset Manager and stores the engineering datasets used by ABAS.

Information from these systems is refreshed within APRAS every night. Since APRAS is very dependent on RMS and BMS2 data, the relationship between permitting and maintenance of RMS and BMS2 is critical. APRAS relies upon the data in these systems being consistent and accurate. Any inconsistencies in RMS or BMS2 data detected through APRAS must be modified within RMS or BMS2. APRAS also interfaces with the **SAP** system for its financial transactions.

### 10.5.1 APRAS Related Data in BMS2

CODING INSTRUCTIONS: Data from the BMS2 that is needed for APRAS is contained on BMS2 Screens SL, SS and SC. For coding instructions, see applicable sections of Publication 100A.

STRUCTURES REQUIRING APRAS DATA:

- Bridge Capacity Data: All bridges or structures greater than 20 feet in length that carry vehicular traffic on State Routes, regardless of structure ownership, are to have sufficient load capacity data to allow APRAS to perform structure reviews. This information may include capacity rating factors and engineering datasets for analysis/rating software.

- Restricted **Vertical Clearance:** All bridges or structures over State Routes with a vertical clearance less than 16'-0", regardless of structure ownership, are to have vertical clearance data for the State Route in APRAS. This includes truss bridges with overhead members, sign structures over pavement or shoulder area of roadway, overhead utility bridges, etc. See Appendix IP 10-C for the APRAS Vertical Clearance Flowchart.

- Reduced **Horizontal Clearance:** All bridges or structures over 8 feet in length that carry State Routes, regardless of ownership that have a face to face barrier distance of 16'-0" or less are to have horizontal clearance data. Consideration should also be given to structures adjacent to State Routes and encroach upon the horizontal clearance from edge of pavement (e.g., retaining walls), regardless of structure ownership.

RESPONSIBILITY FOR APRAS DATA: The District Bridge Unit is responsible to maintain sufficient clearance information and/or live load capacity information for the structures requiring APRAS data as outlined above to enable APRAS to perform the Route Analysis for a permit load on State Routes, regardless of structure ownership. This may require the monitoring of the clearance envelope by District staff for non-Department structures over State Routes. For LOBSTORS or other non-Department bridges carrying State Routes, the Districts are to provide information not available from the bridge owner, if the District is authorized to grant permits. Also see IP 10.3.1.

### 10.5.2 Automated Bridge Analysis System - ABAS

The APRAS subsystem that performs the load evaluation of bridges under permit vehicles is called ABAS – Automated Bridge Analysis System. ABAS was designed to replicate the manual bridge review process used by the District Bridge Units before APRAS was available.

Basically, that manual permit review process for bridges had two components:

- Checking the bridge's load capacity rating factor for HS vehicle against the maximum load effect of the permit vehicle as a ratio of the HS vehicle.
- Using a bridge analysis program to check the actual permit vehicle

Generally, the capacity comparison method was used for bridges for which an analysis was not readily available (e.g., complex bridges, bridges rated by engineering judgment) or as a threshold value to minimize computational efforts.

The BMS2 APRAS data allows the District to use either the capacity comparison method and/or the direct analysis method. For the direct analysis method, engineering datasets for the input data to the analysis and rating software must be established in bridge rating library on the Department's Dataset Manager. If the bridge fails during the capacity comparison method, APRAS automatically runs the analysis programs using the input dataset and the axle weight/spacing data from the permit application. If the bridge fails the analysis program, the application is sent to the Bridge Unit for a manual review.

The District Bridge Engineer may determine that a manual review is to be required for all permits on a bridge and an indicator is placed on the SC screen that instructs APRAS to analyze the bridge, but not approve it. For manual reviews, the bridge reviewer must approve each permit individually. All permit applications that pass all analyses and have the Bridge Unit's approval on manual reviews are approved for the load capacity evaluation. Appendix IP 10-B contains an abbreviated flowchart of the ABAS load evaluation for simple spans to demonstrate the basic concepts used.

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# APPENDIX IP 01-A

Bridge Safety Inspection Program
Strategic Plan

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# Mission Statement

To ensure the public safety and efficiency of the bridge system throughout Pennsylvania by advancing bridge inspection policies and procedures, bridge management practices, emerging technologies, and partnerships with business partners.

## Goals

To fulfill the mission, four goals with objectives and supporting strategies are defined as follows:

- Ensure all highway bridges are safe
- Support principles of Bridge Management
- Use bridge inspection technologies and equipment to supplement visual bridge inspections
- Strive for continual improvement

Goal: Ensure all highway bridges are safe.

1. **Objective:** Defined policy & procedures

   Strategies

   a. Defined inspection roles and responsibilities
   b. Defined critical deficiencies
   c. Defined inspection types
   d. Defined inspection methods
   e. Defined inspection intervals
   f. Defined inspection report
   g. Defined QC/QA program

2. **Objective:** Safety Inspections of Bridges

   Strategies

   a. Compliance with National Bridge Inspection Standards and PennDOT publications
   b. Proper inspection planning
   c. Thorough inspection and reporting
   d. Conduct NSTM inspections
   e. Conduct underwater inspections
   f. Provide consultant resources to Districts
   g. Provide consultant resources to local governments

3. **Objective:** Load Rating of Bridges

   Strategies

   a. Proper identification of need to re-evaluate load ratings
   b. Proper documentation of load ratings
   c. Provide Engineering tools
   d. Standardized Posting Evaluation Process
   e. Evaluate new software
   f. Correct problems in existing PennDOT software
   g. Support of APRAS -Evaluation of overweight permit vehicles
   h. Provide support to Districts for super-loads

4. **Objective:** Critical deficiencies

   Strategies

   a. Proper discovery and prioritization
   b. Develop Mitigation Measures
   c. Develop Structural Repairs

Goal: Support principles of Bridge Asset Management.

1. **Objective:** Data Management
   Strategies
   a. Inventory elements
   b. Conduct element level inspections for all State-owned bridges
   c. Ensure data quality
   d. Ensure data integrity
   e. Perform data analysis
   f. Develop data retrieval and reports

2. **Objective:** Information Intelligence
   Strategies
   a. Ensure accurate data/information for executive management decision making
   b. Align with Commonwealth and Department IT initiatives
   c. Develop data performance metrics

3. **Objective:** Bridge Planning for maintenance
   Strategies
   a. Establish systematic preservation program
   b. Address on-demand structural maintenance
   c. Conduct annual preventive maintenance
   d. Establish needs-based budgets or set-asides as function of assigned priority and/or cost-benefit
   e. Evaluate new materials and technologies and their effectives for bridge maintenance
   f. Provide the Districts an annual summary of the data included in the preservation program including maintenance recommendations

Goal: Use new bridge inspection technologies and equipment to supplement visual bridge inspections.

1. **Objective:** Structural Response

   Strategies
   a. Use appropriate technology to assess cause to develop structural repair
   b. Evaluate technology through State and National research programs
   c. Coordinate with FHWA Turner-Fairbank Research Center
   d. Participate on task forces to develop national policy
   e. Partner with universities and private businesses to use appropriate technology

2. **Objective:** Detecting Loss of Strength (Section Loss, Structural Cracking, Fatigue, Bearing)

   Strategies
   a. Use of non-destructive techniques and minimally invasion techniques
   b. Use of measurement devices
   c. Evaluate technology through State and National research programs
   d. Coordination with FHWA Turner-Fairbank Research Center
   e. Participate on task forces to develop national policy
   f. Partner with universities and private businesses to use appropriate technology

3. **Objective:** Standardized/consistent equipment

   Strategies
   a. Ensure bridge safety field inspectors use appropriate inspection and access equipment
   b. Use appropriate digital technology – mobile office
   c. Evaluate newly developed access equipment
   d. Evaluate technology through State and National research programs
   e. Coordination with FHWA Turner-Fairbank Research Center

Goal: Strive for continual improvement.

1. **Objective:** Training

   Strategies

   a. Review and update inspection and evaluation training courses annually to account for new bridge safety issues.
   b. Ensure the "take-aways" from the training are clear and concise
   c. Ensure Districts have regular meetings with inspection staff to discuss key issues
   d. Ensure team leaders and review engineers have adequate training and experience
   e. Support career path for bridge inspectors
   f. Support mentoring

2. **Objective:** Quality Control and Quality Assurance

   Strategies

   a. Develop QC and QA metrics to validate and rate program performance
   b. Ensure each District bridge inspection unit has a defined Quality Control program that is complied with.
   c. Establish QA program at the District level.
   d. Perform internal and external audits of the District's and State's bridge inspection program to ensure compliance, as necessary.
   e. Continue to perform Statewide QA bridge inspection program to assess compliance with established standards and to identify areas for improvement.
   f. Review consultant QC/QA plans

3. **Objective:** Improve data collection and reporting processes

   Strategies

   a. Improve quality of inspection documentation
   b. Further implement the electronic document management system named Enterprise Content Services (ECS)
   c. Evaluate and implement inspection reporting technologies.

4. **Objective:** Responsive Organization

   Strategies

   a. Respond to State and National bridge issues in timely manner
   b. Conduct forensic investigations
   c. Respond to natural disasters affecting bridges in timely manner, Mobilize Department forces rapidly

5. **Objective:** Communication

   Strategies

   a. Maintain constant lines of communication between Districts and the Bureau of Bridge
   b. Maintain constant lines of communication between District Bridge units and District Maintenance operations
   c. Effectively communicate the Department's bridge inspection goals and policies to other Bureaus, other bridge owners and business partners
   d. Address public concerns

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# APPENDIX IP 01-B

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# APPENDIX IP 01-C

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# APPENDIX IP 01-D

## Local NBIS Inspection Notification Letter

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pennsylvania
DEPARTMENT OF TRANSPORTATION

**DATE:**

**SUBJECT:** Inspection Notification – National Bridge Inspection Standards

**TO:** **LOCAL BRIDGE OWNER - TO BE COMPLETED BY DISTRICT**

**FROM:** **TO BE COMPLETED BY DISTRICT**

This letter addresses bridge safety inspection regarding:

- Critical Structural Deficiencies
- Scour Critical Bridges and Scour Plans of Action
- Notification of Inspection Due Dates
- Inspection Agreements with PennDOT
- Responsibility for Inspection Interval Compliance
- Bridge Safety Inspection Reporting and Scheduling
- Bridge Posting Requirements
- Bridge Closure Reporting
- Inspection Requirements for Complex and NSTM Bridges
- List of Bridges to be inspected

**Critical Structural Deficiencies**

When a critical structural deficiency that threatens to compromise public safety is identified, bridge owners are obligated to act expeditiously either to repair that deficiency or to take other actions, such as restricting or closing the bridge. The bridge inspector will identify the critical deficiency and the required actions in the inspection report and/or in a separate critical deficiency letter. In addition, the inspector is required to develop a Plan of Action (POA) to address the critical structural deficiency and schedule a meeting with the bridge owner to present details of the POA to the bridge owner. This meeting is to be held within three calendar days and must be attended by appropriate officials from **DISTRICT TO INSERT MUNICIPALITY NAME.** Immediate action may be required to mitigate any critical deficiency posing a danger to the public. Such action should not be deferred pending results of the meeting. The meeting should not be adjourned until agreement has been reached regarding specific actions to be taken, as well as a schedule for implementation. Failure to take appropriate actions for public safety may result in the Department's restricting or closing the bridge. The non-federal share of the cost incurred by the Department in undertaking such restrictions may be deducted from the local bridge owner's liquid fuels allocation in the future, pursuant to the statutory authority described in the section entitled "Notification of Inspection Due Dates".

**<u>Scour Critical Bridges and Scour Plans of Action</u>**

As required by the 2005 National Bridge Inspection Standards (NBIS), Section 650.313 (e) (3), bridge owners are required to develop a Scour Plan of Action (POA) for bridges in Scour Categories A, B, C, and D. This plan describes the required monitoring procedures and documents known critical deficiencies, as well as all needed scour protection measures for scour critical bridges. Scour critical bridges have been identified by Categories A, B and C based on the scour vulnerability of each bridge with a Category A being the most vulnerable to scour. A Scour POA document includes sections for a monitoring program and a Bridge Closure Plan when flooding of bridges is reported or observed. A Scour POA has information specific to **DISTRICT TO INSERT MUNICIPALITY NAME** including responsibilities for monitoring, closure, and contact information. The Scour POA also contains other pertinent bridge inventory data, previous scour inspection data, and completed and recommended maintenance data related to scour.

The Scour POA serves two purposes:

1. Establishes a systematic process of monitoring and closing bridges to ensure public safety during a significant flood event and criteria for re-opening and inspection after a significant flood event.
2. Assists bridge owners to program and prioritize the installation of scour countermeasures to protect scour critical bridges from flood damage.

Bridge owners must coordinate with their District Municipal Services representative to ensure that all scour critical bridges are identified, and Scour POA documents have been updated with completed sections for Monitoring Program, Bridge Closure Plan and current contact information. For informational materials pertaining to monitoring scour critical bridges during flood events, visit PennDOT's website for Local Scour Critical Bridges located at:

https://www.pa.gov/agencies/penndot/programs-and-doing-business/bridges/local-scour-critical-bridges.html

The current scour critical bridge list specific to **DISTRICT TO INSERT MUNICIPALITY NAME** is also enclosed. Maps showing the location and category of county, and municipal scour critical bridges may be accessed by going to the PennDOT Local Scour Critical Bridge Map website located at:

https://padotgis.maps.arcgis.com/apps/webappviewer/index.html?id=0f62b3249c12447082f5b3151eadfeaf

The website assists local bridge owners to identify, locate, and confirm ownership of scour critical bridges. The map is updated on a monthly basis because the scour critical category of a bridge may change based on bridge inspection findings. Clicking once on the map symbol for a bridge will provide a pop-up information box. The bridge-specific blank monitoring log can be downloaded from the link inside the pop-up box. The log, in portable document format (PDF), includes basic information for the bridge and general monitoring instructions. The monitoring log provides both a means for recording monitoring activities and is also a decision tool for determining when to close a bridge for safety. Please note that these completed monitoring logs must be filled out by the local bridge owners and kept on file for later review, if requested by FHWA.

As previously noted, bridge owners need to be aware that the Scour Critical status of bridges can change based on field conditions observed during the biennial bridge inspection. Bridge inspection files may require periodic updates to the Scour POA, or a new Scour POA must be developed for a bridge when inspection findings identify the bridge as Scour Critical. Owners should be aware that periodic updates of these Scour POA's may be required as scour conditions at the bridge change.

**Notification of Inspection Due Dates**

The majority of locally owned bridges in Pennsylvania are inspected on time. Municipalities and counties have formed a cooperative working relationship with the Pennsylvania Department of Transportation (PennDOT) to ensure that timely and accurate inspections occur. PennDOT, in its oversight capacity, is attempting to be more proactive in communicating with local bridge owners during the bridge inspection process. The enclosed list of bridges in **DISTRICT TO INSERT MUNICIPALITY NAME** indicates inspection due dates between April **YEAR** and March **YEAR**. The process of sending this annual letter serves as a reminder to local bridge owners, county planning agencies, engineering consultants, and PennDOT staff, in an effort to prevent bridge inspections from becoming past due.

In the event that a bridge inspection does become past due, PennDOT intends to conduct inspections on locally owned bridges during the month in which the inspection is due or within fifteen (15) days if the bridge becomes past due at the end of the month. The Department's authority to carry out the inspections is stated in Section 2002(a)(19) of the Administrative Code of 1929, as amended by Act 44 of 1988, 71 P.S. § 512(a)(19). PennDOT is issuing this advance written notice to all local bridge owners in order to adhere to a statutory requirement for a sixty- (60) day notification. PennDOT is also authorized by Act 44 of 1988 to deduct the non-reimbursed bridge inspection costs from liquid fuels allocations. This statutory authorization is found in Section 2001.5 of the Administrative Code of 1929, as amended, 71 P.S. § 511.5, which was added by Act 44 of 1988.

**Inspection Agreements with PennDOT**

PennDOT enters into agreements for bridge inspection with individual municipalities or with multiple municipalities under an agreement through a county or regional government body. Please confirm that the listed bridges are covered under a current agreement. If an agreement has expired, or will expire prior to the inspection due dates, please contact **INSERT NAME AND PHONE NUMBER OF LOCAL BRIDGE INSPECTION COORDINATOR.** If a pending agreement has been drafted, but not yet executed, please contact **INSERT NAME OF LOCAL BRIDGE INSPECTION COORDINATOR** to ensure that the agreements can be executed in sufficient time to conduct the bridge inspections.

Local municipal bridge owners may contract individually with a qualified engineering consultant firm or use a PennDOT countywide or district-wide (umbrella) consultant engineering agreement for municipal bridges. Regardless of whether the inspection is performed with in-house staff or from engineering consultants, all personnel performing the inspection must meet NBIS and PennDOT qualifications.

**Option 1 – Owner utilizes PennDOT hired consultants:**

- Using PennDOT-hired consultants in ECMS will save effort and some cost since:
    - PennDOT manages the inspections thus avoiding the local owner's administrative staff effort in reviewing statements of interest, selecting and performing the contractual execution for the consultant inspection firm(s).
    - No initial inspection cost incurred by the local owner, thus no administrative effort required to review invoices submitted by inspection consultant. PennDOT handles the reimbursement requirements to the consultant firms through ECMS.
    - PennDOT will automatically withhold the non-reimbursable local share (20%) from the municipality's liquid fuels allocations in the fiscal year following the inspection. This will benefit the local owner in one, not having to budget 100% of the costs upfront and seeking the 80% reimbursement along with associated administrative efforts and costs and two, the 20% comes out of the following fiscal year prior to issuing the total liquid fuels amount for that municipality.

**Option 2 -Owner contracts individually with a qualified engineering consultant firm:**

- Per PennDOT's Local Project Delivery Manual (PennDOT Publication 740) all new third-party engineering agreements for local projects are required to use ECMS. The use of ECMS for consultant selection and executing engineering agreements should result in time and cost savings for the local project sponsors.
- The procedure is as follows:
    - Owner hires a qualified engineering firm through a qualification-based selection process and enters into an Engineering Agreement. The owner is responsible for management of the agreement.
    - Owner pays 100% of the cost of the inspections to the engineering firm directly.
    - Owner is reimbursed 80% of the costs through a Reimbursement Agreement with PennDOT.

**Responsibility for Inspection Interval Compliance**

The following bridge inspection program activities are assigned in PennDOT's Publication 238, Bridge Safety Inspection Manual:

- Federal Highway Administration (FHWA) – Verify the National Bridge Inspection Standard (NBIS) compliance for all States.
- The Bridge Inspection Section in PennDOT's Bureau of Bridge – Collect and compile all bridge inventory and inspection data for all public roads in Pennsylvania.

- PennDOT Engineering Districts – NBIS compliance for all bridges in the District's jurisdiction.

- Local Bridge Owner – Inspecting and rating of all bridges (by in-house staff or by consultant). Provide inspection reports to PennDOT.

Collectively, all parties have a role to make certain that timely inspections occur for all highway bridges in Pennsylvania. NBIS is set forth in the 23 CFR Part 650 which states in §650.311 Inspection interval:

> *(a) Routine inspections. (1) Method 1. (i) Regular intervals. Each bridge must be inspected at regular intervals not to exceed 24 months, except as required in paragraph(a)(1)(ii) of this section and allowed in paragraphs (a)(1)(iii) of this section.*

Typically, inspections for individual bridges occur within the same calendar month for each inspection cycle. The field inspection for a bridge is considered to be past due if it has not been completed by the last day of the scheduled calendar month. This also applies to bridges that are inspected more frequently than the twenty-four month cycle.

**<u>Bridge Safety Inspection Reporting and Scheduling</u>**

Inspections must be accompanied by timely reports submitted to the Department for review. PennDOT's Publication 238, Bridge Safety Inspection Manual, requires that draft inspection reports are due within 30 days after the field inspection has been completed. Particular emphasis needs to be placed on submission of the draft inspection report to facilitate review and comment prior to acceptance of the final report. This ensures the Department's Bridge Management System (BMS2) is updated within timeframes specified by NBIS.

The current inspection date, interval, and scheduling for each bridge can be found in BMS2; it is accessible via the internet to local bridge owners and/or their consultants who have registered as a Business Partner with PennDOT. This specific information is found on the Ratings & Schedule link in fields 7A01 and 7A09, respectively. The information in field 7A10, Next Dt - Next Inspection Date, does not automatically update; therefore, the "Next Inspection Date Calculation" button must be clicked in order to update the information. In order for the date to calculate correctly, the interval located in 7A09 must be verified for accuracy.

**<u>Bridge Posting Requirements</u>**

PennDOT is required by the Pennsylvania Consolidated Statutes Title 75 Vehicles, Chapter 49 Size, Weight and Load Restrictions and Act 44 of 1988, 71 P.S. § 512(a)(19) to ensure compliance of all Municipal and County owned bridges with regards to bridge load postings.

Once a bridge load posting has been recommended, the bridge owner is to implement the posting in accordance with PennDOT Department Policy. The bridge owner shall notify the local PennDOT District Bridge Inspection Staff once the load posting has been implemented.

Also, all pertinent BMS2 data fields shall be updated accordingly by the bridge owner or their inspection engineer.

If it is determined that the bridge load posting has not been implemented according to Department Policy, PennDOT shall notify in writing the bridge owner immediately that the Department intends to implement a bridge posting. The Department shall invoice the bridge owner for all non-reimbursed costs associated with the bridge posting implementation.

**Bridge Closure Reporting**

After notifying the internal agency staff upon closure of a bridge, bridge owner should notify PennDOT Local District Office Bridge Inspection Section staff regardless of reason for closure after the closure is safely and securely in place. The notification should include the Bridge Management System (BMS) number or bridge name and the reason for closure. This notification coincides with Bridge Inspection Section's efforts to gather the most up to date information for all of the bridges meeting the NBIS requirements and will allow for more efficient planning of bridge inspection resources as may arise.

**Inspection Requirements for Complex and NSTM Bridges**

All bridges with complex features and NSTM bridges shall have written specialized inspection procedures in a stand-alone document filed in the bridge file in BMS2. The written inspection procedures shall include requirements for the NBIS Certified Team Leader to be present at all times during the inspection; details on the scope and interval of Routine, NSTM, and In-Depth inspections; inspection procedures for mechanical and electrical components for moveable bridges; and inspection procedures for any other unique or complex portions of the structure. The written procedures for NSTM bridges shall be spelled out within the Fatigue and Fracture (F&F) Plan. See Publication 238, Section IP 2.4.5.1 for requirements for the F&F Plan.

**List of Bridges to be Inspected**

As discussed above, an enclosed list indicates the bridges within **DISTRICT TO INSERT MUNICIPALITY OR COUNTY NAME** and their respective inspection due dates. A second enclosed list of Scour Critical bridges identifies the Scour Critical category for each structure and indicates whether or not a Scour POA is on file. A copy of this letter and bridge lists are also being forwarded to **DISTRICT TO INSERT APPLICABLE COUNTY/REGION PLANNING AGENCY** and/or **LOCAL BRIDGE CONSULTING ENGINEER.**

Should you require any additional information, please contact, **TO BE COMPLETED BY DISTRICTS.**

Encl: List of Bridges to be Inspected
Scour Critical Bridge List

cc: County/Region Planning Agency (as applicable)
Designated Consulting Engineer (as applicable)
District Municipal Services Supervisor

# Attachment – List of Bridges to be Inspected

## Attachment – Scour Critical Bridge List

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# APPENDIX IP 01-E

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# APPENDIX IP 01-F

## General Scope of Work for Consultant Agreements – Safety Inspection of State and Local Bridges

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**Description:** Safety Inspection of State and Local Bridges

**Objective:** Inspect, load rate, inventory and appraise all types of structures and perform follow-up work as directed.

**Statement of Work:** The requirements of the latest versions of the Department accepted AASHTO and FHWA manuals and the latest versions of Department Publications and Policy, including any updates, shall be followed in the performance of the Scope of Work. See the Department's Bridge Safety Inspection Manual, Publication 238, IP 1.3.2 and IP 1.3.3 for a list.

**Scope:** The scope of work will include the following activities:

**I. TYPES OF SAFETY INSPECTION WORK**

**A. Initial Inspection** - Insufficient or no data is available in BMS2. An inspection fulfilling NBIS requirements has never been performed. For bridges carrying highway traffic, a separate Bridge Load Rating work item must also be completed and its results incorporated into this initial inspection report (see Scope Section II.E., "Bridge Load Rating"). Additional inspection requirements for an Initial Inspection are outlined in Publication 238, IP 2.3.1.

State-Owned Bridges and Locally-Owned Bridges on the National Highway System (NHS) – An AASHTO / PennDOT bridge element inventory and assessment for bridges has never been performed. Identify and inventory elements and calculate element quantities and scale factors. Identify and record the defect code(s) and condition state quantities for the inventoried elements.

**B. Routine Inspection** - An NBIS Inspection has been previously completed within the last two (2) years (or 4 years if approved for extended interval) and that inspection report and/or documentation is available. Additional inspection requirements for a Routine Inspection are outlined in Publication 238, IP 2.3.2.

State-owned Bridges and Locally-Owned Bridges on the National Highway System (NHS) - The AASHTO / PennDOT bridge element inventory and assessment for the bridge has been previously completed and documentation is available. Identify and inventory elements and calculate element quantities and scale factors. Identify and record the defect code(s) and condition state quantities for the inventoried elements.

**C. Special Inspection** - An NBIS Inspection has been previously completed. The structure is included in the BMS2 and the previous inspection report is available. Perform an inspection that is usually limited to portion(s) of the structure which require a reduced interval inspection. Specific inspection requirements are outlined in Publication 238, IP 2.3.5. The scope of work for a Special Inspection must be approved by the District Bridge Engineer prior to initiating work.

State-owned Bridges - The AASHTO / PennDOT bridge element inventory and assessment for the bridge has been previously completed and documentation is available. Identify and inventory elements and calculate element quantities and scale factors. Identify and record the defect code(s) and condition state quantities for the inventoried elements.

**D. Supplemental Inspection** - Perform in-depth work beyond the scope of Routine inspections, focusing on a specific area or the entire structure (as in an In-Depth Inspections as outlined in Publication 238, IP 2.3.4) or specific components (as in a Special Inspections as outlined in Publication 238, IP 2.3.5). In-depth tasks may include the following:

- Non-Destructive Testing (except dye penetrant),
- Laboratory Analysis,
- Geotechnical sampling and testing,
- Structure instrumentation, and/or

- Underwater inspection.

The scope of work authorizing a Supplemental Inspection should have provisions for these tasks identified in the LIST OF SPECIAL REQUIREMENTS. The scope of work for an In-Depth or Special type of inspection must be approved by the District Bridge Engineer prior to initiating work.

**E. Bridge Load Rating** - Perform a structural analysis and load rating of the structure to determine its ability to carry PA's legal loads.

**F. Critical Deficiency Meetings** - Coordinate and conduct a meeting with local bridge owners to discuss critical structure deficiencies found during the recent inspections.

## II. INSPECTION REQUIREMENTS

### A. Initial Inventory and Inspection

1. Conduct a complete inventory and field inspection utilizing BMS3.
2. Complete BMS2 Inventory data items via BMS2 and BMS3 Inspection Pages.
3. If structure carries highway traffic, incorporate the Bridge Load Rating performed under separate work item into the initial inspection report. Evaluate bridge for posting needs.
4. Prepare an Inspection Report.

For all State-owned Bridges and Locally-Owned Bridges on the National Highway System (NHS):

5. Identify AASHTO / PennDOT Bridge elements.
6. Calculate element quantities and scale factors.
7. Prepare an element summary table for all elements and provide supporting calculation for each element.
8. Identify and record defect code(s) and condition states for all elements.

### B. Routine Inspection

1. All bridges, except closed bridges.
   a. Conduct a complete field inspection utilizing BMS3.
   b. Update/supplement the evaluation for posting needs for the structure's current condition. Determine if re-rating is warranted by comparing new vs. existing section loss measurements. If structure is to be re-rated, use the new load rating summary.
   c. Update/amend the Inspection File providing new photographic documentation and/or sketches as needed.
   d. Update and/or complete the required minimum BMS2 inventory and inspection items via BMS2. See Scope Section III.C., "Minimum Required Inventory and Inspection Data," for minimum BMS2 items required.
   e. Incorporate the results of previous or new load ratings into the report.
   f. Prepare an Inspection Report to document all work and findings.

For all State-Owned Bridges and Locally-Owned Bridges on the NHS:

   g. Identify and record defect codes and condition states for all elements.
   h. Update elements and/or quantities based on changed field conditions.

2. Closed Bridges

   a. Inspect bridges closed to highway traffic to assure that the physical barriers are maintained and that the public safety is not jeopardized. Assess the physical integrity of the structure and any potential hazards to the public on or beneath the structure, especially if pedestrian use is to be allowed.

   b. Use PennDOT BMS3 Inspection Pages for field notes. Include a minimum of two (2) photos showing bridge with in-place barriers.

   c. Prepare an Inspection Report to document all findings.

3. Partially Closed Bridges

   a. Inspect the open portions of bridges partially closed for staged construction as outlined in Scope Section II.B.1.

   b. Prepare an Inspection Report to document all findings.

**C. Special Inspection**

1. Inspect the specified portion(s) of the structure as authorized by the District Bridge Engineer. Use PennDOT BMS3 Inspection Pages.
2. Update/supplement the posting evaluation of the portion inspected.
3. Update/amend the portion of the Inspection Report dealing with the portion inspected.
4. Update and/or complete the required minimum BMS2 inventory and inspection items via BMS2 relevant to the portion inspected.
5. Prepare an Inspection Report to document all work and findings.

**D. Supplemental Inspection**

1. Conduct inspection of structure as directed by the Department. Use PennDOT BMS3 Inspection Pages.
2. Perform follow-up sampling and testing as specified.
3. Update/amend the portion of the Inspection Report dealing with the portion inspected.
4. Update and/or complete the required minimum BMS2 inventory and inspection items via BMS2 dealing with the Supplemental Inspection.
5. Prepare an Inspection Report to document all work and findings.

**E. Bridge Load Rating**

1. Perform or update the structural analysis and load ratings for all PA legal loads using the latest specification and programs.
2. Identify the structural components or members that govern the ratings.
3. Prepare a load rating summary table and/or stress table for the Inspection Report. Reference calculation page number for ratings.
4. When appropriate update ratings directly in BMS2.

**F. Critical Deficiency Meetings**

1. Arrange and conduct a meeting with local bridge owners to discuss critical deficiencies found during the inspection.

2. Prepare informal meeting minutes and supply copy of the minutes to the District and municipality.

## III. BMS2 INVENTORY AND INSPECTION DATA

**A.** **Department Structures** - Complete and/or update all applicable data items of the BMS2 D-491 Forms printout unless otherwise instructed. Complete new forms for new bridges.

**B.** **Local Government Bridges and Others** - Provide complete data as in the Scope Section III.A., unless otherwise directed to provide only minimum data.

**C.** **Minimum Required Inventory and Inspection Data -** Minimum inventory and inspection data includes all BMS2 Items identified with an asterisk in BMS2 Coding Manual, Publication 100A, and the following BMS2 Items where applicable or attainable:

| | |
|---|---|
| 5C12 | Future ADT |
| 5C13 | Future ADT Year |
| 5C30 | School Bus Route |
| 5C32 | Transit Bus Route |
| 6A04 | Co Municipality Boundary Code |
| 6A38 | Bridge Deck Type |
| 6A43 | Approach Pavement Width |
| 6A44 | NSTM Group # |
| 6A45-6A48 | Critical Ranking Factor of NSTM |
| 6A53 | Cum Truck Traffic for Fatigue Damage |
| 6B24 | Agency Hiring Consultant |
| 6B26 | Inspection Crew Hours |
| 6B27 | Crane Hours |
| 6B32-6B34 | Inspection Cost |
| 6B36 6B37 | Paint Condition |
| 6B38 | Approach Slab Condition |
| 6B39 | Approach Roadway Condition |
| 6B40 | Deck Wearing Surface Condition |
| 6C10 | Highway System |
| 6C11 | State Network |
| 6C35-6C38 | Vertical Clearance Signing |
| 7A03 | Type of Inspection |
| 7A05 | Consultant Name, Inspection By |

| | |
|---|---|
| 7A09 | Inspection Interval |
| 7A14 | Next Inspection By |
| VP01 | Status Date |
| VP03 | Special Restrictive Posting |
| VP04 | Posted Weight Limits |
| VP06 | Reason for Posting or Closing |
| VD14 | Abutment Type |
| VD15 | Abutment Foundation Type |
| VD16 | Pier Type |
| VD17 | Pier Foundation Type |
| VD19 | Length of Culvert |
| IR06 | Rating Method |
| IR07 | IR Controlling Member |
| IR08 | Fatigue Category Controlling Member |
| IR09 | Fatigue Controlling Load Type |
| IR14 | AASHTO Manual Year |
| IR15 | AASHTO Spec Year |
| IR18 | Fatigue Stress Range |
| IF | Items for FCMs IF01-IF06 |
| IN, IU | All IN and IU items |
| IM | Items IM01 - IM15 (except IM10) |
| FR03 | Service Status of Railroads |
| FR04 | Railroad Mile Post |
| FR05 | AAR Number |
| FR06 | Number of Electrified Tracks |
| FW01 | Name of Stream |
| SP01 | Span Type |
| SP02 | Label |
| SP03 | Span Length |
| SP04 | Span Deck Width |
| 6B48 | Combustible Materials Under the Bridge 1B |
| | Inspection Element Detail Fields |

Note: Only applicable items need to be coded. All submitted data will be stored in BMS2. Owners are encouraged to collect and submit all inventory and inspection information available.

## IV. FIELD INSPECTION AND ASSESSMENTS

**A.** Completely inspect all bridge elements including the foundations that support the substructure elements. Clean members as needed to assess condition. For a Special Inspection, inspect only the specified areas/members. However, report any public safety threatening deficiencies that are observed elsewhere on the bridge. Include inspection of any sign structures attached to the bridge.

**B.** Clearly record all inspection field notes in BMS3. Provide sufficient written comments within BMS3 to outline the bridge's condition and to justify all condition and appraisal ratings.

Precisely locate and describe deterioration and all areas of section loss. Perform dye penetrant testing if cracking is suspected or found. Determine if current conditions warrant a re-rating for load capacity. Determine if current load posting status is appropriate. Prepare sketches and obtain photographic documentation.

**C.** Inspect all substructure units and culverts (e.g. abutments, piers, footings, etc.) and culverts visually or by feel (e.g. probing) for condition, scour, integrity, safe load capacity, etc. Use BMS3 Inspection Pages to record findings.

Conduct evaluation of the site and structure to determine the risk from scour. Investigate the scour potential and determine structure stability. Determine channel condition and waterway adequacy. Update scour assessment as warranted. Propose countermeasures appropriate for conditions. Determine the need for an underwater inspection by a professional diver and record reasons in the Recommendation section of the report.

Provide/update plan view sketch of bridge and stream to denote channel changes, scour deposition, etc. Provide/update waterway opening sketch (cross section) to denote bottom of stream, superstructure and substructure units. Measurements from permanent marker should be in table form to compare with previous inspections.

**D.** Identify locations and provide description of NSTMs and fatigue prone details. Use BMS, NSTM/Fatigue page to record findings. Discuss future inspection interval and procedures for these FCMs. Perform hands-on inspection of all FCM's as indicated in the F&F Plan.

**E.** Identify and record all maintenance and major improvement needs utilizing BMS3 Maintenance page.

**F.** For State-Owned Bridges and Locally-Owned Bridges on the NHS conduct a complete field AASHTO / PennDOT Bridge element assessment utilizing BMS3 Elements page. Identify and record condition states for all elements and defect codes. Provide sufficient comments within BMS3 to describe the element's condition and corresponding location to justify all condition states. Update elements and/or element quantities based on changed field conditions.

**G.** Provide emergency retrofit schemes, as directed, to any critical conditions uncovered.

**H.** Arrange for rigging, inspection cranes, platform lift trucks, ladders, boats, etc. The use of safety boats or skiffs should be considered when working over water and the risk of falling is high. Arrange for any needed Traffic Control. Ensure the safety of inspectors and public at all times. Identify these access needs on the Inspection Planning Screen in BMS2.

**I.** For highway bridges over railroads, coordinate with the railroad to arrange access for inspection of portions of bridges affected by railroad electrification and for railroad protective services while working in the track area and as required by the railroad. Obtain necessary permits and insurance.

## V. STRUCTURAL ANALYSIS, LOAD RATING, AND POSTING EVALUATION

**A.** Acquire authorization from the District Bridge Engineer prior to updating or performing a structural analysis or load rating.

**B.** Perform or update the structural analysis and load ratings using Load Factor methodology where applicable. Where Load Factor is not applicable, rate the bridge using a method acceptable to AASHTO and the Department. Load rate all bridges at Inventory and Operating levels for AASHTO H, AASHTO HS, PA's ML-80, and PA's TK527 vehicle configurations.

**C.** Use conventional methods of analysis unless more complex and refined methods are specified or warranted and specifically authorized by the Department. (Refer to Publication 238, IP 3).

**D.** Identify the structural components or members that govern the ratings. Define any section losses and/or other deficiencies on these members. Provide or reference typical cross-sections and/or framing plans. Include a table of stresses and a rating summary in the report. Reference calculation page number for values in rating summary.

**E.** Calculate the load ratings using data available from inspection files and reports, supplemental field information and testing data. When no data or drawings (or sketches) are available, field measure members and document findings on a sketch for use in calculating load ratings. The sketch should include enough information to complete the rating analysis including typical cross section with dimensions, remaining section of components (where loss is noted), and locations of deterioration. Sketches shall clearly label elements and members.

**F.** Ensure that all computations are in accordance with current Department and AASHTO Specifications. Update existing computations accordingly.

When computer analysis is used, provide program input and output, calculations to prepare input, documentation of all assumptions, and any other post-processing calculations. Index computations so key data is readily available.

**G.** Use the Department's latest version of the appropriate bridge software for analysis and rating, if applicable.

**H.** Perform a structural analysis of the substructure only if its structural adequacy is at risk due to scour or section loss as a result of the field inspection findings or its unusual component makeup.

**I.** Evaluate each bridge to determine its capacity in its current condition relative to the four vehicle configurations (H, HS, ML-80, TK527) used to represent PA's legal loads and the need for a weight restriction and the level of posting.

For those situations where the Load Factor method results in lower ratings, a second rating utilizing an accepted method may be used to establish the posting levels.

## VI. DRAWINGS

**A.** Update existing drawings or sketches whenever possible, rather than preparing new drawings.

**B.** If no plans are available, prepare sufficient drawings to document the makeup of the structure. Include data and view as follows:

1. General plan and elevation.
2. Cross sections.
3. Framing plan.
4. Sketches of structural members (including dimensions).

5. Stress sheets.

6. Results of field inspection, analysis, and historical data, when appropriate.

7. Streambed cross sections, profile and soundings including areas of bed and bank scour.

8. Structural details, including all FCMs unless adequately documented by photographs.

**C.** For small and/or simple structures, sketches of 8½" X 11" format are acceptable. Prepare sketches using straight edges etc.

**D.** When retrofit schemes are requested, provide full size plan sheets (22" x 34").

## VII. PHOTOGRAPHS

Provide new digital photographs in the inspection report to supplement field inspection notes and drawings and to document current conditions. Provide photographs sufficiently clear, properly identified, dated, and indexed. See Pub 238 Section IP 8.3.1 for minimum photographs to include in reports.

## VIII. MEETINGS TO DISCUSS CRITICAL DEFICIENCIES WITH LOCAL OWNERS

Discuss all critical structural and safety-related deficiencies, including posting and repair recommendations, as well as critical and high priority proposed maintenance recommendations and alternatives contained in the current inspection report with the bridge owner at a formal meeting. A meeting is not required for critical deficiencies that involve only missing/damaged weight limit, vertical clearance, or any other regulatory signs. For County bridges, a Commissioners' meeting is appropriate. For Municipalities, arrange for appropriate officials to be present. The contracting agency (such as the County, if applicable) may also attend.

Place emphasis of discussion on uncorrected critical and other deficiencies brought forward from the previous inspection report. Ensure these deficiencies are highlighted in the current inspection report. Prepare informal minutes of the meeting that include attendance, issues discussed, proposed solutions, and needed follow-up items for the deficiencies.

This meeting may also be held to discuss inspection findings, general bridge condition and maintenance needs if requested by bridge owner and authorized by the District Bridge Engineer.

**A.** Convene the meeting within three days after identifying a critical structural deficiency and present a Plan of Action (POA) addressing the deficiencies to the owner. For high priority structure deficiencies, the meeting and POA must be conducted within seven days. Refer to Publication 238, IP 2.13.2. and IP 2.14 for information regarding the general requirements for the POA. During the meeting, ensure the owner has a thorough understanding of the critical nature of the defect(s) and the need for timely action as identified in the POA. Attendance by the engineering services consultant for the local owner is limited to the role of advising, communicating and facilitating the owner's understanding of the deficiency's effects on safety and development of the POA.

**B.** Provide liaison between the District and the owner when it is necessary to take immediate actions, permanent or temporary in nature, to safeguard public safety (e.g. temporary shoring, bridge closing) before the POA is fully developed.

**C.** If more time is needed to develop the POA because of the complexity of the problem, request a POA documentation time extension via e-mail to the District Bridge Engineer, copy to the local Bridge Coordinator and Chief Bridge Engineer, before the end of day 3 for Priority 0's and day 7 for Priority 1's. Include a description of actions taken to date to ensure public safety.

**D.** The Plan of Action shall provide essential information and be structured to match the BMS2 field information for IM01-IM15. The narrative format must show all maintenance actions and schedule required for repairs and restoration of safety to an acceptable level. In addition to the coding IM01-IM05 describing and prioritizing the deficiency in BMS2, address the following fields:

IM06 Date Recommended

IM07 Status of Work Candidate – Identify if the work will be done by the local owner forces or contractor, and whether work has been completed using appropriate coding. Typical codes include 1,2,5,6. (Note: For local and other owners for codes 1 and 5, “Dept” indicates “owner”).

IM08 Target Year IM09 Location

IM11 Work Assign IM12 Drawing Indicator IM13 Permit Indicator IM14b POA Date

IM14a Date Completed and IM14c Mitigated Date must be entered following completion of the work

IM15a Notes - A brief description of steps taken to address the deficiency which can include closing, posting, restricting, traffic temporary shoring, etc. The actions must restore the structure safety to an acceptable level. Note that additional work may be required at a later time to restore full level of service. A schedule for additional work should be included. Following completion of the immediate work and based on justification included in the plan, record the remaining additional repairs in BMS2 and note as such in item IM15b.

**E.** The meeting with the local owner should not be adjourned until agreement has been reached regarding specific action to be taken and associated schedule. After confirming the finished plan’s acceptability with the District Bridge Engineer, enter the appropriate information for each critical deficiency in the BMS2 IM fields (IM01-IM15). Upload documents related to the critical deficiency into BMS2 via the Documents link such as narrative version of the plan, sketches, meeting minutes, etc.

**F.** Immediately notify the District Bridge Engineer if the critical deficiency will not be addressed within seven (7) days or if the high priority maintenance item will not be addressed within 6 months.

**G.** Provide follow-up monitoring of the progress toward completion of the POA and report via BMS2 to confirm completion of the approved maintenance action(s) identified in the POA. Follow-up monitoring is also required for regulatory sign critical deficiencies. Enter the completion date(s), IM14a and IM14c, as appropriate in BMS2. The consultant shall immediately notify the owner and the District should problems arise with respect to the completion of the work within the required timeframe.

## IX. MATERIAL SAMPLING AND TESTING OR BRIDGE INSTRUMENTATION

Structural materials evaluation, Non-Destructive Testing (except dye penetrant tests) and bridge instrumentation are not a routine part of a bridge inspection. They are to be conducted only when required to eliminate unacceptable engineering uncertainties or to more accurately assess the structure’s load carrying capacity.

Justify the use and obtain the District Bridge Engineer authorization before initiating any materials sampling and testing and/or instrumentation program.

## X. EXISTING RECORDS AND DATA

The Department will provide BMS2 access for bridges to be inspected.

The Department and owner, if requested, will give the consultant access to any available pertinent information for short term use and copying. This information could include existing bridge drawings, load capacity analysis and design computations, inspection reports and other pertinent information. Files may be made available through the Department's BMS2 application. Some data may be available only on microfilm.

Review the existing records for a bridge prior to the in-field inspection including, but not limited to the following (as applicable): previous inspection report(s) and sketches, past priority maintenance correspondence, as-built or design drawings, current load rating analysis, F&F Plan, existing POAs, etc.

## XI. QUALIFICATIONS OF PERSONNEL

Personnel assigned to the Inspection Project by consultant shall meet the qualification requirements set forth in the National Bridge Inspection Standards for all work levels.

For State and Locally-Owned bridges, all inspection personnel must hold a valid certification as "Bridge Safety Inspector" issued by the Department.

## XII. TRAFFIC CONTROL

Provide any needed traffic control. Comply with the Department's Publication #213, "Work Zone Traffic Control Guidelines."

## XIII. RELEASE OF INFORMATION

Place the stamp appropriate to structure owner per Publication 238, IP 1.8.3 on the front cover of the inspection report. Do not release or distribute inspection information without the written permission of the District Bridge Engineer for State structures or the structure owner.

When portions of a report are approved for release, include the language provided in Publication 238, Figure IP 1.8.3-2 to each page of the structure inspection report that is released.

## XIV. AUTHORIZATION OF WORK AND DEADLINES

**A.** Be prepared to start work immediately upon receiving Notice to Proceed. Complete all work including the final report submission within agreed time schedule. Perform inspections to maintain the 24/48 month inspection interval or other reduced interval as specified during the Scope of Work meeting.

**B.** Upon receipt of Notice to Proceed, start work on all Initial Inventory and Inspection safety inspections, and Periodic (Routine) NBIS Inspections, as they come due.

**C.** The following work items require the prior authorization by the District Bridge Engineer before work can begin:

- Load Rating (or Re-rating) of Bridges
- Special Inspections
- Supplemental Inspections
- Critical Deficiency Meetings
- Material Sampling and Testing
- Bridge Instrumentation
- Creation of F&F Plan

Request authorization for work involving these items by submitting appropriate justification to the Department. Outline the proposed scope of work for task on each bridge in the justification. Do not proceed with these tasks until either oral or written authorization from the District Bridge Engineer is received.

**Scope Deliverables:**

**I. INSPECTION REPORT**

**A.** Prepare a report to document the inspection, the bridge, its condition, the structural analysis, load rating, posting evaluation, and recommendations. The report must be 8½" x 11" in size and copied on both sides (when hard copies are requested).

**B.** See Pub. 238, IP 8.3.1 for report requirements.

**II. EMERGENCY REPORTING**

Notify the bridge owner (if applicable) and the District Bridge Engineer immediately whenever a potentially perilous or hazardous condition is observed. Provide written notification to the owner and the District Bridge Engineer within 24 hours. This task is incidental to inspection work. Examples of such situations could include:

- Distress in primary members to the point where there is doubt that the members can safely carry the loads for which they are subjected and partial or complete failure of the bridge is a possibility.
- Scour at or under the abutments or piers of a stream bridge is such that significant movement is likely which could cause the bridge to collapse.
- Substructure movement or distress which is so excessive that there is a clear possibility that it may not be capable of supporting the superstructure and partial or complete failure is a possibility.
- Suspected cracks in pins or hangers of two girder/truss bridges.
- Missing weight restriction signs or vertical clearance signs.
- Any situation where the structural integrity of the bridge is such that its safety is in question.

**III. SUBMISSIONS**

**A.** Work Schedule and Status: Submit a horizontal bar graph type work schedule within two weeks of notice to proceed. Submit monthly schedules and progress updates to the District Bridge Engineer and contracting agency.

**B.** Personnel Qualifications: Thirty (30) days prior to beginning work, submit the list of names and qualifications of inspection personnel to the District Bridge Engineer. For Statewide agreements origination from Central Office, submit the list of names to the Assistant Chief Bridge Engineer - Inspection and/or the District Bridge Engineer, as necessary.

**C.** Field Inspection Data: Submit inspection data from BMS3 to BMS2 for Department's approval within ten days of the completion of each field inspection. In addition, submit one (1) copy of BMS2 Printout marked with revisions and/or Form D-491 and/or BMS3 Report within ten days of the completion of each field inspection.

**D.** Draft Inspection Reports: Submit the draft report within 30 days of the completion of each field inspection for review. Space submissions at frequent intervals to facilitate reviews.

**E.** Final Inspection Reports: For State bridges, submit the Final Report to the District and also upload to the BMS2 Documents link. Submission to the District shall be electronic PDF format unless hard copies are requested by the District. For local bridges, submit an electronic copy in PDF format to both the District and local owner unless a hard copy report is requested in advance. All hard copy reports are to be bound with non-exposed fasteners.

**F.** Load Rating/Re-Rating: For State bridges: Submit the Load Rating Analysis (including back-up documentation) within 2 months of each field inspection for review. Submission to the District shall be electronic PDF format unless hard copies are requested by the District. For local bridges, submit 3 copies and one (1) electronic copy in PDF format to the Districts. Submit BAR7 input file to the District. Update Load Ratings in BMS2.

**G.** Minutes of Critical Deficiency Meetings with owners: Submit one copy each to District Bridge Engineer, owner, and contracting agency within 7 days of meeting.

**H.** Plan of Action for Critical Deficiencies: See Pub 238 IP 2.14.

**I.** F&F Plan: For NSTM bridges with missing or incomplete F&F plans, create and submit a complete F&F plan to be include in the bridge file for use in future inspections (see Publication 238, IP 2.4.5.1).

***Sample Exhibit 1 –*** The following are sample tables, to be created outside of ECMS and attached to the scope of work, as referenced in the Details section of 2.7.5.

**EXHIBIT 1 – LIST OF BRIDGES**

**BRIDGE SAFETY INSPECTION**

| | | | **Proposed Scope of Work*** | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| **No.** | **BMS#** | **Owner** | **Initial NBIS** | **Routine Insp** | **Partial Insp** | **Closed Br Insp** | **Suppl. Insp** | **Load Rating** | **Comments** |
| 1 | 12-7456-7890-1234 | Twp | | A1 | | | | | |
| 2 | 23-7567-8901-2345 | County | | A2 | I2 (2) | | | | 6 month Special, sub. only, Req'd 2 times. |
| 3 | 34-7678-9012-3456 | County | | | | BC | | | Closed bridge |
| 4 | 45-7789-0123-4567 | Boro | | C2 | | | | | |
| 5 | 56-7890-1234-5678 | RR | A1 | | | | | | RR over hwy. No load rating req'd |
| 6 | 67-7901-2345-6789 | County | | A3 | | | | A3 | NHS, LF rating needed |
| 7 | 78-7012-3456-7890 | Twp | | C1 | I4 | | | | 12 month Special, super only |
| 8 | 87-7012-3456-7890 | Twp | A1 | | | | | | New bridge, design rating available. |
| 9 | 89-7123-4567-8901 | Twp | | A2 | | | | | |
| 10 | 23-7567-8901-2345 | Twp | | B1 | I5 | | | | 12 month, floorbeam only |
| 11 | 34-7678-9012-3456 | Twp | | A1 | | | | | |
| 12 | 45-7789-0123-4567 | Twp | | A2 | | | | | |
| 13 | 56-7890-1234-5678 | Twp | | B3 | | | | | |
| 14 | 67-7901-2345-6789 | Twp | | A1 | | | | | |
| 15 | 78-7012-3456-7890 | Twp | | A1 | | | | | |
| 16 | 87-7012-3456-7890 | Twp | | A1 | | | | | |
| 17 | 89-7123-4567-8901 | Twp | | A2 | | | | S1 | See Special Instructions for Suppl Insp SOW |
| 18 | | | | | | | | | |
| 19 | | | | | | | | | |
| 20 | | | | | | | | | |

***See Table 1 on next page for Work Categories**

| **No.** | **Critical Deficiency Meetings** | **Category of Work** |
|---|---|---|
| 1 | w/Tionesta Township | **M1** |
| 2 | w/Forest County | **M1** |

Notes:

1. Unless otherwise noted, only one Interim inspection per bridge allowed.
2. Other Interim inspections will be requested on an as-needed basis.
3. Other Bridge Load Ratings will be requested on as-needed basis.
4. Unless otherwise noted, only one Critical Deficiency Meeting per owner allowed.

| Table 1: Work Categories for Proposed Scope of Work | | |
|---|---|---|
| Structures | | Work Category |
| Types | Length | |
| Culverts, Slabs, Stringers, Multi-girders, & Arches (Except Open Spandrel Arches) | 20’-80’ | A1 |
| | 81’ to 150’ | A2 |
| | 151’ to 300’ | A3 |
| | 301’ to 600’ | A4 |
| | 601’ to 1000’ | A5 |
| | Greater than 1000’ | A6 |
| | All Lengths (Closed) | AC |
| Girder/Floorbeam Systems and Open Spandrel Arches | 20’ to 150’ | B1 |
| | 151’ to 300’ | B2 |
| | 301’ to 600’ | B3 |
| | 601’ to 1000’ | B4 |
| | Greater than 1000’ | B5 |
| | All Lengths (Closed) | BC |
| Trusses | 20’ to 150’ | C1 |
| | 151’ to 300’ | C2 |
| | 301’ to 600’ | C3 |
| | 601’ to 1000’ | C4 |
| | Greater than 1000’ | C5 |
| | All Lengths (Closed) | CC |
| All Others | 20’ to 80’ | D1 |
| | 81’ to 150’ | D2 |
| | 151’ to 300’ | D3 |
| | 301’ to 600’ | D4 |
| | 601’ to 1000’ | D5 |
| | Greater than 1000’ | D6 |
| | All Lengths (Closed) | DC |
| | | |
| Interim | | I |
| Supplemental | | S |
| Critical Deficiency Meetings | | M |

# APPENDIX IP 01-G

General Scope of Work –
Safety Inspection of State and Local Tunnels

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# General Scope of Work – Safety Inspection of State and Local Tunnels

**Description:** Safety Inspection of State and Local Tunnels

**Objective:** Inspect, load rate, inventory and appraise tunnels and perform follow-up work as directed.

**Statement of Work:** The requirements of the latest versions of the Department accepted AASHTO and FHWA manuals and the latest versions of Department Publications and Policy, including any updates, shall be followed in the performance of the Scope of Work. See the Department's Bridge Safety Inspection Manual, Publication 238, IP 1.3.2 and IP 1.3.3 for a list.

**Scope:** The scope of work will include the following activities:

## I. TYPES OF SAFETY INSPECTION WORK

**A.** **Initial Inspection** - Insufficient or no data is available in BMS2. An inspection fulfilling NTIS requirements has never been performed. For tunnels with structural members carrying highway traffic (directly over or through the tunnel), a separate Tunnel Load Rating work item must also be done and its results incorporated into this initial inspection report (see Scope Section II.E., "Tunnel Load Rating") when applicable. Additional inspection requirements for an Initial Inspection are outlined in Publication 238, IP 2.3.1.

**B.** **Routine Inspection** - An NTIS Inspection has been previously completed within the last two (2) years and that inspection report and/or documentation is available. Additional inspection requirements for a Routine Inspection are outlined in Publication 238, IP 2.3.2.

**C.** **Special Inspection** - An NTIS Inspection has been previously completed. The structure is included in the BMS2 and the previous inspection report is available. Perform an inspection that is usually limited to portion(s) of the structure which require reduced interval inspections. Specific inspection requirements are outlined in Publication 238, IP 2.3.5. The scope of work for a Special Inspection must be approved by the District Bridge Engineer prior to initiating work.

**D.** **Supplemental Inspection** - Perform in-depth work beyond the scope of Routine inspections, focusing on a specific area or the entire structure (as in an In-Depth Inspection as outlined in Publication 238 IP 2.3.4) or specific components (as in a Special Inspection as outlined in Publication 238 IP 2.3.5). In-depth tasks may include the following:

- Non-Destructive Testing (except dye penetrant),
- Laboratory Analysis,
- Geotechnical sampling and testing, and/or
- Structure instrumentation

The scope of work authorizing a Supplemental Inspection should have provisions for these tasks identified in the LIST OF SPECIAL REQUIREMENTS. The scope of work for an In-Depth or Special type of inspection must be approved by the District Bridge Engineer prior to initiating work.

**E.** **Load Rating** - Perform a structural analysis and load rating of the structure to determine its ability to carry PA's legal loads when applicable.

**F.** **Critical Finding Meetings** - Coordinate and conduct a meeting with local tunnel owners to discuss critical findings noted during the recent inspections.

## II. INSPECTION REQUIREMENTS

### A. Initial Inventory and Inspection

1. Conduct a complete inventory and field inspection utilizing BMS3.

2. Complete BMS2 Inventory data items via BMS2 web and BMS3 Inspection Pages.

3. If structural members of the tunnel carry highway traffic either directly over or through the tunnel, incorporate the Tunnel Load Rating performed under separate work item into the initial inspection report. Evaluate tunnel for posting needs.

4. Prepare an Inspection Report.

5. Identify SNTI elements.

6. Calculate element quantities.

7. Prepare an element summary table for all elements and provide supporting calculation for each element.

8. Identify and record condition states for all elements, including defects.

**B. Routine Inspection**

1. All tunnels, except closed tunnels.

    a. Conduct a complete field inspection utilizing BMS3.

    b. Update/supplement the evaluation for posting needs for the structure's current condition. Determine if re-rating is warranted by comparing new vs. existing section loss measurements. If structure is to be re-rated, use the new load rating summary.

    c. Update/amend the Inspection File providing new photographic documentation and/or sketches as needed.

    d. Update and/or complete the required minimum BMS2 inventory and inspection items via BMS2 web. See Scope Section III.C., "Minimum Required Inventory and Inspection Data," for minimum BMS2 items required.

    e. Incorporate the results of previous or new load ratings into the report.

    f. Prepare an Inspection Report to document all proposed and completed maintenance recommendations.

    g. Identify and record condition states for all elements.

    h. Update elements and/or quantities based on changed field conditions.

    i. Verify functional systems are being tested on the interval outlined in the tunnel specific inspection documentation and the results of the testing are properly documented in the tunnel file.

2. **Closed Tunnels**

    a. Inspect tunnels closed to highway traffic to assure that the physical barriers are maintained and that the public safety is not jeopardized.

    b. Use PennDOT BMS3 Inspection Pages for field notes. Include a minimum of two (2) photos showing tunnel with in-place barriers.

    c. Prepare an Inspection Report to document all findings.

3. **Partially Closed Tunnels**

    a. Inspect the open portions of tunnels partially closed for staged construction as outlined in Scope Section II.B.1.

    b. Prepare an Inspection Report to document all findings.

**C. Special Inspection**

1. Inspect the specified portion(s) of the structure as authorized by the District Bridge Engineer. Use PennDOT BMS3 Inspection Pages.
2. Update/supplement the posting evaluation of the portion inspected.
3. Update/amend the portion of the Inspection Report dealing with the portion inspected.
4. Update and/or complete the required minimum BMS2 inventory and inspection items via BMS2 web relevant to the portion inspected.
5. Prepare an Inspection Report to document all work and findings.

**D. Supplemental Inspection**

1. Conduct inspection of structure as directed by the Department. Use PennDOT BMS3 Inspection Pages.
2. Perform follow-up sampling and testing as specified.
3. Update/amend the portion of the Inspection Report dealing with the portion inspected.
4. Update and/or complete the required minimum BMS2 inventory and inspection items via BMS2 web dealing with the Supplemental Inspection.
5. Prepare an Inspection Report to document all work and findings.

**E. Tunnel Load Rating**

1. Perform or update the structural analysis and load ratings for all PA legal loads using the latest specification and programs.
2. Identify the structural components or members that govern the ratings.
3. Prepare a load rating summary table and/or stress table for the Inspection Report. Reference calculation page number for ratings.
4. When appropriate update ratings directly in BMS2 web.

**F. Critical Finding Meetings**

1. Arrange and conduct a meeting with local tunnel owners to discuss critical findings found during the inspection.
2. Prepare informal meeting minutes and supply copy of the minutes to the District and municipality.

## III. BMS2 INVENTORY AND INSPECTION DATA

**A. Department Structures** - Complete and/or update all applicable data items of the BMS2 D-491 Forms printout unless otherwise instructed. Complete new forms for new tunnels.

**B. Local Government Tunnels and Others** - Provide complete data as in the Scope Section III.A., unless otherwise directed to provide only minimum data.

**C. Minimum Required Inventory and Inspection Data -** Minimum inventory and inspection data includes the following BMS2 Items:

| | |
|---|---|
| 5A01 | Tunnel ID |
| 5A02 | Tunnel Name |

| | |
|---|---|
| 5A03 | Tunnel Number |
| 5A04 | Highway Agency District |
| 5A05 | County Code |
| 5A06 | Place Code |
| 5A08 | Facility Carried |
| 5A10 | Tunnel Portal's Latitude |
| 5A11 | Tunnel Portal's Longitude |
| 5A12 | Border Tunnel State or County Code |
| 5A14 | State Code |
| 5A15 | Year Built |
| 5A16 | Year Rehabilitated |
| 5A19 | Number of Lanes |
| 5A21 | Owner |
| 5B05-5B06 | Sidewalk Width |
| 5B18 | Tunnel Length |
| 5C04 | Route Type |
| 5C06 | Route Number, Route Direction |
| 5C08 | Total Number of Lanes |
| 5C10 | Annual ADT |
| 5C11 | Year of Annual ADT |
| 5C12 | Future ADT |
| 5C13 | Year of Future ADT |
| 5C14 | Annual ADTT |
| 5C15 | Detour Length |
| 5C18 | LRS Mile Point |
| 5C20 | LRS Route ID |
| 5C21 | Toll |
| 5C22 | Functional Classification |
| 5C23 | Direction of Traffic |
| 5C27 | Roadway Width, Curb-to-Curb |
| 5C28 | STRAHNET Designation |
| 5C30 | School Bus Route |
| 5C32 | Transit Bus Route |

| | |
|---|---|
| 6A04 | Co Municipality Boundary Code |
| 6A43 | Approach Pavement Width |
| 6B24 | Agency Hiring Consultant |
| 6B26 | Inspection Crew Hours |
| 6B32-6B34 | Inspection Cost |
| 6B48 | Combustible Materials Stored |
| 6C10 | Highway System |
| 6C11 | State Highway Network |
| 6C15 | NHS Designation |
| 6C20-6C21 | Minimum Vertical Clearance |
| 6C35-6C38 | Vertical Clearance Signing |
| 7A01 | Inspection Date |
| 7A03 | Type of Inspection |
| 7A05 | Consultant Name |
| 7A08 | Last Inspection Date |
| 7A09 | Inspection Interval |
| 7A10 | Next Inspection Target Date |
| 7A14 | Next Inspection By |
| VM03-VM04 | Inspection/Maintenance Responsibility |
| VP01 | Posting Status Date |
| VP02 | Tunnel Load Posting Status |
| VP03 | Special Restrictive Posting |
| VP04 | Posted Weight Limits |
| VP06 | Reason for Posting or Closing |
| IR06 | Load Rating Method |
| IR07 | IR Controlling Member |
| IR14 | AASHTO Manual Year for Rating |
| IR15 | AASHTO Spec Year for Rating |
| IM01-IM15 | Maintenance Recommendations |
| 1B | Inspection Element Detail Fields |
| I.17 | Border Tunnel Number |
| A.8 | Service in Tunnel |
| C.8 | Urban Code |

| | |
|---|---|
| L.10 | Height Restriction |
| L.11 | Hazardous Material Restriction |
| L.12 | Other Restrictions |
| S.1 | Number of Bores |
| S.2 | Tunnel Shape |
| S.3 | Portal Shape |
| S.4 | Ground Condition |
| S.5 | Complex |

Note: Only applicable items need to be coded. All submitted data will be stored in BMS2. Owners are encouraged to collect and submit all inventory and inspection information available.

## IV. FIELD INSPECTION AND ASSESSMENTS

**A.** Completely inspect all tunnel elements including civil, mechanical and electrical elements. Verify all functional systems are being tested on the interval specified in the tunnel-specific inspection documentation and the results of the testing are properly documented in the tunnel file. Clean members and disassemble systems as needed to assess condition. For a Special Inspection, inspect only the specified areas/members. However, report any public safety threatening deficiencies that are observed elsewhere in the tunnel. Include inspection of any sign structures attached to the tunnel.

**B.** Clearly record all inspection field notes in BMS3. Provide sufficient written comments within BMS3 to outline the tunnel's condition and to justify all condition state classifications. Precisely locate and describe deterioration and all areas of section loss. Perform dye penetrant testing if cracking is suspected or found. Determine if current conditions warrant a re-rating for load capacity. Determine if current load posting status is appropriate. Prepare sketches and obtain photographic documentation.

**C.** Identify and record all maintenance and major improvement needs utilizing BMS3 Inspection Pages.

**D.** Complete field SNTI tunnel element assessment utilizing BMS3 Inspection Pages. Identify and record condition states for all elements. Provide sufficient comments within BMS3 to describe the element's condition and corresponding location to justify all condition states. Update elements and/or element quantities based on changed field conditions.

**E.** Provide emergency retrofit schemes, as directed, to any critical conditions uncovered.

**F.** Arrange for rigging, inspection cranes, platform lift trucks, ladders, etc. Arrange for any needed Traffic Control. Ensure the safety of inspectors and public at all times.

## V. STRUCTURAL ANALYSIS, LOAD RATING, and POSTING EVALUATION

**A.** Acquire authorization from the District Bridge Engineer prior to performing a structural analysis or load rating.

## VI. DRAWINGS

**A.** Update existing drawings or sketches whenever possible, rather than preparing new drawings.

**B.** If no plans are available, prepare sufficient drawings to document the makeup of the structure. Include data and view as follows:

1. General plan and elevation.
2. Cross sections.
3. Framing plan.
4. Sketches of structural members (including dimensions).
5. Stress sheets.
6. Results of field inspection, analysis, and historical data, when appropriate.

**C.** For small and/or simple structures, sketches of 8½" X 11" format are acceptable. Prepare sketches using straight edges etc.

**D.** When retrofit schemes are requested, provide full size plan sheets (22" x 34").

## VII. PHOTOGRAPHS

Provide new digital photographs in the inspection report to supplement field inspection notes and drawings and to document current conditions. Provide photographs sufficiently clear, properly identified, dated, and indexed. Include views of the overall tunnel at each portal, the approach roadway and its alignment, typical condition of major components, deficiencies, posting restrictions, structural details, functional systems and other important features.

## VIII. MEETINGS TO DISCUSS CRITICAL FINDINGS WITH LOCAL OWNERS

Discuss all critical structural and safety-related deficiencies (critical findings), including posting and repair recommendations, as well as steps taken to date to ensure public safety in regards to the critical findings with the tunnel owner at a formal meeting. For County tunnels, a Commissioners' meeting is appropriate. For Municipalities, arrange for appropriate officials to be present. The contracting agency (such as the County, if applicable) may also attend.

Place emphasis of discussion on uncorrected critical and other deficiencies brought forward from the previous inspection report. Ensure these deficiencies are highlighted in the current inspection report. Prepare informal minutes of the meeting that include attendance, issues discussed, proposed solutions, and needed follow-up items for the deficiencies.

This meeting may also be held to discuss inspection findings, general tunnel condition and maintenance needs if requested by tunnel owner and authorized by the District Bridge Engineer.

**A.** Convene the meeting within three days after identifying a critical finding and present a Plan of Action (POA) addressing the deficiencies to the owner. Refer to Publication 238, IP 2.13.2. and IP 2.14 for information regarding the general requirements for the POA. During the meeting, ensure the owner has a thorough understanding of the critical nature of the defect(s) and the need for timely action as identified in the POA. Attendance by the engineering services consultant for the local owner is limited to the role of advising, communicating and facilitating the owner's understanding of the deficiency's effects on safety and development of the POA.

**B.** Provide liaison between the District and the owner when it is necessary to take immediate actions, permanent or temporary in nature, to safeguard public safety (e.g. temporary shoring, tunnel closing) before the POA is fully developed.

**C.** If more time is needed to develop the POA because of the complexity of the problem, request a POA documentation time extension via e-mail to the District Bridge Engineer, copy to the local Bridge Coordinator and Assistant Chief Bridge Engineer - Inspection, before the end of day 3. Include a description of actions taken to date to ensure public safety.

**D.** The Plan of Action shall provide essential information and be structured to match the BMS2 field information for IM01-IM15. The narrative format must show all maintenance actions and schedule required for repairs and restoration of safety to an acceptable level. In addition to the coding IM01-IM05 describing and prioritizing the deficiency in BMS2, address the following fields:

IM06 Date Recommended

IM07 Status of Work Candidate – Identify if the work will be done by the local owner forces or contractor, and whether work has been completed using appropriate coding. Typical codes include 1,2,5,6. (Note: For local and other owners for codes 1 and 5, “Dept” indicates “owner”).

IM08 Target Year IM09 Location

IM10 Estimated Cost IM11 Work Assign IM12 Drawing Indicator IM13 Permit Indicator IM14b POA Date

IM14a Date Completed and IM14c Mitigated Date must be entered following completion of the work

IM15a Notes - A brief description of steps taken to address the deficiency which can include closing, posting, restricting, traffic temporary shoring, etc. The actions must restore the structure safety to an acceptable level. Note that additional work may be required at a later time to restore full level of service. A schedule for additional work should be included. Following completion of the immediate work and based on justification included in the plan, record the remaining additional repairs in BMS2 and note as such in item IM15b.

**E.** The meeting with the local owner should not be adjourned until agreement has been reached regarding specific action to be taken and associated schedule. After confirming the finished plan’s acceptability with the District Bridge Engineer, enter the appropriate information for each critical finding in the BMS2 IM fields (IM01-IM15). Upload documents related to the critical deficiency to the BMS2 Documents link such as narrative version of the plan, sketches, meeting minutes, etc.

**F.** Immediately notify the District Bridge Engineer if the critical finding will not be addressed within a timely manner. Timeframe for repairs shall be set by the District Bridge Engineer at the meeting.

**G.** Provide follow-up monitoring of the progress toward completion of the POA and report via BMS2 to confirm completion of the approved maintenance action(s) identified in the POA. Follow-up monitoring is also required for regulatory sign critical deficiencies. Enter the completion date(s), IM14a and IM14c, as appropriate in BMS2. The consultant shall immediately notify the owner and the District should problems arise with respect to the completion of the work within the required timeframe.

## IX. MATERIAL SAMPLING AND TESTING OR TUNNEL INSTRUMENTATION

Structural materials evaluation, Non-Destructive Testing (except dye penetrant tests) and instrumentation are not a routine part of a tunnel inspection. They are to be conducted only when required to eliminate unacceptable engineering uncertainties or to more accurately assess the structure’s load carrying capacity.

Justify the use and obtain the District Bridge Engineer or local Owner authorization before initiating any materials sampling and testing and/or instrumentation program.

## X. EXISTING RECORDS AND DATA

The Department will provide BMS2 web access for tunnels to be inspected.

The Department and owner, if requested, will give the consultant access to any available pertinent information for short term use and copying. This information could include existing tunnel drawings, load capacity analysis and design computations, inspection reports and other pertinent information. Files may be made available through the Department’s BMS2 web application.

Review the existing records for a tunnel prior to the in-field inspection including, but not limited to the following (as applicable): tunnel-specific inspection procedures document, previous inspection report(s) and sketches, past priority maintenance correspondence, as-built or design drawings, current load rating analysis, existing POAs, etc.

## XI. QUALIFICATIONS OF PERSONNEL

Personnel assigned to the Inspection Project by consultant shall meet the qualification requirements set forth in the NTIS for all work levels.

All inspection personnel must be registered with the Assistant Chief Bridge Engineer - Inspection as detailed in Section IP 2.1.3.3.

## XII. TRAFFIC CONTROL

Provide any needed traffic control. Comply with the Department's Publication 213, Work Zone Traffic Control Guidelines.

## XIII. RELEASE OF INFORMATION

Place the stamp appropriate to structure owner per Publication 238, IP 1.8.3 on the front cover of the inspection report. Do not release or distribute inspection information without the written permission of the District Bridge Engineer for State structures or the structure owner.

When portions of a report are approved for release, include the language provided in Figure IP 1.8.3-2 to each page of the structure inspection report that is released.

## XIV. AUTHORIZATION OF WORK AND DEADLINES

**A.** Be prepared to start work immediately upon receiving Notice to Proceed. Complete all work including the final report submission within agreed time schedule. Perform inspections to maintain the 24-month inspection interval or other reduced interval as specified during the Scope of Work meeting.

**B.** Upon receipt of Notice to Proceed, start work on all Routine NTIS Inspections, as they come due.

**C.** The following work items require the prior authorization by the District Bridge Engineerbefore work can begin:

- Load Rating (or Re-rating) of Tunnels
- Special Inspections
- Supplemental Inspections
- Critical Finding Meetings
- Material Sampling and Testing
- Structure Instrumentation
- Creation of Tunnel Specific Inspection Procedures Document

Request authorization for work involving these items by submitting appropriate justification to the Department. Outline the proposed scope of work for task on each tunnel in the justification. Do not proceed with these tasks until either oral or written authorization from the District Bridge Engineer is received.

**Scope Deliverables:**

## I. INSPECTION REPORT

**A.** **Prepare a report** to document the inspection, the tunnel, its condition, the structural analysis, load rating, posting evaluation, results of functional system checks, critical findings, and recommendations. The report must be 8½" x 11" in size and copied on both sides (when hard copies are requested).

**B.** **A general outline of the report is as follows:**

1. Title page (Structure ID Number, BRKEY, tunnel name, location, inspection dates, inspector names, prepared for and by, and Pennsylvania P.E. seal, signature and date). Label Posted tunnels as "Posted", "New Posting", or "Posting Change".
2. Table of contents.
3. Location map(s). Map(s) must be of sufficient detail to locate structure.
4. General description and sketches and/or photographs of the overall structure.
5. Field inspection findings (completed BMS3 Inspection Pages, plus photographs and supplemental narrative to document findings).
6. References, list plans, previous reports, etc. used in the preparation of the report.
7. Load rating summary and posting evaluation.
8. Recommendations
9. Appendices:
   a. Inventory Data: Marked-up copy of BMS2 file printout or completed copy of coding D-491 forms
   b. Inspection Data: Completed BMS3 Inspection Pages.
   c. Structural analysis and load rating computations and a table of stresses.
   d. Member deficiency sketches where applicable.

**C.** **Include the following in the report Narrative:**

1. General description of the structure condition.
2. Summary of inspection findings and comparison with those of previous inspection.
3. Structural adequacy and safety of the structure and all systems and components.
4. Discuss relevant historical data.

**D.** **Include the following in the Recommendations section:**

1. Need for Special Inspection and/or Supplemental Inspections.
2. Need for new or revised weight restrictions.
3. Signing needs: Vertical clearance, etc.
4. A prioritized and time scheduled listing of immediate, short- and long-term improvement needs for:
   a. Maintenance: Complete BMS3 Maintenance page
   b. Rehabilitation: Complete BMS3 Maintenance page or D491IM
   c. Replacement: Complete BMS3 Maintenance page or D491IM

Recommendations in report should be in "plain English."

**E. Other Report Requirements**

1. Routine NTIS Inspections without re-rating - The complete detailed structural analysis and load rating computations (see Scope Deliverables, Section I.B.9.c) from previous inspection/rating need not be included, unless otherwise specified. The load rating summary and date of load rating must still be included with the posting evaluation.

Review/perform the posting evaluation for each tunnel to ensure its posting status is appropriate for its just inspected condition.

2. Routine NTIS Inspections for Closed or Partially Closed Tunnels - A letter report stating date of inspection, status of closure with photo, BMS3 Inspection Pages, and other pertinent information will suffice, unless otherwise specified.

3. Special and Supplemental Inspections - The report format and contents are to be agreed upon at the time of authorization for each structure.

## II. EMERGENCY REPORTING

Notify the tunnel owner and the District Bridge Engineer immediately whenever a potentially perilous or hazardous condition is observed. Provide written notification to the owner, the District Bridge Engineer and FHWA within 24 hours. This task is incidental to inspection work. Examples of such situations could include:

- Distress in primary members to the point where there is doubt that the members can safely carry the loads for which they are subjected and partial or complete failure of the tunnel is a possibility.
- Substructure movement or distress which is so excessive that there is a clear possibility that it may not be capable of supporting the superstructure and partial or complete failure is a possibility.
- Missing weight restriction signs or vertical clearance signs.
- Any situation where the structural integrity of the tunnel is such that its safety is in question.

## III. SUBMISSIONS

**A.** Work Schedule and Status: Submit a horizontal bar graph type work schedule within two weeks of notice to proceed. Submit monthly schedules and progress updates to the District Bridge Engineer and contracting agency.

**B.** Personnel Qualifications: Thirty (30) days prior to beginning work, submit the list of names and qualifications of inspection personnel to the Assistant Chief Bridge Engineer - Inspection.

**C.** Field Inspection Data: Submit inspection data from BMS3 to BMS2 web for Department's approval within 30 days of the completion of each field inspection. In addition, submit BMS2 Printout marked with revisions and/or Form D-491 and/or BMS3 Report within 30 days of the completion of each field inspection.

**D.** Draft Inspection Reports: Submit the draft report within 60 days of the completion of each field inspection for review. Space submissions at frequent intervals to facilitate reviews.

**E.** Final Inspection Reports: For State tunnels, submit the Final Report to the District after acceptance of the draft.

**F.** Critical Findings: Submit documentation of critical findings to FHWA and the Program Manager within 24 hours of discovery. Contact information and responsibilities are outlined in the tunnel specific inspection agreement.

**G.** Plan of Action for Critical Findings: Submit to District Bridge Engineer, owner and FHWA within 3 days.

**H.** Minutes of Critical Finding Meetings with Owners: Submit to District Bridge Engineer, owner, FHWA, and contracting agency within 7 days of meeting.

Tunnel-Specific Inspection Procedures Document: For tunnels with an incomplete or missing tunnel-specific inspection procedures, complete and submit this document to be include in the tunnel file for use in future inspections.

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# APPENDIX IP 01-H

General Scope of Work –
Underwater Inspection of Bridges

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# General Scope of Work – Underwater Inspection of Bridges

**Description:** Underwater Bridge Inspection – Cost per Unit

**Objective:** Underwater bridge inspection

**Statement of Work:**

## I. GENERAL

**A. Purpose:** Inspect and appraise the underwater portion of bridges.

**B. Guidance:** The requirements of latest versions of the Department accepted AASHTO and FHWA manuals and the latest versions of Department Publications and Policy, including any updates, shall be followed in the performance of the Scope of Work. See the Department's Bridge Safety Inspection Manual, Publication 238, IP 1.3.2 and IP 1.3.3 for a list.

**C. Scope:** This scope will include the following activities:

## II. TYPES OF UNDERWATER INSPECTION WORK

### A. Description of Work Types

| | |
|---|---|
| **Diving** | Underwater bridge inspections that require hard hat/scuba diving. |
| **Probing** | Underwater bridge inspection performed by probing and/or sounding methods. |
| **Mobilization** | Pre-inspection work required for diving inspections |
| **Travel** | Travel to and from the bridge and/or for specific work types |
| **Meetings** | Participation in meetings with bridge owners. For locally-owned bridges, this may include additional units to develop a POA. |
| **Special** | Underwater bridge inspection related work not covered by the other types of work and as directed by the Bureau of Bridge. This work may include field work, reports, studies, and other tasks. |

### B. Categories of Work

Work for diving and probing inspections is to be divided into categories related to the water depth, water velocity, and number of units to be inspected at each bridge site as indicated in Exhibit 1.

### C. Assignment of Work

1. Diving and Probing Inspections

The Districts will assign this work to the Consultant via Work Orders listing the bridges and substructures to be inspected.

Prior to finalizing your work schedule for a Work Order, review and verify the available information on the list of assigned bridges with the District (and local owner if appropriate) to confirm that the Type and Category of Work proposed is consistent for the work required at each site. This review will minimize duplication of effort and will provide the basis for formulating specific directions and expectations on each structure.

2. Mobilization

The District may authorize and assign this work to the consultant for diving inspections based on the depth of the water and the specialized equipment required for certain inspections.

3. Travel

The District may authorize and assign this work to the consultant for either diving inspections based on the depth of water or to meet with bridge owners.

4. Meeting with Bridge Owners

The District may authorize and assign this work to the consultant if requested by the bridge owner. If a critical deficiency is identified, the District may authorize specific POA units for the consultant to develop the POA for the local owner.

5. Special Underwater Inspection Related Work

This work may only be authorized and assigned by the Assistant Chief Bridge Engineer - Inspection.

**D. Cold Weather Adjustment for Inspections**

1. Cold Weather (Diving)

A Cold Weather Add-On Cost is to be allowed for diving inspections when the site's ambient temperature at the time of inspection is less than 35 degrees Fahrenheit.

2. Cold Weather (Probing)

For cold weather probing inspections, a Cold Weather Add-On Cost may be allowed for emergency inspections where ambient temperature is less than 20 degrees Fahrenheit or if the ice surrounding at the substructure unit is at least 4" thick or more at the time of inspection.

3. Water Velocity (Diving)

For diving inspections, a Water Velocity Add-On Cost is to be allowed for diving inspections when the sites water velocity exceeds 3 feet per second.

All approvals will require documentation in the inspection report of the site conditions for which the consultant is requesting additional units. For State and local bridges, authorization will be made at the District level. For Other State Agency bridges, authorizations will be granted by the Bridge Inspection Section in the Bureau of Bridge.

## III. BMS2 DATA

Complete all applicable portions of the bridge inspection pages, BMS2 Coding Forms D-450's and/or the BMS2 file printout, including items 5A07, VD14-VD17, 6A51, FW11, 4A21-4A24, 6A10, VI12, 6B01, 7A03, 7A05, 7A06, 7A07, 7A09, 7A10, 6B26, 1A02, 1A05, 1A03, 1A06, 6B48, 4A08, the IN, IU and IM screens as it relates to channel, waterway, substructure, plan of actions, etc. as well as inspection element-level fields.

## IV. FIELD INSPECTION

**A. General**

1. Prior to beginning field inspection, inform the bridge owner of the schedule andthe names of the inspectors.
2. Record the ambient temperature, water velocity, and ice thickness at each substructure at the time of inspection.

**B. Structure Inspection**

1. Inspect the portions subjected to being submerged for damage, cracking, settlement, steel corrosion, deteriorated and scoured concrete, deteriorated pointing, broken and/or dislodged stones in masonry structures, deterioration and/or damage to piling, insect damage or wood decay, etc.
2. Provide special attention to determine the uniformity of bearing of footings and surrounding foundation

materials and the lateral stability and soil support to the pile foundations, the effect or potential effect of scour, and also the soundness or effectiveness of any previous repairs.

3. Sound all timber and probe with a heavy duty 6-inch (min.) blade, ice pick or awl.

4. Identify limits of past scour protection.

**C. Streambed Inspection**

1. Inspect the streambed in the area of the substructure unit as to type of material, evidence of scour, condition of existing scour protection, debris, etc. Obtain elevations relative to a fixed permanent reference point marker to provide for accurate plotting of streambed contours and/or streambed profiles in the areas suspect of scour.

2. Provide stream bottom data on a minimum five (5) foot grid around each pier to extend beyond the scour hole but in no case less than twenty-five (25) feet beyond the footing area. Estimate flow velocities and direction of flow relative to the foundation structure. Note all turbulence and unusual flow conditions.

3. Obtain channel cross-sections at the bridge and two bridge lengths upstream and downstream. For bridge lengths, greater than 100', obtain channel cross-sections at 200’ upstream and downstream. Significant features directly observable but beyond 200’ from the bridge should be included. Do not create new cross-sections, if existing cross- sections can be utilized. Mark any changes on the existing cross-sections from the bridge inspection file.

   The cross section at the fascia must include the following:

   a. Top-of-bank to top-of-bank channel section at upstream face of bridge.
   b. Geometry of principal bridge openings up to the anticipated high-water elevation.
   c. Foundation units.
   d. Stream bed materials and boring information.
   e. Roadway profile in the vicinity of the bridge.
   f. Discernible scour holes.
   g. Structural countermeasures at the bridge.
   h. Discernible high-water marks at the bridge.
   i. Stream level at the upstream side of the bridge at time of inspection.
   j. Reference marks on the bridge.
   k. The upstream and downstream sections may be sounded from a surveyed water surface elevation. Obtain sounding by using a continuous reading strip chart Fathometer unless water conditions preclude use of a boat, in which case sounding poles or lead lines may be utilized.

**D. Reference Point Marker**

Place a permanent marker if one does not exist already (drill hole, nonferrous PK nail) on each abutment/pier (elevation/datum) that correlates to the report findings and which may be used in future underwater investigations and/or rehabilitation work. Provide one such marker per unit at a convenient location. This bench mark is to be referenced on the plan.

**E. Verification of Field Conditions for the Observed Scour Assessment**

At the substructure units being inspected, verify and update the field conditions recorded in the most recent Observed Scour Assessment Report. The District will provide one copy of the appropriate portions of the Observed

Scour Assessment Report.

In the “UNDER THE BRIDGE” section of the Assessment, review data applicable to the substructure units being inspected. Mark all changes in red ink without erasing or obliterating original information. If previous information is valid, note each data item as such with a checkmark or the words “No change”. Update the Plan Sketch and Channel Cross Section at the Bridge accordingly. Record the date of the underwater inspection and inspection leader on each page.

This update of the Scour Assessment Plan and Cross Section is not in lieu of and does not satisfy the Underwater Inspection Report requirements set forth in Sections IV, Parts A through D and V of this Scope.

**F. State-owned Bridges and Locally-Owned Bridges on the National Highway System (NHS) -**

The AASHTO / PennDOT bridge element inventory and assessment for the bridge has been previously completed and documentation is available. Identify and inventory elements and calculate element quantities and scale factors. Identify and record the defect code(s) and condition state quantities for the inventoried elements.

## V. DRAWINGS

### A. General

Prepare sufficient drawings to document the condition of the substructure units and stream. Use sketch abbreviations, terminology, and symbols as shown on the attached sketches and list of abbreviations (Attachment A).

### B. Type 1 Diving Inspection Requirements

Include the following:

1. General plan, elevation, and contours.
2. Channel Cross sections showing Foundation Material, Footings, portions of substructure units which are underwater, remediation material and areas of scour.
3. Results of underwater inspections including details of section loss due to deterioration, or damage.
4. Streambed profile and soundings including areas of bed and bank scour.
5. Unusual structural elements unless documented by photographs.

### C. Probing Inspection Requirements

For probing inspection furnish all the diving inspection information available.

### D. Special Inspection Requirements

Drawing requirements for special inspections must be detailed in the proposal if available.

## VI. PHOTOGRAPHS

Provide new photographs of the bridge and new underwater photographs to supplement field inspection notes and drawings. Photos may be used in lieu of detail sketches if the pictures are sufficiently clear, adequately dimensioned, and are properly identified and indexed.

Xerographic/laser copies of photographs, electronically scanned prints, and prints from a digital electronic camera are acceptable substitutes for report photographs if the resolution and quality is acceptable to the Districts.

## VII. INSPECTION REPORT

### A. Report Requirements

Provide a written report using explicit terminology and language covering the factors relevant to the condition of the substructure, such as:

1. Detail general condition as revealed by the field inspection; past, present and potential flooding conditions, if relevant; history of repairs; and all other features which may affect the service life of the substructure.
2. Provide detailed descriptions of the inspection. Such details shall be referenced and shown on the drawings. Sketches should be such that aggradation and degradation of material around piers can be readily identified during subsequent inspections.
3. Compare channel cross-sections with those obtained in previous inspections and significant stream changes shall be identified.
4. Provide recommendations as to: need for minor repairs; need for major repairs; scheduling of repairs; anticipated useful life of the substructure; recommended intervals for future inspections; and any other recommendations which may be pertinent to the perpetual safety of the structure, such as scour computations or substructure analysis.
5. When repairs are recommended, estimate quantities and cost of the repairs. Prioritize these repairs.
6. Highlight critical deficiencies and/or other important findings on a separate sheet(s).
7. List names of Certified Divers in the report.
8. Sign and seal the report by a Professional Engineer licensed in Pennsylvania.

### B. Submission of Reports

1. Draft Report:

Submit a draft copy of the inspection report to the Engineering District within four weeks of completion of each field inspection for review and comments. For the first five draft reports completed, also submit a copy to the Bridge Inspection Section in the Bureau of Bridge.

2. Final Report:

Submit the final report within four weeks after receipt of review/comments of the draft copy. An electronic submission is preferred. Only submit a hard copy if requested by the District.

3. Submission Schedule:

Space submissions at frequent intervals to facilitate review.

## VIII. QUALIFICATIONS OF PERSONNEL

Personnel assigned to the Inspection Project by consultant shall meet the requirements set forth in the National Bridge Inspection Standards for all work levels.

A Professional Engineer shall be on-site at all times during the inspection, either in the boat or as an underwater bridge inspection diver.

A Team Leader meeting NBIS qualifications requirements shall be on site at all times during the inspection. The Team Leader must also hold a valid certification as “Bridge Safety Inspector” issued by the Department. The Professional Engineer on site or the underwater bridge inspection diver can also be the Team Leader if they meet all the requirements of this paragraph.

Prior to the start of this work, submit a detailed resume of each inspection team member for approval to the Bridge Inspection Section in the Bureau of Bridge. Team members may be added later provided similar approval has been granted.

**A.** Engineer in charge – the engineer in charge of an inspection and preparation of the inspection report must possess the following minimum qualifications:

a. Be a Professional Engineer licensed in Pennsylvania.

b. Have a minimum of five years' experience in underwater inspection assignments in responsible capacity

c. Must hold a valid certification as a Bridge Safety Inspector issued by the Department.

**B.** Underwater Bridge Inspection Diver – The underwater diver shall:

a. Hold a valid certification as Bridge Safety Inspector issued by the Department AND

b. Successfully completed NHI's Course No. 130091 – Underwater Bridge Inspection

## IX. SAFETY OF PERSONNEL

Safety is of utmost importance. Take all necessary precautions for the safeguard of all personnel involved in this project and follow all applicable Department and OSHA, Part 1910 requirements.

## X. REPORTING CRITICAL DEFICIENCIES TO OWNER

Contact owner and District Bridge Engineer immediately if there is an emergency situation (i.e. critical deficiency). Telephone conversation and notes/sketches sent by email or facsimile will suffice for initial notification.

Place emphasis on discussion of uncorrected critical and other high priority deficiencies brought forward from the previous inspection report. Highlight these deficiencies in the current inspection report.

This task is to be considered incidental to diving and probing inspection work.

## XI. MEETINGS WITH BRIDGE OWNERS

This work is to provide for meetings to interpret and discuss the technical findings of the underwater bridge inspection reports with the bridge owners and/or public.

Discuss critical structural and safety related deficiencies, recommendations and alternatives contained in the report. Make summary notes of the meeting, including any handouts provided, attendance list, items discussed, and any follow-up items needing resolution. Provide three copies of the notes (one copy to owner, District Bridge Engineer and Bridge Inspection Section in the Bureau of Bridge), within one week of the meeting.

## XII. MEETING TO DISCUSS CRITICAL DEFICIENCIES WITH LOCAL OWNERS

Discuss all critical structural and safety-related deficiencies, including posting and repair recommendations, as well as critical and high priority proposed maintenance recommendations and alternatives contained in the current inspection report with the bridge owner at a formal meeting. A meeting is not required for critical deficiencies that involve only missing/damaged weight limit, vertical clearance, or any other regulatory signs. For County bridges, a Commissioners' meeting is appropriate. For Municipalities, arrange for appropriate officials to be present. The contracting agency (such as the County, if applicable) may also attend.

Place emphasis of discussion on uncorrected critical and other deficiencies brought forward from the previous inspection report. Ensure these deficiencies are highlighted in the current inspection report. Prepare informal minutes of the meeting that include attendance, issues discussed, proposed solutions, and needed follow-up items for the deficiencies.

This meeting may also be held to discuss inspection findings, general bridge condition and maintenance needs if requested by bridge owner and authorized by the District Bridge Engineer.

**A.** Convene the meeting within three days after identifying a critical structural deficiency and present a Plan of Action (POA) addressing the deficiency to the owner. For high priority structure deficiencies, the meeting and POA must be conducted within seven days. Refer to Publication 238, IP 2.13.2. and IP 2.14 for information regarding the general requirements for the POA. During the meeting, ensure the owner has a thorough understanding of the critical nature of the defect(s) and the need for timely action as identified in the POA. Attendance by the engineering services consultant for the local Owner is limited to the role of advising, communicating and facilitating the owner's understanding of the deficiency's effects on safety and development of the POA.

**B.** Provide liaison between the District and the owner when it is necessary to take immediate actions, permanent or temporary in nature, to safeguard public safety (e.g. temporary shoring, bridge closing) before the POA is fully developed.

**C.** If more time is needed to develop the POA because of the complexity of the problem, request a POA documentation time extension via e-mail to the District Bridge Engineer, copy to the local Bridge Coordinator and Chief Bridge Engineer, before the end of day 3 for Priority 0's and day 7 for Priority 1's. Include a description of actions taken to date to ensure public safety.

**D.** The Plan of Action shall provide essential information and be structured to match the BMS2 field information for IM01-IM15. The narrative format must show all maintenance actions and schedule required for repairs and restoration of safety to an acceptable level. In addition to the coding IM01-IM05 describing and prioritizing the deficiency in BMS2, address the following fields:

IM06 Date Recommended

IM07 Status of Work Candidate – Identify if the work will be done by the local owner forces or contractor, and whether work has been completed using appropriate coding. Typical codes include 1,2,5,6. (Note: For local and other owners for codes 1 and 5, "Dept" indicates "owner").

IM08 Target Year IM09 Location

IM10 Estimated Cost IM11 Work Assign IM12 Drawing Indicator IM13 Permit Indicator IM14b POA Date

IM14a Date Completed and IM14c Mitigated Date must be entered following completion of the work

IM15a Notes - A brief description of steps taken to address the deficiency which can include closing, posting, restricting, traffic temporary shoring, etc. The actions must restore the structure safety to an acceptable level. Note that additional work may be required at a later time to restore full level of service. A schedule for additional work should be included. Following completion of the immediate work and based on justification included in the plan, record the remaining additional repairs in BMS2 and note as such in item 15b.

**E.** The meeting with the local Owner should not be adjourned until agreement has been reached regarding specific action to be taken and associated schedule. After confirming the finished plan's acceptability with the District Bridge Engineer, enter the appropriate information for each critical deficiency in the BMS2 IM fields (IM01-IM15). Upload documents related to the critical deficiency to the BMS2 Documents link such as narrative version of the plan, sketches, meeting minutes, etc.

**F.** Immediately notify the District Bridge Engineer if the critical deficiency will not be addressed within seven (7) days or if the high priority maintenance item will not be addressed within 6 months.

**G.** Provide follow-up monitoring of the progress toward completion of the POA and report via BMS2 to confirm completion of the approved maintenance action(s) identified in the POA. Follow-up monitoring is also required for regulatory sign critical deficiencies. Enter the completion date(s), IM14a and IM14c, as appropriate in BMS2. The consultant shall immediately notify the owner and the District should problems arise with respect to the completion of the work within the required timeframe.

## XII. SPECIAL UNDERWATER INSPECTION RELATED WORK

The description and requirements of this work will be determined at the time of assignment.

## XIII. NONPROFESSIONAL SERVICES BY OTHERS

Examples of this type of work include, but are not limited to:

a. Use of Geophysical methods to detect scour

b. Nondestructive testing of subsurface elements

c. Coring of wood or concrete

d. Soil sampling of material around units and upstream

These items, when sub-contracted, will be paid as a direct cost for non-professional services. The consultant will be paid the actual cost based on certified invoices.

If the anticipated cost for non-professional services exceeds Ten Thousand and 00/100 Dollars ($10,000.00) per Work Order, solicit three (3) bids for that work. The responsible vendor submitting the lowest bid shall be engaged for the services.

## XIV. METHOD OF PAYMENT

The method of payment for this will be Cost Per Unit of Work, based on the Categories of Work listed in Exhibit 1.

The Cold Weather Add-On Cost will be paid only for those substructures authorized by the Department.

## XV. DEADLINES

Prepare to start work immediately upon receiving Notice to Proceed. Complete all work expeditiously, but not later than the date specified in each Work Order.

## XVI. DEPARTMENT PROJECT MANAGEMENT

PennDOT Project Manager:

Bridge Inspection Section in the Bureau of Bridge

Work Order Administrator and Technical Review:

District Bridge Unit in the Engineering District issuing Work Order.

## XVII. RELEASE OF INSPECTION INFORMATION

Place the stamp appropriate to structure owner per Publication 238, IP 1.8.3 on the front cover of the inspection report. Do not release or distribute inspection information without the written permission of the District Bridge Engineer for State structures or the structure owner.

When portions of a report are approved for release, include the language provided in Publication 238, Figure IP 1.8.3-2 to each page of the structure inspection report that is released.

***Sample Exhibit 1 –*** The following are sample tables, to be created outside of ECMS and attached to the scope of work, as referenced in the Details section of 2.7.5.

## EXHIBIT 1 – LIST OF BRIDGES UNDERWATER INSPECTION

| | | Proposed Scope of Work | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| **BRKEY** | **BMS#** | **MOB Dive < 30'** | **MOB Probing** | **Dive < 15'** | **Probe Unit** | **Report, Initial Dive Unit** | **Report, Additional Units** | **Report, Probe Unit** | **Travel Time Dive Team** | **Units/Comments** |
| 30940 | 53-7230-0435-0301 | 1 | | 1 | | 1 | | | 3 | FAB |
| 30986 | 53-7405-0000-0421 | 1 | | 1 | 2 | 1 | 2 | | 3 | NAB, P01, FAB |
| 43171 | 48-7215-0000-0001 | 1 | | 1 | | 1 | | | 3 | FAB |
| 23530 | 39-7404-0000-9001 | 1 | | 1 | | 1 | | | 3 | P04 |
| 30878 | 53-7207-0753-0073 | 1 | | 1 | | 1 | | | 3 | NAB |
| 23492 | 39-7301-0000-0015 | 1 | | 1 | | 1 | | | 3 | P01 |
| 23484 | 39-7212-0810-9027 | 1 | | 1 | 1 | 1 | 1 | | 3 | NAB/FAB |
| 23503 | 39-7301-0000-9001 | 1 | | 1 | 2 | 1 | 2 | | 3 | NAB, P01, Intrados |
| 23511 | 39-7301-0000-9029 | 1 | | 1 | | 1 | | | 3 | P02 |
| 43171 | 48-7215-0000-0001 | 1 | | 1 | | 1 | | | 3 | FAB, 6 mth interim |
| 5487 | 06-7301-0000-9171 | 1 | | 1 | | 1 | | | 3 | P04 |
| 5366 | 06-7215-0505-9466 | 1 | | 1 | 2 | 1 | 2 | | 3 | P01, P02, P03 |
| | Unforeseen | 1 | 1 | 1 | 1 | 1 | | 1 | 4 | |
| | | | | | | | | | | |

| No. | Critical Deficiency Meetings | Category of Work |
|---|---|---|
| 1 | w/ Union Township | **M1** |
| 2 | w/ Lehigh County | **M1** |

Note: Unless otherwise noted, only one Critical Deficiency Meeting per owner allowed.

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# APPENDIX IP 01-I

Minimum Inventory/Inspection Items for
Non-Highway Bridges over State Routes

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# Minimum Inventory/Inspection Items for Non-Highway Bridges over State Routes

Follow the General Scope of Work – Safety Inspection of State and Local Bridges for the inspection of non-highway structures over State routes. Since the inspection of non-highway structures is not reported to the NBIS, the Department requires less inventory and inspection data be collected for these structures.

**Minimum Required Inventory Data:**

Minimum inventory data includes all BMS2 Items identified with an asterisk(*) on Forms D-491, 1A – 6C and the following BMS2 Items:

| | | | |
|---|---|---|---|
| 6A04 | County/Municipality Boundary | 6A44 | NSTM Group # |
| 5C30 | School Bus Indicator | 6A45-6A48 | Critical Ranking Factor |
| 5C32 | Public Trans Indicator | VD14 | Abutment Type |
| FR06 | # Electrified Tracks | VD15 | Abutment Foundation Type |
| FR03 | Service Status of Railroad | VD16 | Pier Type |
| FR05 | AAR Number | VD17 | Pier Foundation Type |
| FR04 | Rail [Trail] Milepost | 5C01 | Name of Stream<br>(for bridges also over water) |
| 6C10 | Hwy System | VP03 | Spec Restrictive Posting |
| 6C11 | State Network | VP04 | Posted Load Limits |
| 6C24 | Vertical Clearance Sign | VP01 | 1st and Last Date Posted |
| VD19 | Length of Culvert | VP01 | Date of Bridge Closure |
| 5B01 | Bridge Deck Type | VP06 | Reason for Posting |
| SP03<br>SP04 | Spans, Number & Length | | |

Note: Only applicable inventory items need be coded. All data submitted will be stored in BMS2. Owners are encouraged to collect and submit all inventory information they have available.

**Minimum Required Inspection Information:**

1. Minimum inspection data includes all BMS2 Items identified with an asterisk(*) on Forms D-491E and the following BMS2 Items:

| | | |
|---|---|---|
| 4A11 | Underclearance Appraisal | |
| 4A03-4A06 | Safety Features | |
| 7A09 | Inspection Interval | |
| 7A03 | Type of Inspection | |
| 7A05 | [Current] Inspection By | |
| 7A05 | Consultant Name | |
| 6B24 | Agency Hiring Consultant | |
| 6B40 | Deck Wearing Surface Condition | |
| 6B36 | Paint Condition | |
| 3A | Items 3A04-3A08 | |
| 6A, 7A | Items 6A44 and 7A01 | For NSTMs |
| IN, IU | All IN and IU items except IN22 and IN23 | For bridges also over water |

2. The following BMS2 inspection items are not required for non-highway bridges as they relate more to the planning and programming:

| | | |
|---|---|---|
| 4A09 | Structural Condition Appraisal | Not required |
| 4A10 | Deck Geometry Appraisal | Not required |
| 4A02 | Approach Alignment Appraisal | Not required |
| 4B03 | Bridge Capacity Appraisal | Not required |

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# APPENDIX IP 01-J

Guidelines for Preparation
of Safety Inspection Agreements

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## Publication 238 (2024 Edition), Appendix IP 01-J
## Guidelines for Preparation of Safety Inspection Agreements

The following guidelines are to assist in the development of engineering agreements for bridge inspection services using the General Scope of Work (SOW) for Safety Inspection of State and Local Bridges and Tunnels in Appendices IP 01-F and IP 01-G of the Department's Bridge Safety Inspection Manual, Publication 238.

**A. Costs for District Estimates and Price Proposals**

1. The bridge inspection work consists of field work and office work. The District's estimate of costs and the consultant's Price Proposal are to provide the crew hours and costs for field and office work for each major task, and within each major task then subdivide into crew hours and rate of pay by classification of personnel, where applicable. Bridge inspection work consists of:

    a. Professional Services (Direct Payroll Costs)
    b. Direct Costs other Than Payroll
    c. Non-professional Services (Indirect Costs)

2. Field costs include any required cleaning and gauging of members, engineering, and any other incidental items. Office costs include structural analysis, preparation of drawings and reports, and any other incidental items.

3. If a bridge consists of more than one structure type (i.e., main span truss, approach spans-multi-stringer system), show costs for each structure type.

4. For non-professional services such as core borings, rigging, crane rental, traffic control, and laboratory testing, follow the procedures in the latest version of PennDOT Publication 93, Policy and Procedures for the Administration of Consultant Agreements.

5. For professional services such as materials sampling and testing, etc., include copies of the proposals from sub-consultant, with the prime proposal.

(For more details, refer to Publication 93.)

**B. Work Categories**

1. An example of Work Categories is shown in Table 1 at the end of Appendix IP 01-F of Publication 238.

    For the following Types of Inspection Work, Initial Inspections, Routine Inspections, Closed Bridge Inspections and Load Ratings, use the standardized categories of work as shown on Table 1 (see above paragraph). The standardized categories are to be used on all agreements to assist in review of agreements and for inspection programming.

    The Districts may further <u>subdivide</u> these standard work categories if desired. Examples could include A1s to indicate steel stringers, A1P for prestressed girders. However, the first two digits of standard categories must be maintained.

    For Partial/Interim inspections, see next section.

**C. Partial/Interim Inspections**

1. If Method of Payment is **Cost Per Unit of Work** (CPUW), the following mechanism to allow quick agreement on the work required and basis of payment:

    a. Establish a series of interim inspection types in the Agreement with work categories with <u>a specified number of crew hours</u> to be paid. For example:

        <u>Partial/Interim Inspections</u>
        Work Category I1 - 2 crew hours
        Work Category I2 - 4 crew hours
        Work Category I3 - 6 crew hours
        Work Category I4 - 8 crew hours
        Work Category I5 - 10 crew hours
        Work Category I6 - 12 crew hours
        Work Category I7 - 14 crew hours

(The above types are for example only and should be tailored for the specific bridges in your Agreement. When developing the cost for Partial/Interim inspections the Profit shall be limited to 10% of the direct and indirect Payroll costs.)

b. When an interim inspection is needed, the District and consultant must first reach agreement on the SOW for that inspection. Then the Work Category appropriate to that inspection is agreed upon and the work authorized.

In the request letter, the consultant outlines the scope for the interim inspection work at each bridge along with the appropriate work category for District review and authorization.

2. If **COST PLUS FIXED FEE** method of payment is used, a number of Partial/Interim inspections may be set aside in a separate part of the agreement for the purpose of Partial/Interim inspections. Again, the consultant would request authorization via letter and the District would issue the Notice to Proceed for the agreed upon Partial/Interim inspections.

3. Note:

   a. The interim inspections must always be recorded on inspection pages and the data entered into BMS2. Ensure that the report format and requirements are established before authorization.

   b. Partial/interim inspections where data need not be recorded in BMS2 or those that require less than 2 crew hours probably fall under the category of owner's responsibilities and not the NBIS program.

4. For bridges with a straightforward and uncomplicated structure framing plan where interim inspections (such as for a posted stringer bridge) are needed, and if the scope and work category can be established easily, the authorization can be requested and granted through the agreement SOW and list of inspections.

5. More extensive inspections than those above may be better handled under Routine NBIS inspections or Supplemental Inspections.

6. Partial/Interim inspections can be used for flood inspections. Common sense must be used to avoid misuse of federal funding. Mere water depth checks and other cursory inspections do not fall under NBIS.

**D. Supplemental Inspections**

For both CPUW and Cost Plus Fixed Fee agreements, the scope and crew hours for Supplemental Inspections need to be established at time of SOW or must be accomplished through a supplemental engineering agreement.

**E. List of Bridges and Special Requirements**

1. Provide list of bridges by S.R. ID with scope/category of work for each bridge.

   A sample of such a list for a CPUW agreement is attached to Publication 238 Appendix IP 01-F as Exhibit 1. Please note that, on the sample, authorization has been requested for interim inspections on certain bridges and for critical deficiency meetings. Interims/meetings may also be added later on an as needed basis. Specify a list format to meet your needs.

2. Add special instructions for work requirements, project management, emergency communications, M&P Traffic, etc. here. The intent of the SOW should not be altered, but the means/methods of accomplishing that work may be tailored to suit the needs of the various parties.

# APPENDIX IP 02-A

Scour Critical Bridge Monitoring
Field Manual And
High Water Inspection Weekly Report Form

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# SCOUR CRITICAL BRIDGE MONITORING FIELD MANUAL

## PROCEDURES FOR MONITORING SCOUR CRITICAL BRIDGES DURING FLOOD EVENTS

**July 2009**
(Revised 2021)

## TABLE OF CONTENTS

**Scour Critical Bridge Monitoring Field Manual**

## 1.1 Introduction

Pennsylvania has numerous highway bridges that have been identified as Scour Critical, meaning the foundations are at risk of becoming unstable due to potential scour. These Scour Critical bridges will require corrective action, monitoring, or closure, when necessary, in order to protect the traveling public. This manual is intended as a reference for use by field personnel performing monitoring during flood events.

The monitoring personnel for Scour Critical bridges have a critical role with significant responsibilities. As the personnel assigned to specific bridges, the monitoring personnel assess the field conditions during a flood event. To properly perform the assessment, personnel must understand the general nature of scour, know what to look for in the field and properly execute notification processes and a bridge closure should a serious problem be discovered. Serious problems can include overtopping, movement or settlement of the bridge, erosion of approach roadways or stream banks adjacent to the bridge, or heavy debris buildup on the bridge supports.

Section 2 explains basic scour concepts and definitions. Section 3 describes the content of Scour Plans of Action (POA) for Scour Critical bridges. A Scour POA provides important information including background data for the bridge, monitoring recommendations, proposed and completed scour repairs, and scour issues previously identified from bridge inspections. Section 4 describes monitoring and reporting procedures for Scour Critical bridges and associated bridge closures. Section 5 describes emergency procedures.

## 2.1 Definitions and Basic Scour Concepts

Personnel responsible for monitoring Scour Critical bridges should have a basic understanding of scour concepts. This section contains definitions, types of scour, and the factors that influence scour potential. The five key items defined below are Local Scour, Pressure Flow, Scour Critical Bridge, Scour Plan of Action and Significant Flood Event.

### 2.1.1 Definitions

The following is a list of scour related terms and their definitions in alphabetical order:

| | |
|---|---|
| Aggradation | General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition |
| Alluvium | Loose material deposited in the streambed by stream flow |
| Bank | The side slopes of a channel between which the flow is normally confined |
| BMS2 | Bridge Management System 2 – Stores bridge inventory and inspection data for all PennDOT owned bridges 8 feet span and greater and for locally-owned bridges 20 feet span and greater |
| Bridge Watch | Web-based system that tracks multiple data sources, provides real-time precipitation data and notifications for Scour Critical bridges Statewide. System also stores post flood data. |
| Channel | The bed and banks that confine the flow of surface water in a stream |

| | |
|---|---|
| Contraction | The effect of constricting channel flow by a reduction in the waterway width such as that caused by a bridge opening that is narrower than the normal channel width or flood plain |
| Contraction Scour | Scour in a channel or on a flood plain that is not localized at a pier, abutment, or other obstruction to flow. In a channel, contraction scour usually affects all or most of the channel width and is caused by a reduction in the waterway opening. |
| Countermeasure | A measure intended to prevent, delay or reduce the severity of scour |
| Debris | Floating or submerged material, such as logs or trash, transported by a stream |
| Degradation | A general and progressive lowering of the channel bed due to scour |
| Erosion | The wearing away of soil particles on the land surface or along channel banks by flowing water |
| Flood Plain | A nearly flat, alluvial lowland bordering a stream, that is subject to inundation by floods |
| Lateral Migration | Horizontal movement of the channel position due to bank erosion on one side with simultaneous build-up on the other |
| **Local Scour** | **Scour in a channel or on a flood plain that is localized at a pier or abutment or other flow obstruction. Local scour, if unarrested, may result in the exposure or undermining of a footing or foundation.** |
| Overbank Flow | Water movement over top of the banks where water flows into the flood plain |
| **Pressure Flow** | **Where flows through the bridge opening is contracted vertically, submerging or partially submerging the bridge load carrying members and creating increased hydraulic pressure. In other words, water is flowing against the beams of a typical bridge or truss, or the slab of a slab bridge, or the opening of a culvert is completely under water.** |
| RCRS | Road Condition Reporting System: Reports active road closures, lane restrictions, and winter road conditions throughout the State of Pennsylvania. State owned highways are reported on in this system. Turnpikes, toll roads and bridges, and other non-State maintained roads are not included. Bridges closed due to flood damage are reported as a “Bridge Outage”. Bridges closed due to overtopping, pressure flow, or debris buildup can be reported as a “Bridge Precautionary Closure”. |
| Riprap | Rock of a specified gradation which has been engineered and placed to protect a structure or embankment from scour |
| Scour | The result of the erosive action of running water that excavates and carries away material from a channel bed |

| | |
|---|---|
| **Scour Critical Bridge** | **A bridge determined to be Scour Critical through engineering calculations or by observation of certain field conditions** |
| **Scour Plan of Action** | **A document for each identified Scour Critical Bridge that provides a single point reference of the scour inspection information, flood monitoring program, and a schedule for countermeasures to make a bridge safe from scour and stream stability problems.** |
| **Significant flood event** | **A precipitation-based stream response of a magnitude which could endanger the safety of the travelling public because of significant risk of bridge failure due to scour** |
| Streambank erosion | Removal of soil or other particles from a bank surface due primarily to water action |
| Undermining | The erosion or scouring of bed material from beneath the structure foundation or footings |

### 2.1.2 Types of Scour

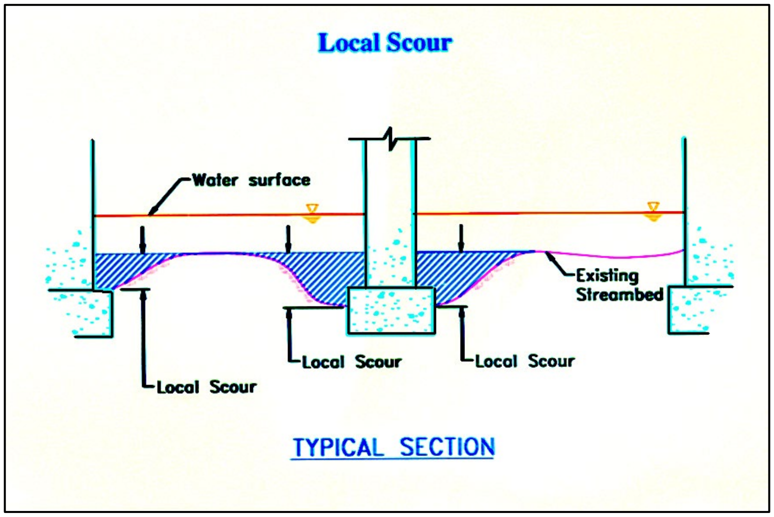
Scour is the result of erosion due to flowing water, removing sediment from the streambed and banks of streams and from around or under supports of bridges. Different types of streambed materials scour at different rates.

Streambed material and streamflow rates are important factors when assessing the scourability at a bridge. Examples of different streambed materials and their scour rates follow:

- Sand and gravel – hours
- Cohesive soils (clay) – days
- Glacial till, sandstone and shale – months
- Limestone – years
- Dense granite – centuries

Scour at bridges can be categorized by one of three types:

1. Local scour at piers and abutments (most common)
2. General or Contraction scour
3. Long term aggradation and degradation




Local scour is scour in a channel or flood plain that is localized at an obstruction. Local scour occurs at piers and abutments and is caused by turbulence at the substructure units creating a localized scour hole which can eventually remove material from under the footing which is known as undermining. The larger the substructure unit width, the greater the turbulence can become which can result in a greater depth of scour. Flow velocity and water depth are also contributing factors to scour depth. Debris accumulation at a substructure unit can increase the apparent size of the unit and lead to more turbulent flow resulting in increased scour.

Contraction scour is a type of general scour caused by a constriction in a channel or flood plain and generally affects the entire channel width. Contraction scour at a bridge is the result of the narrowing of the width of flow as it passes through the bridge opening which creates an acceleration of the flow and results in the removal of streambed material under the bridge and immediately upstream and downstream from the bridge.

General scour is a lowering of the channel bed over time. General scour is different from long-term degradation in that general scour may be cyclic and/or related to the passing of a flood.

Long term aggradation and degradation is the general rise of the streambed by increased deposits of streambed material (aggradation) or uniform lowering of the streambed by scour over a long period of time (degradation).

### 2.1.3 Lateral Migration

In addition to the three types of scour described above, lateral migration (horizontal change in waterway alignment) of the stream should be assessed when evaluating scour at bridges. Lateral migration of the main channel often occurs naturally, but can also be induced or magnified by flooding. A channel moving laterally may affect the stability of piers along the channel bank, can cause erosion behind abutments, damage the approach roadway embankments, or increase the potential for scour by changing the flow direction at piers and abutments.

## 2.2 Tips to Remember Concerning Scour

- Flow Depth – As the flow depth increases, the scour rate may increase.
- Flow Velocity - Faster flow produces deeper scour.
- Pier Width – Wider piers create more turbulence than narrow piers and result in increased local scour depths. Wider piers also contribute to an increase in contraction scour.
- Flood Debris – Debris lodged at a substructure unit will increase the obstructed width of the unit and change its shape. This can result in additional contraction scour as well as additional local scour. The additional turbulence caused by debris can increase scour damage at bridges.
- Flow Alignment – When upstream flow approaches a structure at an angle toward the bridge substructure units, the development of both local and contraction scour can increase rapidly.
- Floodplain – Low banks allow high flows to extend onto the floodplain. When overbank flow is forced back into the main channel or through a bridge opening, contraction scour can occur.
- Pressure Flow – Pressure flow begins when flowing water fills the entire bridge opening, submerging or partially submerging the bridge load carrying members. It can greatly increase velocity and accelerate scour rates.
- Countermeasures - Riprap and other countermeasures may protect a substructure unit from failure due to scour.

## 3.1 Scour Plans of Action (POA) for Scour Critical Bridges

Each scour-critical bridge in Pennsylvania must have an associated Scour Plan of Action (POA) developed. The Scour POA shall include the following:

- General information about the bridge including inventory data (i.e. bridge type, foundation type, location, etc.), scour information, and select inspection data condition ratings.
- The plan of action category and procedures including the monitoring program, scour countermeasures, closure indicators, and bridge specific information.
- Contact information.
- Other pertinent information such as a plan view scour sketch, stream cross-sections, elevation views, and photos.

The elements contained in a Scour POA for a given bridge will vary slightly depending on the Scour Critical Bridge Indicator (SCBI) code. Pennsylvania's scour susceptible bridges are subdivided into four categories. Categories A through C are defined as Scour Critical and are described below.

**Category A** These bridges have an SCBI code (BMS2 Item 4A08) of 2 OR an Observed Scour Rating (BMS2 Item IN03) of 3 or less. Field conditions indicate that extensive scour, including undermining, has occurred and the affected substructure units are at high risk of becoming unstable due to the potential for scour. Category A bridges present a significant safety hazard under high water conditions since additional undermining of the substructure could cause a partial failure or collapse of the bridge in a short amount of time.

The Scour POA for Category A bridges includes monitoring these bridges for distress during significant flood events and for completing a flood monitoring log with entries on the log for each visit to the bridge. Monitoring is initiated based upon notification of bridge outage/closure due to flooding from RCRS, Bridge Watch, County Maintenance, a 911 center, or the police.

Bridges in this category are to be closed at such time as the bridge approach roadway is submerged, or the bridge goes into pressure flow, whichever occurs first. If it is deemed necessary to keep the bridge open while it is in pressure flow then the bridge must be continuously monitored (i.e. placed under a 24/7 watch) in order to assure its safety for use. If it cannot be continuously monitored while under pressure flow, then it <u>must</u> be closed.

**Category B** These bridges have an SCBI code of 3 AND an Observed Scour Rating of 4. Category B bridges are Scour Critical, and the foundation is at medium risk of becoming unstable due to the potential for scour below or within limits of the spread footing or pile tips. In addition, field observations identified one or more of the following: serious scour, erodible streambed material, poor waterway opening and/or poor stream alignment.

The Scour POA for Category B bridges includes a recommendation for monitoring these bridges during significant flood events whenever possible, especially those bridges located on the Interstates and the National Highway System (NHS).

**Category C** These bridges have an SCBI code of 3 AND an Observed Scour Rating of 5 through 7. Category C bridges are scour critical, and the foundation is at low risk of becoming unstable due to the potential for scour below or within limits of the spread footing or pile tips. In addition, field observations identified one or more of the following: advanced scour hole (i.e. very slight undermining), erodible streambed material, and/or increased risk of debris blockage.

The Scour POA for Category C bridges includes a recommendation for monitoring these bridges during significant flood events whenever possible, especially those bridges located on the Interstates and the National Highway System (NHS).

### 4.1 Monitoring Procedures

As bridge flood problems and/or closures of bridges are reported by Bridge Watch, the Traffic Management Center (TMC), County Maintenance organizations , 911 Centers, or police departments, and as the status of closings are reported through the RCRS as a “Bridge Precaution”, “Bridge Outage” or “Flooding”, Department personnel will need to initiate monitoring of scour critical bridges. Step-by-step bridge monitoring procedures are listed in Publication 23, Chapter 9.11.

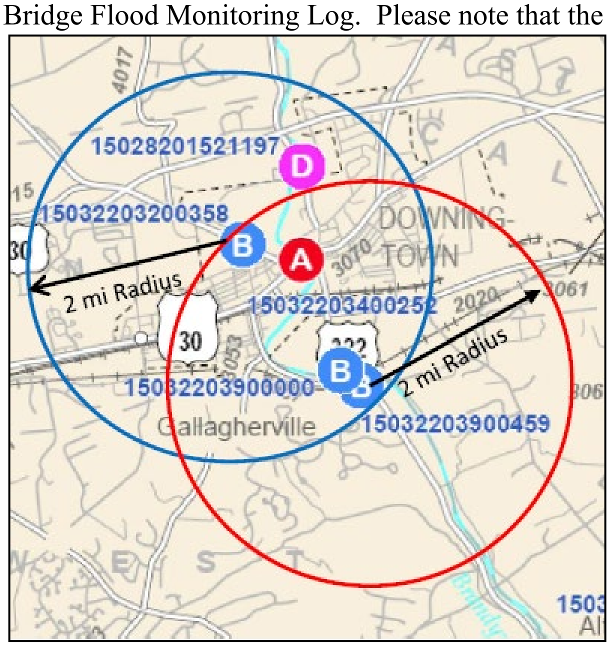
The Districts and counties are to maintain lists of Scour Critical bridges that are to be monitored during significant flood events. These lists are to be generated from a standard Crystal Report titled “Scour Critical Bridge Category List” that is available on Crystal Enterprise. The Districts are to re-generate these lists on a monthly basis and are to supply them to the counties. They are also added to the Bridge Watch program. Districts and counties must use these lists and Bridge Watch as the primary sources to determine which bridges are to be monitored.

GIS maps showing the locations of all Scour Critical bridges in categories A, B and C as well as non-Scour Critical bridges in Category D will be updated by Central Office and provided to the District bridge units every month or can be found in Bridge Watch. Districts and counties can use these maps to locate the bridges to be monitored.

All Category A bridges within the vicinity of the reported bridge flood problems and/or closures are to be monitored. Category B and C bridges should also be monitored depending on the severity of flooding and at the direction of the District Bridge Engineer or the Highway Maintenance Manager. Site visit Scour Critical bridges using one of two (2) methods.

**Method 1** is the current method of monitoring all bridges within a minimum 2-mile radius of the Bridge Watch notification or RCRS reported bridge closure in order to determine whether those bridges meet the requirements for closure per the Bridge Flood Monitoring Log. The visits to scour critical bridges are to be systematically expanded in 2-mile increments beyond any bridge where closure is required. An example of this is as follows:

1) RCRS bridge closure reported as bridge 15032203900459 (a Category B bridge). A site visit to all scour critical bridges within a 2-mile radius of the RCRS reported closure (represented by the red circle) is conducted to determine whether those bridges meet the requirements for closure per the Bridge Flood Monitoring Log. Please note that the Scour Critical Bridge Maps have a scale of 1 inch = 1 mile when printed at full scale. Therefore, a 2-mile radius is anything within 2 inches as measured on the map.



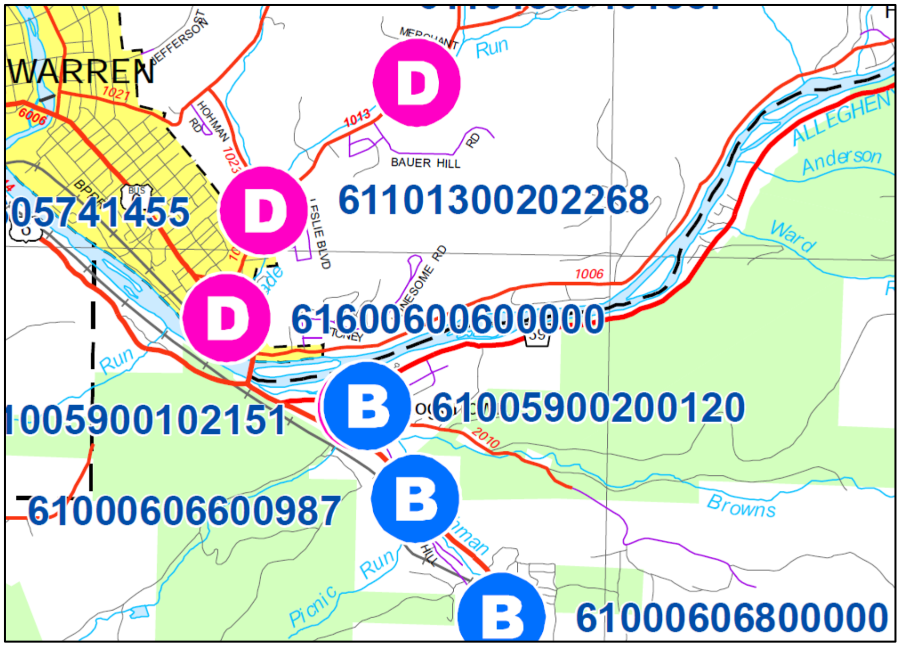

2) Bridge 15032203200358 (northwest of the first reported closure) is determined to meet the requirements for closure per the Bridge Flood Monitoring Log. A two-mile radius from this bridge is to be added (represented by the blue circle). The visits to scour critical bridges are to be systematically expanded in 2-mile increments beyond any scour critical bridge where closure is required.
3) No additional bridges are found that meet the requirements for closure per the Bridge Flood Monitoring Log. All Category A bridges within the red and blue circles are to be monitored. Category B and C bridges should

also be monitored depending on the severity of flooding and at the direction of the District Bridge Engineer or Highway Maintenance Manager.

**Method 2** is monitoring all bridges within the same drainage basin of the Bridge Watch notification or RCRS reported bridge closure in order to determine whether those bridges meet the requirements for closure per the Bridge Flood Monitoring Log. An example of this is as follows:

First bridge closure reported as bridge 61000606600987 (a Category B bridge) reported through a NEXRAD Bridge Watch alert (see sketch below). A site visit to all scour critical bridges within the same drainage basin of the Bridge Watch alert is conducted to determine whether those bridges meet the requirements for closure. Drainage Basin is highlighted in blue.

Bridge 61005900200120 (northwest of the first reported closure) and 61000606800000 (southeast of the first reported closure) are the only scour critical bridges in the same drainage basin. They are monitored according to the monitoring schedule in Publication 238 and Publication 23. No additional bridges outside of the drainage basin are required for monitoring.




The decision to perform monitoring of Category B and C bridges is to be coordinated between the Highway Maintenance Manager and the District Bridge Engineer and is to be based on the reported severity of flooding from monitoring personnel and the availability of adequate personnel to do so. Category A bridges are recommended to be monitored at least once every 4 hours. Category B and C bridges, when monitoring is directed, are recommended to be visited at least once every 12 hours for Category B and 24 hours for Category C.

A Bridge Flood Monitoring Log is to be filled out for each Category A bridge that is monitored. The log is to be updated at each visit to the bridge. Any Category A bridge located on the Interstate system must be monitored without exception and a Bridge Flood Monitoring Log must be filled out at each visit. The monitoring log is very brief and will require minimal effort to maintain. Detailed instructions are attached to the monitoring log and include trigger mechanisms for the need to close the bridge based on the observations logged on the sheet. See Appendix B of this manual for a copy of the Bridge Flood Monitoring Log and instructions for its use.

The personnel performing flood monitoring should be focused on looking for signs of any bridge distress during monitoring efforts. Typical examples include, but are not limited to:

- Overtopping of the bridge deck or approach roadway
- Pressure flow at the bridge (the bottom of superstructure mostly or fully submerged)
- Vertical or horizontal displacement of the superstructure or substructure
- Excessive horizontal or vertical separation at bridge deck joints
- Visible damage to the bridge deck, superstructure, or substructure

- Sinkholes, settlement or erosion in the roadway behind the abutments
- Heavy debris buildup on the superstructure or substructure

If signs of structural distress are apparent at any time, the monitoring personnel should initiate a bridge closure and should avoid getting on the bridge if at all possible. Once a bridge is closed, monitoring of that bridge no longer needs to be performed. Once they are closed, scour critical bridges must not be re-opened until inspected and approval to re-open is received from the District Bridge Engineer.

## 4.2 Reporting

Bridge closures are to be reported by the counties using the RCRS and Bridge Watch. RCRS reports all active bridge and roadway closures. Bridges that have been closed as a precaution, such as those that have overtopped or are experiencing pressure flow, are to be reported in RCRS with a “Cause” description of “Bridge Precautionary Closure”. Those bridges closed due to scour failure are to be reported in RCRS with a “Cause” description of “Bridge Outage”. Note that approach roadway overtopping may occur prior to bridge pressure flow. In these instances, the “Cause” description may not indicate that a Bridge has been closed. Bridge Watch reports scour critical bridge issues. Bridges closed in Bridge Watch should select the appropriate closure option.

- During flood events:
  - Per Publication 23, Ch 9.11, County Managers are to report to the District Bridge Engineer by phone or email on a daily basis. Counties may now use Bridge Watch to update the District Bridge Engineer.
  - District Bridge Engineers are to report all counties where active flood monitoring is occurring to the Chief Bridge Engineer on a daily basis. If bridges are reported in Bridge Watch in real time, then this satisfies this requirement.
- After flood events:
  - Highway Maintenance Managers will assemble and submit monitoring logs per Publication 23, Ch 9.11. These logs may be scanned and submitted electronically. Highway Maintenance Managers may upload these logs to Bridge Watch, or monitor bridges directly in Bridge Watch from a desktop or using the mobile app in lieu of monitoring logs and weekly submittals. Highway Maintenance Manager will have the authority to add local partners as users in Bridge Watch.
  - Using Bridge Watch, the information reported by the County Managers, information reported in RCRS for bridge closures, and rainfall accumulation data, the District Bridge Unit will screen and prioritize the bridges to receive post-flood inspections by bridge inspectors. Scour Critical bridge lists, monitoring data from the counties and Scour POA forms are to be referenced for this effort.
  - District Bridge Engineers are to submit weekly status reports to the BIS identifying the bridges monitored and the inspection results of bridges inspected following flood events. Districts may use Bridge Watch for all reporting in real time in lieu of status reports.

## 5.1 Emergency Procedures

Bridge closures will be the primary course of action taken if site conditions present a safety hazard to vehicular traffic.

### 5.1.1 Bridge Closures

During flood events, monitoring personnel may decide that a bridge closure is necessary if signs of bridge distress noted in Section 4.1 are observed. If the monitoring personnel decide that a closure is necessary, take action immediately. The barriers for the closure and the detour signage are to be installed by PennDOT or local government maintenance forces. After the need for closure has been determined, the monitoring personnel must remain at the bridge site until the bridge closure crew or law enforcement arrives at the scene to take over. If the bridge becomes unsafe for traffic while personnel are waiting for a formal bridge closure, they should perform an emergency closure of the bridge by blocking the road with their vehicle and/or placing cones.

The Highway Maintenance Manager needs to assure that all concerned parties are notified that the bridge is closed. At a minimum, the Assistant District Bridge Engineer for Inspection, the District Bridge Engineer, the RCRS Coordinator and the appropriate law enforcement agencies, State and local, as well as emergency services are to be notified of all closures.

Once a bridge has been closed, it must remain closed until the flood has passed. As soon as possible after the flood, a flood inspection of the bridge foundations, substructure, and channel is to be conducted. For PennDOT-owned bridges the District Bridge Engineer has the authority to reopen the bridge to traffic once it has been assessed as structurally sound.

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# APPENDIX A

## Scour Plan of Action
## (Sample)

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# SCOUR CRITICAL BRIDGE - PLAN OF ACTION

## 1. General Information

| | | | |
|---|---|---|---|
| Structure ID Number (5A01): | District, County, Place (5A04);(5A05);(5A06): | | Inspection Date (7A01): |
| BRKEY (5A03): | Length (5B18): 25 | Facility Carried (5A08): | Feature Intersected (5A07): |
| Year Built (5A15): **1973** | Year Rebuilt (5A16): **0** | Structure Type (6A26-29): **21101 RC SLAB (SOLID)** | |
| Number of Spans: Appr (5B14): **0** | Main (5B11) : **1** | 5A10 Lat: | 5A11 Long: |
| Bridge ADT (5C10): **500** | Bridge ADTT (6C27): **25** | ADT Year (5C11): **2013** | BPN (6A19): **L - Local Net (Non-NHS)** |
| Substructure (1A02): **5** | Channel (1A05): **4** | Waterway Adequacy (1A06): **7** | |

## 2. Scour Information

| | | | |
|---|---|---|---|
| SCBI (4A08): 3 | Source of SCBI Code (IU03): O | Minimum Obs. Scour Rating (IN03): 4 | SCBI Category: B |

| 7A01 Inspection Date | 5D02 Structure Unit | IN13 Foundation Type | IN03 Observed Scour Rating | IU27 SCBI CODE | IN18 Water Depth | IN24 Underwater Inspection Notes: |
|---|---|---|---|---|---|---|
| 09/19/2019 | FAB | P - Researched/unknown | 4 | 3 | 2.0 | 4/2019 Inspection: No significant changes. Rock is noted along the entire abutment. Rock was previously noted to seem small and could be washed out during high water events, however no significant change is noted since the previous high water event. The toe is not exposed along the abutment. No UM/Scour is evident along the FAB. |
| 09/19/2019 | NAB | P - Researched/unknown | 4 | 3 | 5.0 | Infilling of stream bed material has decreased the length of footing exposure since the previous underwater inspection. The footing is now exposed for a length of 12.5' along the right (upstream) end of the breastwall and partially along the right wingwall. The top of the footing was located 3.9' below the current waterline and the maximum height of exposure measured 1.6'. No undermining was detected. The downstream half of the breastwall and left wingwall are lined with rock protection and the footing was not detected. The rock protection extends to 5.0' from the abutment. The concrete exhibits light scaling and a few hairline vertical cracks. The streambed in the vicinity of the element consists of silt, sand, gravel, cobbles and rock protection. Probing the streambed resulted in penetrations of up to 0.4'. Soundings indicate advanced scour at the element. The channel flow was parallel with the abutment. |

## 3. Monitoring Program (During Event)

**Monitor Category B bridges at a minimum of twelve (12) hour intervals. Bridge Watch and Scour Critical Bridge Maps may be used as a means for locating these bridges.**

**The Scour POA for Category B bridges includes a need for monitoring these bridges in flooded areas whenever possible, especially those bridges located on the Interstates and the National Highway System (NHS). The recommended frequency for monitoring is once every 12 hours. Note that a minimum of 2 visits are required and monitoring is no longer needed after the bridge is closed. Monitoring of a bridge may be discontinued when the water level is at least two feet below the bridge load carrying members and after it has been clearly observed that the flood waters are receding, i.e., at the time of the visit to the bridge, the water level is now lower than that observed at the previous visit.**

**Local owned bridges: The local owner is responsible for flood monitoring and shall coordinate with PennDOT's local bridge coordinator to determine the bridges requiring monitoring.**

**Note: Slight chance of overtopping bridge deck and roadway approaches**

## 4. Post-Flood (After Event)

A post-flood damage inspection is to be performed after each significant flood event that is experienced in the watershed where this bridge is located. As a guide, the 50 year recurrence interval storm event may be used as a trigger for the need for inspection. If the bridge has been closed due to overtopping of the approach roadway or bridge, or due to pressure flow, then the bridge must be inspected for scour damage before being re-opened to traffic.

Local bridge owners shall coordinate with PennDOT's local bridge coordinator to coordinate the post-flood inspection effort.

## 5. Countermeasures

Any Proposed Maintenance listed below is to be installed in accordance with a schedule consistent with the Priority Level assigned. Proper completion of these maintenance items should protect the bridge from further scour damage or failure.

### Proposed Maintenance

| IM03 Action | IM05 Priority | IM06 Date Recomended | IM07 Status | IM08 Target Year |
|---|---|---|---|---|
| C745301-BKFL.SCOUR HOLE | 3 - Add to Schedule | 04/24/2003 | 0 - Work not planned | 2018 |
| B745301-CONST RCK PROTECT | 2 - Priority | 04/29/2013 | 0 - Work not planned | 2018 |

### Completed Maintenance

| IM03 Action | IM14a Completion Date | IM18 Actual Quantity |
|---|---|---|

## 6. Bridge Closure Plan

Scour Monitoring criteria for consideration of bridge closure:

- **Pressure flow at the bridge (the bottom of superstructure mostly or fully submerged in water)**
- **Water overtopping the bridge deck or approach roadway**
- **Excessive vertical tilt, settlement or horizontal movement of the superstructure or substructure**
- **Excessive horizontal or vertical separation at bridge deck joints**
- **Visible damage to the bridge deck, superstructure or substructure caused by flood waters or floating debris**
- **Sinkholes, settlement or erosion in the approach roadway or loss of roadway embankment**
- **Heavy debris accumulation at or on the bridge severely restricting water flow through the bridge**
- **Washout of rock protection near the bridge substructure that indicates severe scour of the bridge**

Maintenance Responsinility (5A20): 03 - Town/Twp Hwy Agency    Owner (5A21): 03 - Town/Twp Hwy Agency

**Contact Persons Information:**

**District Bridge Engineer**, ACMM, CMM

Name: Phone: Email:

**Local Bridge Coordinator**

Name: Phone: Email:

**Local Bridge Owner**

Name: Phone: Email:

Criteria for re-opening the bridge: After scour inspection performed by Qualified CBSI Inspection Team Leader and approval by the District Bridge Engineer (State bridges) or the Local Owner (non-state bridges).

## 7. Attachments

Please indicate which materials are being submitted with this POA:

- ☐ Attachment A: Plan view scour sketch showing location of scour holes, undermining, debris, etc.
- ☐ Attachment B: Cross sections from current and previous inspection reports which includes substructure unit elevations showing existing streambed, foundation depth(s) and observed scour depths.
- ☐ Attachment C: Map showing detour route(s) (Optional)
- ☐ Attachment D: Photos (Optional)

## 8. Monitoring Log

**SAFETY FIRST - DO NOT ENDANGER YOURSELF OR OTHERS WHILE MONITORING BRIDGES**

**DO NOT ENTER EITHER FLOWING OR STANDING FLOOD WATERS WHILE MONITORING SCOUR CRITICAL BRIDGES**

GENERAL INSTRUCTIONS: Visually examine the bridge and approach roadway each time the bridge is visited. Also look at the upstream and downstream sides of the bridge and waterway channel. Circle the appropriate response for items inspected. Leave blank if not inspected. Circle **"N"** for no, none or no change in condition since beginning of flood monitoring. Circle **"Y"** for yes, when appropriate, based on the descriptions below. Provide a written explanation for a **"Y"** response in the **"Remarks"** section as to what was observed or has changed.

**Should you believe that the bridge is becoming unsafe for any reason, immediately close the bridge. When a "Y" response has been circled, the bridge must be closed. If it is deemed necessary to keep a bridge open for emergency vehicle passage or for emergency evacuations, then the bridge must be monitored at all times (24/7).**

**In order to re-route emergency response vehicles after a bridge has been closed, immediately notify appropriate law enforcement, local emergency responders and county emergency communications center using a non-emergency phone number or pre-established alternate communications. AVOID DIRECTLY CALLING 911 UNLESS THERE IS AN ACTUAL EMERGENCY. Closed bridges may be reopened only after a post-flood bridge safety inspection has been completed by qualified bridge inspectors and only after approval by a professional engineer. Notify PennDOT district office municipal services or bridge inspection personnel of bridge closures whenever possible.**

MONITORING PERSONNEL and TIME: Record name(s) of person and time of monitoring (circle "A" for AM or "P" for PM)

BRIDGE:

- **Pressure Flow** - Has the water level reached the bottom of the bridge beams or the bottom of the bridge deck?
- **Alignment** - Sight along bridge beams, railing, curb, etc., for horizontal misalignment. Check specifically at the joints in the bridge over the piers or at the abutments. If the joint is wider at one side of the bridge than at the other (difference of 1/2 inch or more), when movement of the bridge due to scour should be suspected and the bridge should be closed.
- **Settlement** - Sight along bridge beams, railing, curb, paint striping, etc., for vertical misalignment. Any noticeable dip at a pier or drop at an abutment indicates settlement is occurring and the bridge must be closed immediately.
- **Tilt** - Check the abutments and piers for plumb. If there is any noticeable tilt side-to-side or forward or back then the bridge should be closed immediately.

APPROACH ROADWAY:

- **Settlement** - Check approach pavement for settlement. Water piping through the approach fill can cause erosion under the approach pavement causing the pavement to settle. Does it appear that there is new settlement or are there holes in the pavement?
- **Embankment Erosion** - Check approach roadway embankment slopes, shoulders, and edge of pavement for erosion. Extend limits of inspection to cover sections of the roadway that are parallel to the stream. Does a washout exist on the roadway embankments or shoulders?

WATERWAY CHANNEL:

- **Debris Build-up** - Check the bridge waterway opening for accumulation of trees, branches or other debris that severely restricts the flow of water and creates strong pressure on the bridge.

**SAFETY FIRST** - DO NOT ENDANGER YOURSELF OR OTHERS WHILE MONITORING BRIDGES

**File Scour Critical Bridge Monitoring Logs with other bridge inspection records at the municipality. Monitoring Logs are subject to review by the Federal Highway Administration (FHWA).**

Struct ID: Date:_____/_____/_______

BRKEY:

| | | | Bridge | | Roadway | | Channel |
|---|---|---|---|---|---|---|---|
| Monitoring Personnel | Time | AM or PM | Pressure Flow | Alignment / Settlement / Tilt | Settlement | Embankment Erosion | Debris Buildup |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |

**Bridge Closed:** Date:_____/_____/_______ Time:________:__________

# APPENDIX B

## Bridge Field Monitoring Log

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# Bridge Flood Monitoring Log

**SAFETY FIRST** - DO NOT ENDANGER YOURSELF OR OTHERS WHILE MONITORING BRIDGES

**File Scour Critical Bridge Monitoring Logs with other bridge inspection records at the municipality.**
**Monitoring Logs are subject to review by the Federal Highway Administration (FHWA).**

Struct ID: BRKEY: Date:____/____/____

SR/SEG/OFF: County: Stream:

| | | | Bridge | | Roadway | | Channel |
|---|---|---|---|---|---|---|---|
| Monitoring Personnel | Time | AM or PM | Pressure Flow | Alignment / Settlement / Tilt | Settlement | Embankment Erosion | Debris Buildup |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |
| | | A / P | Y / N | Y / N | Y / N | Y / N | Y / N |
| Remarks: | | | | | | | |

**Bridge Closed:** Date:____/____/____ Time:_____:_____

# Bridge Flood Monitoring Log Instructions

## SAFETY FIRST - DO NOT ENDANGER YOURSELF OR OTHERS WHILE MONITORING BRIDGES

**DO NOT ENTER EITHER FLOWING OR STANDING FLOOD WATERS WHILE MONITORING SCOUR CRITICAL BRIDGES**

GENERAL INSTRUCTIONS: Visually examine the bridge and approach roadway each time the bridge is visited. Also look at the upstream and downstream sides of the bridge and waterway channel. Circle the appropriate response for items inspected. Leave blank if not inspected. Circle **"N"** for no, none or no change in condition since beginning of flood monitoring. Circle **"Y"** for yes, when appropriate, based on the descriptions below. Provide a written explanation for a **"Y"** response in the **"Remarks"** section as to what was observed or has changed.

**Should you believe that the bridge is becoming unsafe for any reason, immediately close the bridge. When a "Y" response has been circled, the bridge must be closed. If it is deemed necessary to keep a bridge open for emergency vehicle passage or for emergency evacuations, then the bridge must be monitored at all times (24/7).**

**In order to re-route emergency response vehicles after a bridge has been closed, immediately notify appropriate law enforcement, local emergency responders and county emergency communications center using a non -emergency phone number or pre-established alternate communications. AVOID DIRECTLY CALLING 911 UNLESS THERE IS AN ACTUAL EMERGENCY. Closed bridges may be reopened only after a post-flood bridge safety inspection has been completed by qualified bridge inspectors and only after approval by a professional engineer. Notify PennDOT district office municipal services or bridge inspection personnel of bridge closures whenever possible.**

MONITORING PERSONNEL and TIME: Record name(s) of person and time of monitoring (circle "A" for AM or "P" for PM)

BRIDGE:

- **Pressure Flow** - Has the water level reached the bottom of the bridge beams or the bottom of the bridge deck?
- **Alignment** - Sight along bridge beams, railing, curb, etc., for horizontal misalignment. Check specifically at the joints in the bridge over the piers or at the abutments. If the joint is wider at one side of the bridge than at the other (difference of 1/2 inch or more), when movement of the bridge due to scour should be suspected and the bridge should be closed.
- **Settlement** - Sight along bridge beams, railing, curb, paint striping, etc., for vertical misalignment. Any noticeable dip at a pier or drop at an abutment indicates settlement is occurring and the bridge must be closed immediately.
- **Tilt** - Check the abutments and piers for plumb. If there is any noticeable tilt side-to-side or forward or back then the bridge should be closed immediately.

APPROACH ROADWAY:

- **Settlement** - Check approach pavement for settlement. Water piping through the approach fill can cause erosion under the approach pavement causing the pavement to settle. Does it appear that there is new settlement or are there holes in the pavement?
- **Embankment Erosion** - Check approach roadway embankment slopes, shoulders, and edge of pavement for erosion. Extend limits of inspection to cover sections of the roadway that are parallel to the stream. Does a washout exist on the roadway embankments or shoulders?

WATERWAY CHANNEL:

- **Debris Build-up** - Check the bridge waterway opening for accumulation of trees, branches or other debris that severely restricts the flow of water and creates strong pressure on the bridge.

# APPENDIX C

## Sample Scour Monitoring Inspection Forms

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# Scour Monitoring Inspection Form Template

BRKEY: ____ Inspection Team: ____

BMS #: ____ ____

Scour Category: ____ Inspection Date: ____

SCBI: ____ Storm Event Date(s): ____

1. Was the bridge closed during the event?

   ☐ Yes ☐ No

2. Was the approach roadway closed during the event (reason not related to structure)?

   ☐ Yes ☐ No

3. Was the bridge overtopped during the event?

   ☐ Yes ☐ No ☐ Unknown/Not enough evidence

4. Was the approach roadway overtopped during the event?

   ☐ Yes ☐ No ☐ Unknown/Not enough evidence

5. Did the bridge experience pressure flow during the event?

   ☐ Yes ☐ No ☐ Unknown/Not enough evidence

6. Determine the high water mark with respect to a permanent landmark or bridge element. Identify the landmark, location in plan, and approximate vertical distance to the high water mark. Generally, try to use the bottom of bridge beam/slab as the landmark for consistency between inspections.

   ____

   ____

   ____

7. Is there NEW damage to approach roadway? (If yes, describe)

   ☐ Yes ☐ No

   Remarks: ____

   ____

8. Is there NEW damage to bridge? (If yes, describe. Initiate a Damage Inspection in BMS)

   ☐ Yes ☐ No ☐ Unknown - Follow up needed (Critical areas inaccessible)

   Remarks: ____

   ____

# Scour Monitoring Inspection Form Template

9. Is there NEW debris present? (If yes, describe)

- [ ] Yes, impacts hydraulic opening
- [ ] Yes, but does not impact hydraulic opening
- [ ] No

Remarks: ______

10. Is there NEW scour present? (choose worst case and describe)

- [ ] Present at subunit(s), major
- [ ] Present at subunit(s), moderate
- [ ] Present away from subunits
- [ ] No change
- [ ] Change unknown, NO undermining
- [ ] Probing not completed - Follow up needed (not accessible)

Remarks: ______

11. Is a Priority 0 or Priority 1 Maintenance Item warranted? (Check all that apply. If yes, describe and initiate a Damage Inspection in BMS)

- [ ] Yes, Damage related
- [ ] Yes, Debris related
- [ ] Yes, Scour related
- [ ] Yes, Other (describe)
- [ ] No

Remarks: ______

12. Do you feel this Scour Monitoring Inspection is warranted?

- [ ] Yes, significant damage or pressure flow or overtopping
- [ ] Yes, despite no significant damage or pressure flow or overtopping
- [ ] Yes, needed despite lack of alert
- [ ] No, request threshold review (email this form to PD-BridgeInspectSection@pa.gov)

Additional Remarks:

______

______

______

# APPENDIX IP 02-B

(Not Used – Reserved for Future Use)

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# APPENDIX IP 02-C

Emergency Bridge Restrictions and
Special Hauling Permits Action Plan

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**Publication 238 (2024 Edition), Appendix IP 02-C**
**Emergency Bridge Restrictions and Special Hauling Permits Action Plan**

When a bridge is no longer able to carry its intended loads it is imperative, for public safety, to prevent further damage or collapse by controlling traffic on the bridge. The need to prevent overloads on a weakened bridge justifies a thorough and urgent response.

For such situations, the Department may impose emergency restrictions on the bridge that include closing, vehicle weight restrictions, lane closures, prohibition of permitted vehicles, and other traffic control deemed necessary. The emergency actions (determined by the District Bridge Engineer) depend upon the bridge conditions and, in large part, to the likelihood of overloads. Because Special Hauling Permits are issued in advance of the actual move, it is more difficult to prevent overloads by already permitted vehicles than traffic generally. Previously approved Permits can be grouped as follows: 97.42% = “single-trip” Permits; 1.91% = annual, “network” Permits (almost entirely international containerized cargo); 0.66% = seasonal/annual, “specified route” Permits; 0.01% = seasonal/annual, “blanket” Permits.

Department Permit and Bridge staffs have gained experience by addressing past emergency bridge restrictions. This Action Plan identifies up to nine steps that may be needed to improve response time and communication to motor carriers operating under various Permits when there is a future emergency bridge restriction, particularly when the affected bridge is carrying an Interstate highway or major Traffic Route.

It is anticipated that all nine steps will need to be pursued in response to an emergency bridge restriction when the bridge is carrying an Interstate highway or major Traffic Route.

| | *ACTION* | *RESPONSIBLE PERSON* | *EXPECTED RESULTS* | *PROS* | *CONS* |
|---|---|---|---|---|---|
| 1 | Establish need for emergency bridge restriction. | District Bridge Engr. | Makes recommendation for:<br>▪ Closing<br>▪ Weight Limit<br>▪ Lane Closure<br>▪ Permit Restrictions<br>▪ Urgency | Maintains public safety. | None. |
| 2 | Establish need for Variable Message Signs (VMS) to notify drivers en route.<br><br>VMSs should be considered:<br>▪ On Interstates and Traffic Routes.<br>▪ On other routes where ADTT > 200.<br>▪ Until more permanent solution is in place.<br>▪ As alternative to more stringent restriction to bridge (Dist. Bridge Engr.).<br>▪ For advance locations, & bridge site.<br>▪ Parking for larger permitted vehicles may be needed in advance of structure until detour is established and Supplements are issued.<br><br>Standard Emergency Closure / Detour Signs with weight *stickers* prepared in advance (e.g., Emergency Bridge Restriction & No Vehicles Over *80,000* lbs. Beyond Next Exit) should be considered:<br>▪ On all State highways.<br>▪ Until restriction is lifted.<br>▪ In addition to VMSs. | Ad hoc committee:<br>▪ District Executive/ Administrator.<br>▪ ADE-Maintenance.<br>▪ Dist. Bridge Engr.<br>▪ Dist. Traffic Engr.<br>▪ Dist. Permit Mgr.<br>▪ Dist. CRC.<br>▪ County Mgr. | Provides notification -- in advance of the new emergency bridge restriction -- to all motor carriers in route to restricted bridge. | ▪ Timely, cost effective action.<br>▪ Last chance to notify driver.<br>▪ Difficult for driver to ignore.<br>▪ 100% prompt notification to affected drivers.<br>▪ Reduces risk and costs of further damage to bridge (when signs are obeyed).<br>▪ Maintains public safety.<br>▪ Consistent with Permit Restriction. | ▪ Some time lag until VMS can be assigned to site and made operational.<br>▪ Suitable parking for larger permitted vehicles may not be available.<br>▪ Drivers *may* not obey message signs. |

| | *ACTION* | *RESPONSIBLE PERSON* | *EXPECTED RESULTS* | *PROS* | *CONS* |
|---|---|---|---|---|---|
| 3 | Post emergency bridge restriction in APRAS database. | District Permit Office. | Assures that emergency bridge restriction will be included immediately as part of analysis of *new* route-specific permit applications. | ▪ Quick, cost effective action.<br>▪ Automatically addresses new route-specific applications.<br>▪ Maintains public safety. | ▪ Internal database update only.<br><u>Does not address:</u><br>▪ Unused single-trip Permits issued during previous 14 days.<br>▪ Seasonal & Annual Permits (see # 7, 8, 9). |
| 4 | Post Administrative "flash" Message about the emergency bridge restriction and need for permittees to obtain route-correction Supplements for previously approved Permits that have not yet crossed restricted bridge. | District Permit Office. | Notifies PennDOT Permit and Bridge staffs of emergency bridge restriction within 10 minutes. | ▪ Quick, cost effective action.<br>▪ Allows internal system users to manually address new applications. | <u>Does not directly address:</u><br>▪ Unused single-trip Permits issued during previous 14 days.<br>▪ Seasonal & Annual Permits (see # 7, 8, 9). |
| 5 | Post WEB Administrative "flash" Message about the emergency bridge restriction and need for permittees to obtain route-correction Supplements for previously approved Permits that have not yet crossed restricted bridge. | Central Permit Office. | Notifies APRAS WEB users of emergency bridge restriction within 20 minutes of initial Administrative Message posting. | ▪ Quick, cost effective action.<br>▪ Allows external system users to manually address new applications. | APRAS WEB users may not identify every previously approved Permit that has not yet crossed restricted bridge. |
| 6 | Send e-mail of Administration Message to APRAS WEB users, again stressing need for permittees to obtain route-correction Supplements for previously approved Permits that have not yet crossed restricted bridge. | Central Permit Office. | Promptly notifies APRAS WEB users with e-mail addresses of restriction. | Additional communication to some APRAS WEB users. | APRAS WEB users may not identify every previously approved Permit that has not yet crossed restricted bridge. |

| | *ACTION* | *RESPONSIBLE PERSON* | *EXPECTED RESULTS* | *PROS* | *CONS* |
|---|---|---|---|---|---|
| 7 | Temporarily deactivate Permit Load Types associated with affected "Network(s)", until first business day of following week. | Central Permit Office. | Assures that emergency bridge restriction will be included as part of analysis of *new* "Network" permit applications. | ▪ Quick, cost effective action.<br>▪ Automatically addresses new affected "Network" applications.<br>▪ Maintains public safety. | Tardy "Network" Permit renewals will not be processed until after weekend "Network" re-analysis and all District Permit and Bridge Unit manual reviews of updated Network(s). |
| 8 | Run "Bridge Crossing" report, using the BMS2 query function.<br><br>Determine risk from still active, previously approved Permits and make follow-up contacts to permittees. | <u>Run Report:</u><br>BDTD or<br>Central Permit Office.<br><br><u>Contact Permittees:</u><br>Affected District Permit Office. | Identifies previously approved Permits that are still valid, including:<br>▪ Annual NON-Blanket Permits.<br>▪ Seasonal Permits.<br>▪ Single-trip Permits issued within past 14 days.<br>Affected District's staff contacts Permittees about still active, previously approved Permits. | ▪ Affected Permittees advised NOT to travel across restricted bridge (and to obtain route-correction Supplements).<br>▪ Eliminates some roadside waiting for Supplements. | ▪ BMS2 query database updated on weekend refresh only.<br>▪ Labor-intensive.<br>▪ Blanket Permits not addressed (see # 9). |
| 9 | Run report to identify active Blanket Permits.<br><br>Initiate mass mailing to Blanket Permittees to inform them of emergency restrictions. | Central Permit Office staff with access to the BMS2 query function can run "reports", including mailing labels. | Notifies Blanket Permittees (who travel non-specified routes). | Assures Blanket Permittees are notified. | ▪ BMS2 query database updated on weekend refresh only.<br>▪ Labor-intensive. |

# APPENDIX IP 02-D

General Scope of Work –
Safety Inspection of Sign Structures

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# I. GENERAL

## A. Purpose

The purpose of this inspection program is to verify each sign structure's inventory data, to determine its physical condition and maintenance needs, and record the information in the Department's Bridge Management System 2 (BMS2). In addition, the scope shall include performing limited bolt replacements and/or tightening as needed and replacing bulbs on lighted signs attached to sign structures.

This document is to provide methodology and procedures for those inspections.

## B. References

The requirements of the latest versions of the Department accepted AASHTO and FHWA manuals and the latest versions of Department Publications and Policy, including any updates, shall be followed in the performance of the Scope of Work. See the Department's Bridge Safety Inspection Manual, Publication 238, IP 1.3.2 and IP 1.3.3 for a list.

# II. TYPES OF SIGN STRUCTURES

Sign structures are typically constructed of either galvanized steel or aluminum. There are also some painted and unpainted weathering steel structures.

## A. The five (5) basic types of sign structures in Pennsylvania:

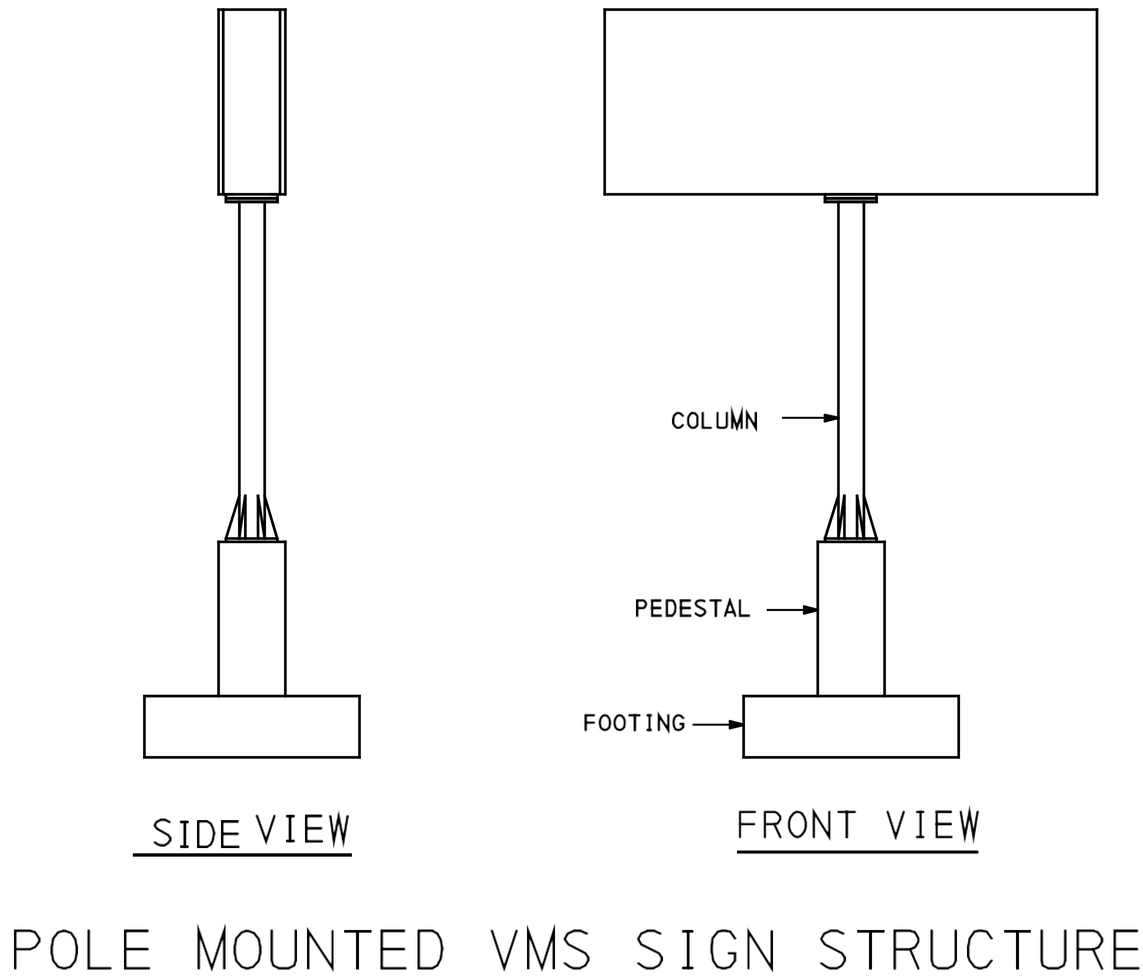
1. **Overhead** - consisting of one or more horizontal members supported at each end. Overhead structures may be multi-span. Subtypes include: planar trusses, 3 or 4 chord trusses, tubular, and rigid frame structures (See BD-642M through BD-645M or BC-742M through BC-745M for examples).
2. **Cantilever** - consisting of one or more horizontal members supported at only one end (See BD-641M or BC-741M for examples)
3. **Center Mount** - consisting of one or more horizontal members supported at the center, often used to support Variable Message Signs (VMS) (See BD-641M or BC- 741M for examples).
4. **Pole-Mounted** – Used exclusively for VMS (see next section), pole mounted sign structures typically have a mounting plate welded to the top of the single column support. The structural framework of the VMS is mounted to a similar sized base plate that bolts directly to the pole mounting plate (See sketch below).
5. **Structure-Mounted** - a sign attachment permanently mounted to the fascia beam and/or face of parapet to be visible to traffic beneath the bridge. These signs are typically inspected during the bridge's NBIS inspections.

Sign Structures that are erected on bridge piers, parapets, brackets, etc., that are visible to traffic on the bridge are to be classified according to type (type 1 thru 4 above) and not as Structure-Mounted. They must be inventoried separately from bridges.

## B. Variable Message Signs (VMS)

Variable Message Signs (also known as Dynamic Message Signs) consist of modern Light Emitting Diode (LED) display boards that can be remotely controlled to provide real-time traffic updates or convey other critical information to motorists as needed. VMS signs require a housing and often have their own "walk-in" structural cabinet to contain the various electronics needed to power the sign. Therefore, VMS are typically much larger and heavier than standard flat panel signs. Failure of a VMS support structure has a much greater potential for catastrophe than that of standard flat panel signs. For this reason, sign structures supporting VMS require special attention during inspections.

EXHIBIT A




## III. TYPES OF SIGN STRUCTURE INSPECTION WORK

There are 5 inspection types, all of which include close visual and hands-on examination of the sign structures. A brief description of each of these is given below:

**A. Initial Inventory**
Insufficient or no data is available in BMS2. An inspection fulfilling the Department's sign structure safety inspection requirements has never been performed. This type of inspection provides for the collection of a sign structure's inventory data for entry into the Bridge Management System 2 (BMS2). All items included on BMS2 Coding Forms D-491 must be completed. This work includes an In-depth inspection as described below.

**B. In-depth**
A close visual and hands-on examination of each component, member, fastener, and weld on the structure, and/or non-destructive field tests, and/or material tests are performed to fully ascertain the existence of or the extent of any deficiency. Lane closures are anticipated to permit access to all portions of structure. Existing inventory data is to be updated

**C. In-depth (Alternate Lanes Closed)**
A close visual and hands-on examination of column bases, end supports, and selected portions of horizontal members. Areas of horizontal members to have close hands-on inspection, and/or non-destructive field tests, and/or material tests performed to fully ascertain the existence of or the extent of any deficiency, are selected to provide overall safety while minimizing traffic disruption. Existing inventory data is to be updated.

**D. Routine**
A close visual and hands-on examination of all portions of the sign structure. Lane closures are anticipated to permit access to all portions of structure. Ladders can be used to access end supports away from traffic. Existing inventory data is to be updated.

**E. Special Inspections**
An inspection to provide in-depth assessment of special conditions when significant structural deficiencies, severe section loss, collision damage, or corrosion have been noted. These inspections will be performed as directed by the District Bridge Engineer.

Inspection types B, C, D and E are done subsequent to the initial inventory inspection and involve only a cursory review of the inventory data to verify correctness. These four different levels of effort can be used to evaluate the sign structure based on its condition and inspection history.

## IV. INSPECTION INTERVALS

The inspection interval and level of inspection intensity for sign structures are influenced by structure material, structure type, condition and age. Table IP 2.11.3-1 in Publication 238 establishes the inspection intervals for the various types of sign structures. Structure-Mounted sign structures are to be inspected along with the other bridge components as part of the biennial NBIS safety inspections. Table IP 2.11.3-2 in Publication 238 lists the typical cycles for conducting safety inspections for different sign structure types and varying conditions.

## V. INSPECTION REQUIREMENTS

**A. Inspection Procedure**
Inspect in accordance with Section VII of this SOW entitled Field Inspection Procedures. Clearly record all inspection field notes on D-491 Forms and on BMS3 Inspection Pages. Prepare sketches, if required. Obtain digital photographic documentation. Refer to Bridge Management System Coding Manual, (Publication 100A) and its updates for completing appropriate portions of forms. Precisely identify all areas of section loss and comment on their impact on the support's structural strength. Perform dye penetrant testing if needed. Indicate maintenance items needed and their priorities on D491 Forms and in BMS3. Complete, update, or amend the required BMS2 inventory and inspection items on the printout of the BMS2 records. *Districts, when practical, are to select groups of sign structures along a given corridor when preparing Work Orders.*

**B. Fastener Installation**
Replace missing or damaged fasteners as required. Select replacement fasteners with closest diameter and size match from hardware provided by the Department. Note that tightening of existing bolts is considered incidental.

**C. Light Bulb Replacement**
Replace all bulbs on lighted signs attached to overhead sign structures (including cantilever sign structures) during Routine Inspections. Light bulbs are to be provided by the Department. The need for routine bulb replacement to be included in Routine sign structure inspections is to be determined by each individual District.

## VI. PRE-INSPECTION DATA COLLECTION AND INVENTORY INFORMATION

### A. Initial Inspection and General Structure History

Prior to the start of field inspections, obtain the following information for each structure:

1. Existing plans or sketches. For VMS's obtain plans or sketches for both the sign housing and the supporting structure.
   a. Design drawings or applicable Standard with dimensions added (BMS2 Item VN05), including any retrofits since the initial construction (i.e. replacement of standard flat panel sign with a VMS).
   b. Shop drawings, including drawings for retrofits (BMS2 Item VN06).
   c. Supplemental sketches with photographs. When shop drawings are not available, provide detailed sketches with dimensions.
2. Map with location of structure - include both a Type 10 County Map and an Interchange Map.
3. Date structure was built (BMS2 Item 5A15). Leave blank if no drawing is available. As a last resort, District Inspection Unit will provide this date using engineering judgment based on when the section of surrounding highway was constructed.
4. The following items are usually available on roadway plans and/or shop drawings, and should be recorded in narrative fields (BMS2 IS Screen):
   a. Material - type of metal (also alloy if known)
   b. Name of fabricator and plant location
   c. Designer/Consultant of Record - provide this narrative on the IS Screen
5. Any records of damage and/or repairs to structure (provide sketches and/or photographs).

### B. Subsequent Inspections

1. Obtain a copy of all previous inspection reports for all sign structures being re-inspected.
2. Inspector to mark any necessary inventory changes on the D-491 forms and provide to the Department for revision.

### C. Referencing Nomenclature

Use the following referencing nomenclature:

1. Near side / far side – the NEAR SIDE of the sign structure is as indicated in Publication 100A.
2. Truss Referencing – The four (4) chord truss system will be referenced as (LF) lower front; (UF) upper front; (LB) lower back; (UB) upper back. The Panel Points will be numbered from left to right, while looking at the Near Side of the Frame. The three (3) chord truss system should be referenced as noted above, except that the back single chord is referred to as the mid-chord. Example: The interior diagonal from upper chord (U6) to mid-chord (M7) should be labeled U6-M7.

### D. Inventory Information

All inventory items for the BMS3 Signs & Lights page are to be completed for initial inspections. This information is to be verified and corrected during all subsequent inspections.

### E. Clearance Sketch

Provide a sketch of the sign structure and roadway features showing all clearances. Roadway centerline, lane widths, shoulders, and guiderails are to be shown. Vertical clearances over roadway pavement and shoulder breaks are to be indicated. This data is to be field verified at each inspection.

### F. APRAS Form SC

RMS Route (SC02), Non Restricted Vertical Clearance (SC03) and Minimum Under- clearances for Routes On and Under the Span (SC09 and SC10) items are to be completed on SC Screen of BMS2.

## VII. FIELD INSPECTION PROCEDURES

### A. General Instructions

These procedures provide instructions for some aspects of cantilever and overhead sign structure inspection. Many of the techniques from bridge inspection are also applicable here. Use appropriate portions of these Guidelines and bridge inspection procedures to inspect Structure-Mounted signs.

1. Completely inspect all elements, cleaning members as needed to assess condition. Clearly record all inspection field notes in BMS3. Provide sufficient comments and descriptions to justify condition and appraisal ratings. (See Publication 100A, Inspection-Signs/Lights) Identify and record in BMS3 locations where visual as opposed to hands on inspections were performed, if any.
2. Record and/or photograph any manufactured markings on the structure.
3. List and prioritize needed Maintenance Items on the BMS3 Maintenance page. Additional bridge items from 3A screen may be added as appropriate (See Publication 100A).
4. Report any public safety threatening deficiencies that are observed and provide emergency retrofit schemes, as directed, to any critical conditions uncovered.
5. Using a magnet, verify whether the metal is steel or aluminum. Check several locations. The same metal is not always used throughout the structure (i.e. steel towers supporting an aluminum truss, or a steel column supporting an aluminum VMS cabinet). Note any variations in material.
6. Inspect for dents, cracks, rust, and overall condition of galvanizing.
7. Visually inspect all welds for cracks, especially where galvanizing is peeling, cracked or shows signs of rust. Primary points of interest are column to base, cantilever arm to pole and splice connection plates welds. Pay particular attention to welds in aluminum structures, and at locations of intersecting welds, such as at corners of base plate welds and gusset plates.

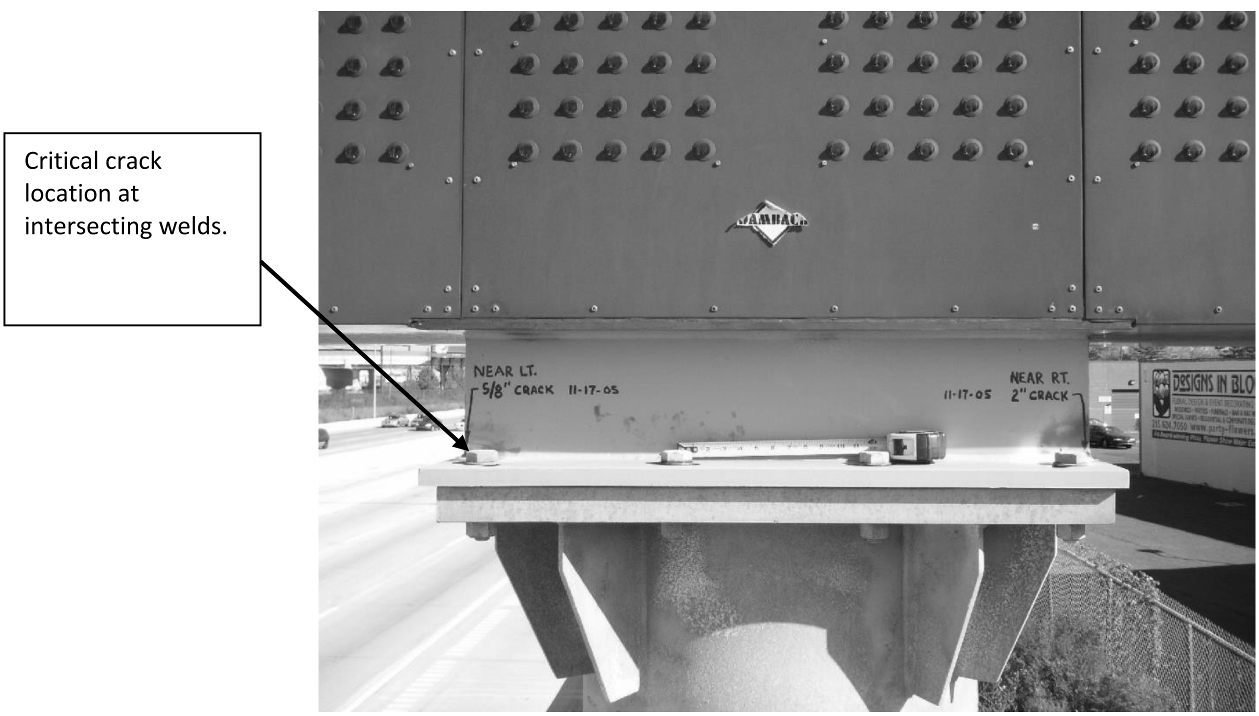



**Figure 1.** Critical Crack Location at Sign Base.

8. Dye-penetrant testing (ASTM E165-92) is to be used whenever cracking is suspected. Results of dye-penetrant tests at critical weld locations outlined above must be provided in all in-depth inspection reports. Other non- destructive testing methods, including ultrasonic (UT), may be used upon the approval of the District Bridge Engineer.
9. Repair portions of galvanizing damaged during testing using galvanizing paint.
10. For all sign structures approved by the District, replace all bulbs on each lighted sign during Routine Inspections.
11. Inspect all bolts and nuts for tightness, any indication of section loss and/or cracking, and rusting.
12. Record any bolts needing replacement and any missing bolts, nuts, or washers in the "Maintenance Items" section of BMS2. Select maintenance priority carefully as this can be a critical deficiency, depending upon location and redundancy. Show location of bolts tightened, replaced or requiring replacement on sketch.
13. Paint the nuts of any replaced or retightened bolts using the following color coding:
    - White - Replace Bolts
    - Yellow - Retighten Bolts
    - Red - Bolts that need to be replaced or those unfit for permanent re-tightening (see below)

<u>NOTE:</u> Because high strength bolts are plastically deformed during installation, subsequent re-tightening will produce further strain in the bolt that may result in fracture.

For A325 bolts: If the nut can be turned <u>by hand up to shank end of threads</u>, it can be safely re-tightened using normal installation procedures. If A325 bolts fail this test or cannot be checked, the nuts may be re-tightened <u>only</u> as a <u>temporary</u> measure. If the bolt and nut are part of the primary support system for a VMS sign, they must be identified for replacement regardless of whether or not they pass the thread test outlined above.

For A490 bolts: Do not re-tighten. Replace bolt and nut.

To install/tighten bolts, use turn-of-nut method. Snug tighten all bolts to the full effort of a person using an ordinary spud wrench; then tighten bolts as indicated in the following Table (from Pub. 408, Section 1050 Table B):

Nut Rotation from the Snug-Tight Condition (1), (2) Geometry of Outer Faces of Bolted Parts

| Bolt length measured from underside of head to end of bolt | Both faces normal to bolt axis | One face normal to bolt axis and other face sloped not more than 1:20 (20:1). Bevel washer not used. | Both faces sloped not more than 1:20 (20:1) from normal to bolt axis. Bevel washers not used. |
|---|---|---|---|
| Up to and including 4 diameters | 1/3 turn | 1/2 turn | 2/3 turn |
| Over 4 diameters but not exceeding 8 diameters | 1/2 turn | 2/3 turn | 5/6 turn |
| Over 8 diameters but not exceeding 12 diameters(3) | 2/3 turn | 5/6 turn | 1 turn |

(1) Nut rotation is relative to bolt, regardless of the element (nut or bolt) being turned. For bolts installed by 1/2 turn and less, the tolerance should be ±30 degrees; for bolts installed by 2/3 turn and more, the tolerance should be plus or minus 45 degrees.

(2) Applicable only to connections in which all material within grip of the bolt is steel.

(3) No research work has been performed by the Research Council on Structural Connections to establish the turn-of-nut procedure if bolt lengths exceed 12 diameters. Therefore, the required rotation must be determined by actual tests, in a suitable tension device, simulating the actual conditions.

14. In accordance with Pub. 15M (DM-4), Section 3.6.3 and Pub. 100A, IS15 (Sign Asset Tags), record the existence and condition of asset tags.

**B. Column Base, Foundation, etc.**

Include an assessment of the concrete foundation conditions, concrete column/pedestal, anchor bolts, grout (if used between foundation and base plate), tubing, and all welding of support members to base plates.

1. Inspect all footings visually or by feel (e.g. probing) for conditions, integrity, safe load capacity, etc.
2. Check concrete foundation for soil erosion, spalling and/or cracks; noting any vegetation growth through cracks. Also check and note condition of grout, if used.
3. Note any settlement or movement of foundation.
4. Note any soil or material build up around foundation, base plate, and/or column and add a maintenance item for cleaning/removal of material.
5. Conduct soundness test of the concrete foundation by listening to the sound made when lightly tapped with a hammer. Any areas around the outside of the foundation that exhibit a hollow ringing sound are to be noted for further evaluation and/or testing.
6. Inspect the base plate, gussets and tubing - outside (and inside where possible) for rust, ponded water, weld cracks, and condition of galvanizing.
7. Inspect the anchor bolts for size, rust, tightness of nuts, washers, section loss at threads, and condition of galvanizing. Use 24-ounce ball-peen hammer when checking bolts. Hit both sides of top nut and top of bolt to check for loose nuts and/or cracked or broken bolts. Tight nuts give sharp ringing sound, loose equates to dull sound. Determine and record bolt pattern, record missing bolts, damage, etc. Tighten any loose nuts with spanner wrench and 4-foot pipe extension in accordance with the table on page 6 and record where the loose or missing nuts are needed. If the full turn of the nut tightening is not obtained as indicated above, provide a recommendation for priority maintenance item. Loose or missing anchor bolts or nuts on cantilevers should be considered a critical deficiency. For other sign structures with 2 or more supports the anchor bolts are priority 2. Establish priority for maintenance items accordingly.
8. Use ultrasonic inspection if deemed necessary by Bridge Engineer to verify anchor bolt length and/or to determine if defects are present.

**C. Columns**

This includes single columns, or multi-column towers for overheads, as well as that portion of rigid frame overhead structures outside the first splice in horizontal portion.

1. Inspect the strut to column connection(s) on cantilever and center-mount structures, and/or the truss connections to the column(s). This includes the connection plate(s) and all welds.
2. Inspect for straightness and plumbness of members.
3. Inspect tension portions of rigid frame "columns".
4. Inspect the bottom of "columns" just above the base plates ultrasonically for internal section loss using an ultrasonic thickness gage (D-meter); perform the same type of inspection at other areas where internal section loss is suspected.
5. Inspect bent sections of monopipe sign structures for cracking and rust which may be a result of crimping during fabrication.

**D. Horizontal Members**

This includes struts, planar/3D trusses, portions of rigid frames and tubular overhead structures between exterior splices, and the framework for structure-mounted signs.

1. Inspect the splice plate and bolts/nuts for rusting, cracks, and condition of galvanizing.
2. Inspect all welded connections of the horizontal, vertical and diagonal members for cracking, weld condition and galvanizing.
3. Special attention must be given to the connections in the vicinity of the attachment to the column or at sign attachments.
4. Inspect for sagging or excessive deflection.

**E. <u>Sign</u>**

This includes the structural condition of the signboard and its accessories/attachments, not the legibility or visibility of the message.

1. Inspect all elements connecting the sign to the supporting structure, and all connections in the sign support framework.
2. Obtain access to the interior of "walk-in" cabinet type VMS signs, or open sign panels for front access VMS signs to inspect any structural framework of the sign that can be accessed by non-destructive means.
3. Report missing and/or broken nuts, or bolts in the maintenance items to the District Bridge Inspection Supervisor.
4. Perform dye-penetrant testing on aluminum or stainless steel sign attachment clip nuts. For Structure-Mounted signs, test 5% of the bolts at or near top of parapet, if any cracking is observed test an additional 10%. For Overheads and Cantilevers, perform dye-penetrant testing if cracked or missing nuts are observed following the above percentages.
5. Tighten loose or replace missing or damaged panel clip assemblies of either stainless steel or aluminum composition with a torque wrench. Apply 225 inch- pounds of torque to each nut with threads clean, dry, and unlubricated.
6. Inspect sign face extrusions for damage.
7. Visually inspect the aluminum sheeting for map cracking or other damage.
8. Record sign legend and approximate area of sign (use of collapsible survey rod would aid in obtaining many of the dimensions).

**F. <u>Powder Actuated Fasteners</u>**

If this fastener type is encountered for structure mounted signs during the course of a normal bridge inspection, the sign must receive a close visual and hands-on inspection for corrosion of the girder or fasteners. (Note: This will probably require a lane closure). Inspectors must peen the nuts of the fasteners and apply a light prying force to the sign to determine if the connections are still intact. Note in the narrative that this fastener type is present on the girders.

Any sign attachment of this type showing significant corrosion or loss of fasteners must be removed immediately (BMS2 Maintenance Priority Code "0"). Sign installations using these fasteners which are found to be still intact, should be given a BMS Maintenance Priority Code of 3, "Add to scheduled work." for replacement of the fasteners.

**G. <u>Lights and Electrical Components</u>**

This is the inspection of structural components of the lights and electrical components. It is not an electrical inspection.

For worker safety, lighted sign structures may have to be de-energized during inspection of sign structure. Districts are to provide needed keys to the inspectors.

The following items must also be completed:

1. Inspect light support for tightness where attached to the truss chords. Also check structural

condition of luminaires.

2. Locate and remove handhole covers in columns to check for drainage, corrosion, and vegetation growth inside the tubing. Remove any rodent nests or vegetation present. Make sure wire screens are installed around base plates. Visually inspect the condition of electrical components in the handhole.
3. Visually inspect electrical control box for damage.
4. For all sign structures approved by the District, replace all bulbs on each lighted sign during Routine Inspections.

### H. Access Conditions (BMS2 Item IS05)

Means of access for inspection and maintenance of sign structure must be evaluated. The adequacy of both the access main members and their attachments must be considered in determining this condition rating.

### I. Condition Ratings for Structural Components

Refer to the BMS Coding Manual's Condition Rating Codes for Sign Structure screen.

### J. Equipment List

Most of the equipment needed for sign inspection is contained in the BIRM's list of typical bridge inspection equipment. The following equipment is needed or especially useful for the sign structure inspections.

1. Dye-penetrant testing kit
2. 24-ounce ball-peen hammer
3. Cans of cold galvanizing or zinc rich paint
4. Binoculars (10 X 50)
5. Magnet
6. Dial-indicating Torque Wrench (0-300 in.-lb. range.)
7. Pipe wrench
8. Spanner wrench and 4-foot pipe extension 9. Wrenches (1/2" to 1- 3/4")
9. Socket wrench and deep sockets (1/2" to 1- 3/4")
10. Standard size nuts, washers, bolts, and clips to replace missing components (provided by the Department)
11. Light bulbs (provided by the Department)
12. Ultrasonic thickness gauge (D-meter)

The consultant must arrange for all field equipment, inspection cranes, platform lift trucks, ladders, etc. Access for inspection of portions of the sign structures that are affected by railroad electrification must be arranged by the Consultant with the railroad company when necessary.

## VIII. STRUCTURAL ANALYSIS

If significant section loss or damage from vehicular impact is found, a structural analysis of the sign structure must be performed. This analysis should be made using computer program(s) and/or hand calculations as needed. The District Bridge Engineer is to approve the need for and methodology of structural analysis. Use the following steps:

**A.** Acquire authorization from the District Bridge Engineer prior to performing a structural analysis.

**B.** Perform or update the structural analysis using a method acceptable to AASHTO and the Department.

**C.** Use conventional methods of analysis unless more complex and refined methods are specified or warranted and specifically authorized by the Department.

**D.** Ensure that all computations are in accordance with current Department and AASHTO Specifications. Update existing computations accordingly. When computer analysis is used, provide program input and output, calculations to prepare input, documentation of all assumptions, and any other post-processing calculations. Index computations so key data is readily available.

**E.** Use the Department's latest version of the SIGN software for analysis, if applicable.

If the structure to be analyzed is supporting a VMS sign, calculate wind forces in accordance with AASHTO Appendix C using a drag coefficient, $C_d$, equal to 1.7 per note 7 of AASHTO Table C-2. Also, in addition to the front or back face, the sides of the VMS are susceptible to ice loading, and the depth adds to the exposed area for calculating fatigue forces due to truck-induced gusting (AASHTO Chapter 11).

## IX. DRAWINGS

**A.** Update existing drawings or sketches whenever possible, rather than preparing new drawings. Use the Department's design drawing revision procedure to note changes since original drawing preparation.

**B.** If no plans are available, prepare sufficient drawings to document the makeup of the structure. Include data and views as follows:
1. General plan and elevation.
2. Cross sections.
3. Framing plan.
4. Sketches of structural members (including dimensions).
5. Results of field inspection analysis, and historical data, when appropriate.
6. Structural details, including all NSTMs unless adequately documented by photographs.
7. Locations of all fastener replacements and anchor bolt tightening.

**C.** Sketches done in an 8½" x 11" format are acceptable provided clarity of details and text can be maintained after being scanned into an Adobe "PDF" electronic format.

## X. PHOTOGRAPHS

**A.** Provide new photographs to supplement field inspection notes and drawings and to document conditions. Include views of overall sign structure plus its side elevation, any defects, structural details, and bolt replacement locations. Photographs shall be sufficiently clear, properly identified, dated, and indexed and of a resolution and quality acceptable to the District.

**B.** All photographs to be stored as 1600x800 high resolution JPEG images.

## XI. INSPECTION REPORT

**A.** Prepare a report to document the inspection, the sign structure, its condition, and recommendations. The report must be 8½" x 11" in size. PDF files are to be submitted via email.

**B.** General outline of the report is as follows:
1. Title page (Structure ID number, location, inspection dates, inspector names, prepared for and by, and Pennsylvania P.E. seal, signature and date.
2. Table of contents.
3. Location map(s). Map(s) must be of sufficient detail to locate sign structure.

4. General description and sketches and/or photographs of the overall sign structure.
5. Field inspection findings (completed BMS3 Inspection Pages, plus new photographs and supplemental narrative to document findings).

   Include the following in the report narrative:
   - General description of the structure condition.
   - Summary of inspection findings and comparison with those of previous inspection, if available.
   - Structural adequacy and safety of the structure.
   - Discuss relevant historical data, if any.
6. References, list plans, previous reports, etc. used in the preparation of the report.
7. Recommendations. As a minimum, address the following:
   - Note legibility and condition of signs.
   - A prioritized time scheduled listing of immediate and long-term improvement needs for:
     - Maintenance — Complete BMS3 Maintenance page
     - Rehabilitation — Complete BMS3 Maintenance page or D-491M
     - Replacement — Complete BMS3 Maintenance page or D-491M
8. Recommendations in report should be in "plain English" and be consistent with the costs indicated on the above pages.
9. Appendices:
   - Inventory Data: Marked-up copy of BMS2 file printout or completed copy of coding Forms D-491.
   - Inspection Data: Completed BMS3 Inspection Pages.
   - Structural analysis, if available.
10. Identify the locations of all bolt replacements and anchor bolt tightening in accordance with the numbering system outlined in the guidelines. Reference bolt replacements in sketch or by photograph and indicate size.
11. Identify the locations of all light bulb replacements performed during the sign structure inspection. Reference location in sketch or by photograph.

## XII. EMERGENCY REPORTING

The consultant shall notify the District Bridge Engineer immediately by phone whenever a potentially perilous or hazardous condition is observed. Provide written notification within 24 hours. Examples of such situations include:

1. Distress in primary members to the point where there is doubt that the members can safely carry the loads for which they are subjected, and partial or complete failure of the structure is likely.
2. Foundation movement or distress which is so excessive that there is a clear possibility that it may not be capable of supporting the structure and partial or complete failure is a possibility.
3. Any situation where the structural integrity of the structure or a portion of it is such that its safety is in question must be reported.

## XIII. MATERIAL SAMPLING AND TESTING

Structural materials evaluation, non-destructive testing (except dye penetrant tests) and structure instrumentation are not a routine part of sign structure inspection. They may be required by the Department to eliminate unacceptable engineering uncertainties or to more accurately access the structure's safety. The owner may elect to use them at any time.

Justify the use and obtain District Bridge Engineer authorization before initiating any materials sampling and testing and/or instrumentation program.

## XIV. EXISTING RECORDS AND DATA

The District will provide BMS2 access for sign structures to be inspected.

The District, if requested by the Consultant, will give the Consultant access to any available pertinent information for short term use and copying. This information could include existing structure drawings, structural analysis and design computations, inspection reports and other pertinent information. This data might be available only on microfilm.

## XV. QUALIFICATIONS OF INSPECTORS

Personnel assigned to the Inspection Project by the Consultant shall meet the requirements set forth in the National Bridge Inspection Standards for all work levels.

Inspection personnel must hold a valid certification as “Bridge Safety Inspector” issued by the Department. See Publication 238, IP 2.1.3, for additional information.

## XVI. TRAFFIC CONTROL

The Consultant shall provide traffic control during the inspection as needed to comply with the Department's Temporary Traffic Control Guidelines, Publication 213 and to insure the safety of inspectors and the traveling public at all times. Whenever possible, coordinate inspection effort with other work that is planned at the site to minimize disruption of traffic.

Any anticipated lane closures should be coordinated in advance since the times for inspection may be restricted by the District Bridge Inspection Supervisor or District Traffic Unit due to anticipated traffic conditions.

For traffic control details differing from Publication 213, submit a sketch to the District for approval. The District will identify these sites.

## XVII. RELEASE OF INFORMATION

The safety inspection of sign structures is not for public record under PA Right to Know Law [ 65 P.S. §67.101 et seq. and PA Vehicle Code, Title 75 [75 Pa. C.S. §3754(6)].

Place the stamp appropriate to structure owner per section IP 1.8.3 on the front cover of the inspection report. Do not release or distribute inspection information without the written permission of the District Bridge Engineer for State structures or the structure owner.

When portions of a report are approved for release, include the language provided in Publication 238, Figure IP 1.8.3-2, to each page of the structure inspection report that is released.

## XVIII. SUBMISSIONS

**A. Work Schedule and Status**
Submit a horizontal bar graph type work schedule within two weeks of notice to proceed for each work order. Also provide Open Plan data file via email unless otherwise directed. Submit monthly overall progress for each work order updates via email.

The Consultant shall be prepared to start work immediately upon execution of the work order. All work shall be completed expeditiously and not later than the date specified on the work order. Prior to beginning work, the consultant shall inform the District Bridge Engineer of the schedule of inspection of their sign structures(s) and names of the inspectors.

**B. Field Inspection Data**

Submit inspection data to BMS2 for Department's approval. In addition, submit a BMS2 printout marked with revisions and/or Form D-491 and/or BMS3 within ten (10) days of the completion of each field inspection.

**C. Draft Inspection Reports**

Submit the draft report within four (4) weeks of the completion of each field inspection for review. Make Inspection Report submissions to the Department at frequent intervals to facilitate timely reviews. Submit an additional sample draft report for the first five (5) submissions to the Department.

**D. Final Inspection Reports**

Submit an electronic file in pdf format of each final report within three weeks of receiving the review comments of draft report. Each report shall be signed by a Pennsylvania Licensed Professional Engineer knowledgeable of the report content. All submissions must be finalized before the contract expiration date.

## XIX. PRICE PROPOSAL

The method of payment will be based on a cost per unit of work for the categories shown in EXHIBIT B.

The District will identify sign structures to be inspected. The District and Consultant shall select the appropriate category applicable to each unit prior to preparation of the work order.

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**EXHIBIT B – PRICE FOR UNIT CATEGORY**

| Inspection Type | | A | B | C | D |
|---|---|---|---|---|---|
| Name of Work CATEGORY | Work Category | Initial Inventory | In-depth* | In-depth* (alt. Lanes Closed) | Routine* |
| Cantilever (1 lane) | C1 | | | | |
| Cantilever (2 Lanes) | C2 | | | | |
| Overhead (≤ 2 lanes) | O2 | | | | |
| Overhead (3 lanes) | O3 | | | | |
| Overhead (4 lanes) | O4 | | | | |
| Overhead (5 lanes) | O5 | | | | |
| Overhead (6 lanes) | O6 | | | | |
| Overhead (7 lanes) | O7 | | | | |
| Overhead (8 lanes) | O8 | | | | |
| Overhead (9 lanes) | O9 | | | | |
| Overhead (≥ 10 lanes) | O10 | | | | |
| Add'l Column/Tower | T1 | | | | |
| Bolt+ Replacement During Inspection(each) | BR1 | | | | |
| Light Bulb++ Replacement During Inspection (per lighted sign attached to sign structure) | LBR1 | | | | |
| Supplemental Inspection | S2 | | | | |

* Inventory data to be provided by District for verification.
\+ Anchor bolt nuts, aluminum sign clips and Hi-lock nuts.
++ Light bulbs.

The Work Category for individual overhead sign structures is to be adjusted for sign structures over wide medians. The adjustment is applied to the number of lanes as follows:

| Work Category Adjustment for Overhead Structures with Wide Medians | |
|---|---|
| Median Width, W (BMS2 Item VS30) U.S. Customary Units (ft.) | Additional No. of Lanes |
| W ≤ 24 | 0 |
| 24 < W ≤48 | 1 |
| 48 < W | 2 |

Examples:

| Sign Structure | Type | Actual Lanes | Median Width | Adjusted Lanes | Work Category |
|---|---|---|---|---|---|
| Overhead | D | 2 | 10 | 2 | 02 |
| Overhead | D | 4 | 30 | 5 | 05 |

For Work Categories 02 through 010, the Scope of Work includes inspection of 2 towers. For sign structures with more than 2 towers, the additional towers will be paid under Work Category T1.

For performing special investigations beyond the scope of work for inspection types A to D, supplemental units of work, consisting of 2 labor hours each, may be assigned by the District.

Bolt replacement (including anchor bolt nuts) will be paid on a cost per unit of work basis for each missing or broken bolt replaced, using bolts, nuts and washers provided by the Department. Tightening of bolts remaining in place is considered incidental to inspection.

Light bulb replacement will be paid on a cost per unit work basis for each lighted sign attached to the sign structure, using bulbs provided by the Department.

Traffic control, fasteners, rigging, testing (except dye penetrant testing), sampling and railroad involvement (if required) will be paid as a direct cost for nonprofessional services. The consultant will be paid the actual cost based on certified invoices. Should anticipated cost exceed three thousand dollars ($3,000) per work order, the consultant must solicit bids.

## XX. GENERAL INFORMATION

Project Manager for Department: The District in which the sign structures are located.

Agreement Administrator: Bureau of Maintenance and Operations, Asset Management Division.

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# APPENDIX IP 02-E

Standard Practices Manual for
Measuring & Documenting Scour
During Bridge Safety Inspections

**TABLE OF CONTENTS**



# INTRODUCTION

This is a Standard Practices Manual (SPM) for documenting the findings of scour-related features during the inspection of bridges over water. Information provided in the SPM is to:

- Address AASHTO's Manual for Bridge Evaluation (MBE) requirements found in Chapter 2:
    - Provide bridge waterway cross sections, channel cross sections or sketches (as needed), soundings and site map.
    - Provide adequate information on the stability of the waterway to assess the risk to the structure.
    - Determine and document an interval for obtaining and updating measurements.
    - Perform a historical comparison to determine the extent of changes over time.
- Determine the extent of stream stability or instability to accurately code streambed scour related items such as 4A08 Scour Critical Bridge Indicator (SCBI), IN03 Scour Rating, IN15 Streambed Material, 1A05 Channel, 1A05b Channel Protection, IU29 Scour Vulnerability, 1A13 Scour, and 1A14 Underwater Inspection.

This SPM is intended to supplement the FHWA NHI Bridge Inspector's Reference Manual (BIRM), specifically Chapter 13 – Inspection and Evaluation of Waterways, and to form uniform practices to inspect bridges for scour in Pennsylvania. Additional resources are the Procedures for Scour Assessment at Bridges in Pennsylvania and the SCBI/SAR Calculator Manual as indicated in Section 2.6; hyperlinks are also provided on the login screen of BMS2. Some practices may not apply to every bridge but are shown here to provide uniformity and completeness of documentation in bridge inspection reports.

As part of inspection planning, inspection managers should review the files and the latest reports of upcoming bridge inspections and note any missing file components (i.e., USGS Data, cross sections, soundings, site map) or items that do not meet current standards. Such deficiencies should be corrected in the next Routine, Underwater or scour related Special Inspection.

Section I focuses on the site map. The remaining Sections, II through IV, describe other important documentation of the waterway-bridge interaction that can assist in determining the cause of scour so that an effective scour countermeasure can be implemented when needed.

All documents should include the originating inspection date and the inspection dates of any subsequent updates. Additional information is presented in the Department's Bridge Scour Evaluation Training Course.

# I. SITE MAP

**DESCRIPTION –** Site maps are used to illustrate stream and site changes and identify the risk to a bridge from scour, lateral migration, channel erosion, and other causes. Site maps should also identify countermeasures that have been constructed to preserve the integrity of the structure. A sample site map is shown in Example 1. The site maps can be a hand sketch.

**WHEN TO USE –** Required for all NBIS length and State-owned >8' bridges and culverts over water.

- Complete during Initial Inspection or next Routine Inspection.
- Update at Routine Inspections if changes occur and indicate when no change is observed.
- Update during Special Inspection when required by Table IP 2.3.2.4-1 or Table IP 2.3.2.4-2 or as necessary for a Post-Flood Damage Inspection.

**PRACTICE (See Example 1):**

**1. Update site map to capture trends and history.**

- Use Observed Scour Assessment (OSA) (by USGS) information when possible as a baseline.
- Use consistent abbreviations, such as OSA abbreviations.
- Document date of changes to map.
- Reproduce the sketch when the map becomes cluttered. Retain the earlier version of the maps in BMS2.

**2. Show position of all substructure units and channel alignment.**

**3. Provide orientation details: North arrow, direction of segments ahead, direction of stream flow**

**4. Show location and quantity of:**

- Scour
- Substructure undermining
- Debris and vegetation
- Measures/Countermeasures
- Aggradation, sandbars, etc.
- Non-designed placed/dumped rock
- Channel erosion, sloughing, cut banks, etc.
- Meanders, lateral migration
- Low/normal water channel location
- Vegetation impeding water flow (e.g., brush, trees)

**5. Locate and quantify undermining of substructures.**

- Indicate location along face of substructure, depth of scour hole, and horizontal extent under footing (Provide a supplemental detail sketch for significant undermining, see Example 1).

**6. Measures/Countermeasures**

- Indicate type; rip-rap (including size), underpinning, grout bags/mattress, streambed paving, etc.
- Show installation date, if known.
- Measures are designed to resist scour; countermeasures are designed or placed to correct a known scour issue.

**7. Show location of channel cross sections (as required by Section IV of ths manual) on Site Map and identify points in the field by marking large trees and/or other features likely to remain stationary for easy future reference.**

**8. Probe Depths**

- Probe depths soundings along the substructure unit are required and locations shall be labeled on the stream sketch. Additional probe depth soundings in the channel can be provided based on site conditions to provide information on channel scour, establish an average stream depth, and to help determine if and underwater inspection is required.

# II. STREAMBED PROBES AT SUBSTRUCTURE UNITS

**DESCRIPTION –** Determining streambed and bottom of scour hole elevations are important relative to the bottom of foundations, typically the substructure footings. The "critical dimension" is the amount of scour necessary to expose the bottom of footing. This scour can lead to undermining as shown in Example 1.

Once undermining occurs, the horizontal extent of undermining relative to the front face of footing shall be documented (see the site map in Example 1) since this will also be a factor in determining the stability of the substructure foundation. For bridges on deep foundations (e.g., piles, caissons, etc.), the exposure height of the deep foundation elements must also be documented to determine stability. Inspectors must document findings in detail to enable a complete assessment of the foundation stability.

**WHEN TO USE –** All NBIS length and State-owned >8' bridges and culverts over water are to have measurements that determine the streambed and the bottom of scour hole elevations, if present, in the bridge file located in Documents screen of BMS2. Required intervals for streambed and bottom of scour hole elevations updates are shown in Tables A & B. Inspectors are still required to perform probing during other Routine or Special Inspections, they simply do not need to update the measurements unless significant changes are found.

**Table A: Streambed Probes at Substructure Units**
**(Bridge Structures and Bottomless Culverts)**

| Criteria | Interval in Months |
|---|---|
| Scour Vulnerability (IU29) = A or B | 72 |
| Scour Vulnerability (IU29) = 0, C or D AND SCBI (4A08) ≥ 3 | 24 or 48[1] |
| SCBI (4A08) ≤ 2 | Update at each inspection until associated scour repairs are complete (Maximum of 6 months). |
| [1] Underwater diving inspections are required at a 48-month inspection cycle instead of a 24-month cycle. Probing's should coincide with the underwater inspection. | |

**Table B: Streambed Probes at Substructure Units**
**(Culverts with Integral Bottoms and Associated Wingwalls)**

| Criteria | Interval in Months |
|---|---|
| Scour Vulnerability (IU29) = A or B | 72 |
| Scour Vulnerability (IU29) = 0, C or D AND SCBI (4A08) ≥ 3 | 24 or 48[1] |
| IN03 ≤ 3 | Update at each inspection until associated scour repairs are complete (Maximum of 6 months). |
| [1] Underwater diving inspections are required at a 48-month inspection cycle instead of a 24-month cycle. Probing should coincide with the underwater inspection. | |

**PRACTICE (See Example 1):**

1. **Determine extent of scour by measuring vertical distance from reference point of known, or assumed, elevation to bottom of streambed to establish elevation of stream bottom. The point of reference should be above the 100-year flood elevation or known high-water elevation listed in BMS2 Field IL05.**

- Establish physical reference point(s) on the structure that are easy to find and reach at all times for "repeatability". <u>The preferred location, top of barrier, will allow for measurement during flood conditions.</u>
  - Top of Barrier
  - Top of Abutment/Pier
  - Bottom of Beam
  - P-K nail in substructure element.
- Field measurements are made:
  - Distance from reference point to water surface (RP Dist)
  - Probe depth (PD) = water depth + soft infill, at various points
- Calculate streambed elevation via spreadsheet:
  - Streambed Elevation , SB Elev = Reference Point Elev – RP Dist –PD

2. **Determine Critical Dimension from bottom of scour hole to bottom of substructure footings in inspection documents.**
   - Determine bottom of footing elevation via bridge plans (if available), or relative elevation based on field measurements. Document source of elevation reference (e.g. design plans, as-built plans, or assumed) Be careful not to mix elevation references without reconciling them to each other.
   - Differentiate between the bottom of footing and underpinning repairs as well as between the footing and aprons placed in front of the footing.
   - Critical Dimension = SB Elev – BFE = Distance from probed streambed to the Bottom of footing elevation.
     - If the scour hole is below the Bottom of Footing Elevation, list the negative value as opposed to a value of 0.

3. **Document scour-related measurements to substantiate SCBI and Observed Scour Rating (OSA), as well as recommended maintenance activities and maintenance priorities.**
   - Scour and/or undermining
     - Note length of footing exposed. Calculate % of footing length exposed.
     - Note area of footing undermined. Calculate % of footing length and area undermined.
     - Note horizontal extent of scour from front face of footing, including notation to distinguish between the orginal footing and any apron constructed afterwards
     - If probing finds soft materials (e.g., in-fill and sediments), include measurements and description – see IN17 Observed Scour Depth and IN24 Inspection Notes.
   - Estimate/calculate % of individual span's waterway opening blocked
   - Estimate/calculate % of total hydraulic area blocked or % area blocked below the design flow elevation

4. **Compare new scour measurements with baseline and previous inspection results.**
   - Establish a set of the earliest available scour measurements as the baseline in tabular form for efficiency (See Example 1). The baseline measurements should be carried forward for all future inspections.
   - Utilize digital methods, as much as is practical, for accuracy and to easily update from one inspection to the next.
   - Review changes from previous inspections and from baseline and use successive inspections for monitoring waterway trends to help aleviate future problems.

5. **Use Probing to measure and document scour (or lack of scour) along face of substructure units.**
   - Probe along face of footing for each substructure unit in water.
     - Establish streambed and bottom of footing elevations using actual or assumed elevation reference points.
     - If water is too deep for probing or if the velocity of the water is too fast, an underwater diving inspection may be necessary.
   - Probe during Routine Inspections in accordance with Tables A & B on the previous page for all NBIS length and State-owned >8' bridges and culverts over water and during Special Inspection when reduced interval is necessitated by scour.

- Probe during each Post-Flood Damage Inspection.
- The intensity and documentation should increase as scour approaches the bottom of footing.
  - When stream bottom adjacent to the substructure unit is above the top of footing, the probing measurements may be limited to intervals along the quarter points of a substructure element.
  - Provide the average elevation of streambed, as well as isolated minor scour hole(s).
  - When stream bottom is below the top of footing of a substructure unit, use additonal intervals at smaller spacings along substructure face as needed to locate deeper holes, using a table to record stream bottom elevation.
  - Note depth of "soft" streambed materials encountered when probing. The soft materials may be merely fine sediment infill of a deeper scour hole.
- Additional guidance on probing documentation:
  - If undermining of footing is present, determine depth and horizontal dimensions. Provide sketch of undermining with dimensions.
  - If underpinning repair has been installed, document horizontal dimensions, depth of repair and the date the repair was completed. Clearly state if scour measurements refer to orignal footing or to underpinning
  - See Example 1 for an example of probing substructure units for scour. Practices include:
    - Streambed elevations are determined relative to a known reference point.
    - Elevation of bottom of footing was determined and shown.
    - Streambed elevations under the bridge are recorded in tabular form using data entry into a spreadsheet.
    - Horizontal extent of undermining recorded in table and shown on sketch.
    - When wing walls experience scour, include probing measurement points at wings.

**6. Store documents in BMS2 Documents for easy retrieval and reference.**

- Use editable documents (like excel) for scour measurement for storing information in BMS2 Documents to facilitate scour documentation for subsequent inspections.

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# III. BRIDGE WATERWAY CROSS SECTIONS

**DESCRIPTION –** Cross section along upstream side of bridge or culvert with elevations of stream bottom, bottom chord of superstructure, water surface, and bottom of footings determined from a known point of vertical reference. Include elevations for the upstream wing walls. If streambed movement is observed, provide additional information such as downstream elevations will be needed.

**WHEN TO USE –** All NBIS length and State-owned >8' bridges and bottomless culverts over water (See also Item 1, Basics of Measuring Scour), except those with Scour Critical Bridge Indicator (SCBI) = 9, are to have a baseline bridge waterway cross-section at the upstream face in the bridge file in BMS2's Documents link. Culverts with integral bottoms and their associated wingwalls are exempt from this practice.

Whenever waterway changes are apparent near the bridge then fully documenting stream changes with soundings, and cross sections is needed. Required interval for Bridge Waterway Cross Section updates are shown in Table C:

**Table C: Bridge Waterway Cross Sections**
**(Bridge Structures and Bottomless Culverts)**

| Criteria | Interval in Months |
|---|---|
| Scour Vulnerability (IU29) = A or B | 72 |
| Scour Vulnerability (IU29) = 0, C or D AND SCBI (4A08) ≥ 3 | 24 or 48[1] |
| SCBI (4A08) ≤ 2 | Update at each inspection until associated scour repairs are complete (Maximum of 6 months). |
| [1] Underwater diving inspections are required at a 48-month inspection cycle instead of a 24-month cycle. Probing's should coincide with the underwater inspection. | |

**PRACTICE (See Example 1):**

1. **Waterway information is needed for all NBIS length bridges and State-owned structures greater than 8' over water with 4A08 SCBI < 9, including bottomless culverts.**
   - The size and height of some bridges can make this impractical and/or diminish the usefulness of the cross section for inspection purposes; an example of an exception to this rule includes large river crossings.
   - Only portions of the structure that are affected by the 100-year floor or fall within the high-water mark need to be recorded as part of the bridge waterway cross section.
   - Record this data during the Initial Inspection.

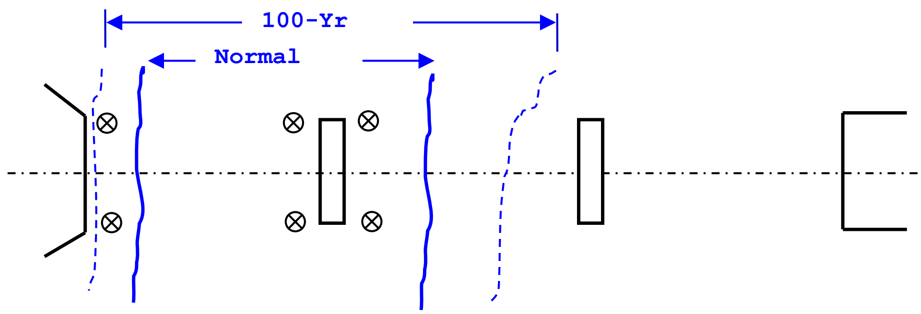
2. **Make measurements to bottom of stream and reference to known point of elevation.**
   - Using water depth measurements alone is not acceptable.
   - The level of accuracy needed for these measurements (+/- 0.1') does not necessitate survey teams.
   - For arches, culverts, and other structures where elevation of bottom of superstructures varies along the span, show those measurements on sketch.
   - The reference points should be easy to access and clearly marked. The reference point shall be to top of barrier if feasible or another reference point if the top of barrier is not feasible. Other reference points include beam seats, bottom of beam, and a pk nail in a substructure element.

3. **Tailor intervals of measurement to bridge size. Suggestions are as follows:**
   - Small spans (< 30'): 3 Points, 1 at each substructure and 1 between to locate the thalweg.
   - Mid-size spans (30' to 200'): 5 to 9 points.
   - Large spans (>200'): Tenth points of the span length, not to exceed 50' between points
   - Add locations if sandbars or aggradation encroaches on more than 10% of the waterway opening.

4. **The cross section at the fascia should include the following:**
   - Geometry of principal bridge openings up to the anticipated high-water elevation
   - Foundation units
   - Discernible scour holes
   - Discernible high-water marks at the bridge
   - Reference marks on the bridge
   - Sketch does NOT need to be a CADD drawing or drawn to scale
   - Provide bridge waterway cross-section for downstream fascia when one or more of the following conditions exist near the downstream opening of the bridge:
     - Significant sediment deposits
     - Advanced scour of more than 20% of footing length occurs
     - Significant stream migration has occurred

5. **Track scour progress over time:**
   - Use table to capture measurements from multiple inspections to allow for tracking.  Minimizes re-drafting cross section.

6. **Use "Corner Scour Measurement" to assist in tracking scour and for monitoring scour during a flood event.  These dimensions correspond to the measurements shown on the bridge cross section (See Example 1)**
   - Identify measurements for streambed elevations near each end of each substructure of every span to be checked during higher water. As indicated below, substructure units outside the 100-year flood limits or high-water mark do not need to be monitored.




LOCATION PLAN FOR CORNER MEASUREMENTS FROM TOP OF BARRIER

- Corner measurements should be taken beyond edge of footing (and underpinning if present) to avoid weights resting on top of the footing.
- Significant change indicates more detailed inspection and/or bridge closure is needed.
- Ensure monitoring measurements during high-water events are added to routine inspection measurements for complete records stored in one location.
- Corner measurements, by themselves, are not an acceptable substitute when full bridge waterway cross sections are called for but are used to supplement that information.

# IV. CHANNEL CROSS SECTIONS

**DESCRIPTION -** Cross section located on upstream or downstream side of bridge showing elevations of stream bottom, high/low water channel, water surface, and vegetation determined from a known point of vertical reference. Channel cross sections are to be approximately parallel to the roadway crossing the structure. The cross sections will be approximated by lateral stream bed movement measurements and soundings for most cases. Actual stream cross sections would require the use of a survey crew; therefore, only provide actual cross sections if deemed necessary. In addition, aerial photograph may be used to monitor channel changes.

**WHEN TO USE –** For both NBIS length and State-owned >8' bridges, as well as bottomless culverts over water, (except those with SCBI = 9) stream cross-sections upstream and/or downstream of the bridge may be very useful in determining the bridge waterway's ability to convey the stream flow without jeopardizing the bridge foundation.

Whenever significant waterway changes are apparent (1A05 ≤ 3), which threaten the bridge and/or approach roadway, then fully documenting stream changes with soundings, and channel cross sections are needed. If advanced channel condition change threatens the bridge, then channel cross sections are needed at the upstream and/or downstream sides of the bridge with an adequate number of sounding necessary to fully document stream changes. Inspectors are reminded that the coding of Item 1A05 – Channel Condition Rating is to be evaluated using both the general condition ratings as well as the channel condition rating code descriptions. Required interval for Channel Cross Section updates are shown in Table D:

**Table D: Channel Cross Section Updates**

| 1A05 Channel Condition or 1A05b Channel Protection Condition | Interval in Months |
|---|---|
| 3# | 12 |
| ≤ 2 | Update at each inspection until associated scour repairs are complete (Maximum of 6 months). |
| #Note: 1A05 or 1A05b Condition Rating = 3 will initiate a reduced inspection interval for the structure | |

**PRACTICE (See Example 1 for Approximated Channel Cross Section):**

1. **Provide approximated channel cross sections for all NBIS length bridges and State-owned structures with structure lengths greater than 8 feet over water with 1A05 ≤ 3 , including bottomless culverts.**

2. **Obtain additional channel cross sections of the channel at locations where:**
   - Channel alignment is Poor (IN10 ≤ 4) or
   - When channel erosion, sloughing, cut banks, etc. has occurred or
   - When channel lateral migration or meandering has occurred or
   - Channel is unstable and threatens the bridge waterway opening

3. **Update existing cross sections drawings for this purpose if acceptable existing cross sections are available from:**
   - Design H+H Reports.
   - Observed Scour Assessment (OSA) by USGS.

4. **Mark any new channel changes on the previous cross sections from the bridge inspection file to establish trends.**

5. **Channel cross sections for larger streams and rivers (streams wider than 200' at normal flow) may be more resource intensive than needed for routine inspections of bridges. Consideration may be given to:**
   - Limiting the cross section to the vicinity of substructures susceptible to scour.
   - Limiting the update of a channel cross section to the portions near substructures susceptible to scour.

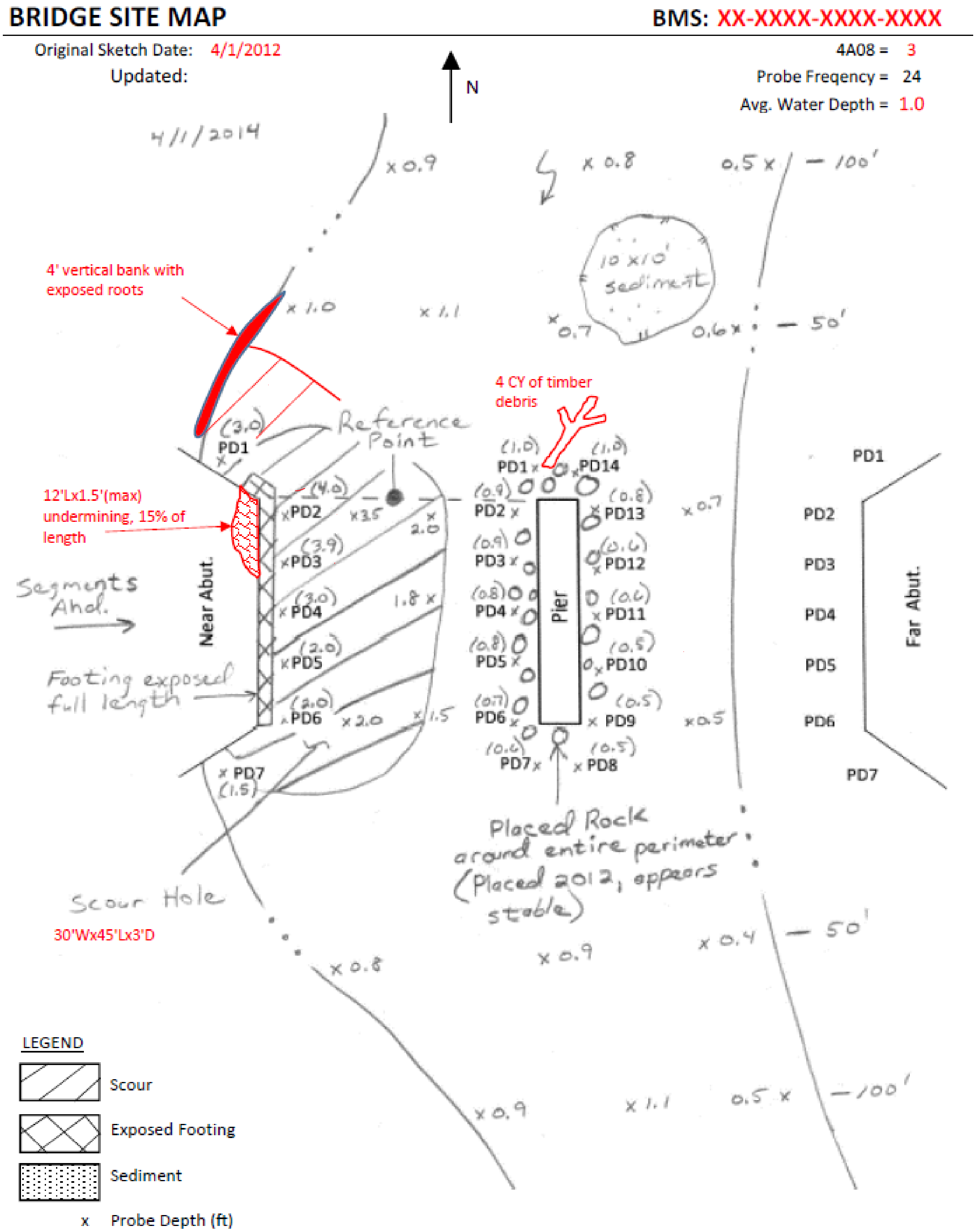
For scour critical bridges, these channel cross section limitations indicated above, may not provide sufficient channel bottom information for analysis and the extent of information required may be greater.

6. **Channel cross sections should:**
   - Be completed in table format, such as a spreadsheet, and stored in BMS2 Documents.
   - Be located on a site map.
   - Include a minimum of one cross-section upstream and one downstream.  Cross-sections from most cases can be approximated with lateral streambed measurements and soundings.
   - Be clearly marked between two landmarks, such as trees or other features likely to remain stationary. Landmarks shall be documented with information such as diameter of tree, approximate location from the bridge, and photographs shall be included to document the landmarks for future inspections.
   - Include lateral measurements to the edge of water at the substructure corners.
   - Approximately be parallel to the roadway.

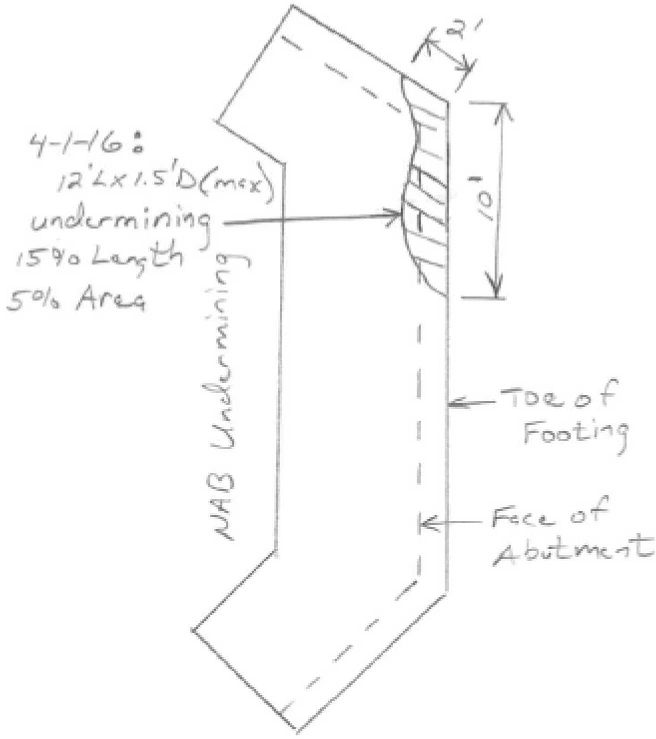
## *EXAMPLE 1 – SAMPLE SCOUR DOCUMENTATION:*

***Note: A sample spreadsheet template will be made available by PennDOT Central Office.***

### *I. SITE MAP:*



**Undermining Detail (Provide as needed to document undermining and track changes):**

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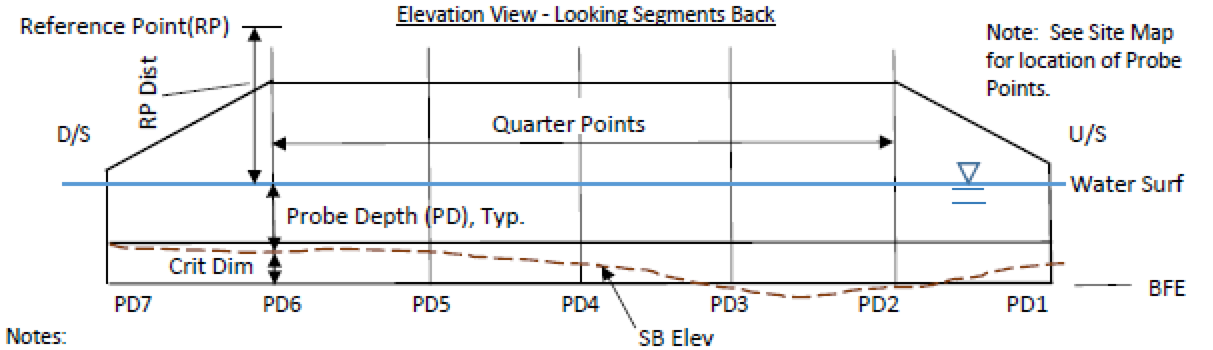
## *II. Streambed Probes at Substructure Units:*

**Streambed Probes at the Near Abutment** **BMS: XX-XXXX-XXXX-XXXX**

BFE = 82 (Calculated from Plans xxxxx)
PTE = NA

Reference Point (RP) = Top of Barrier at Span 1 Midspan
Reference Point Elev = 100 (From Design Drawings)



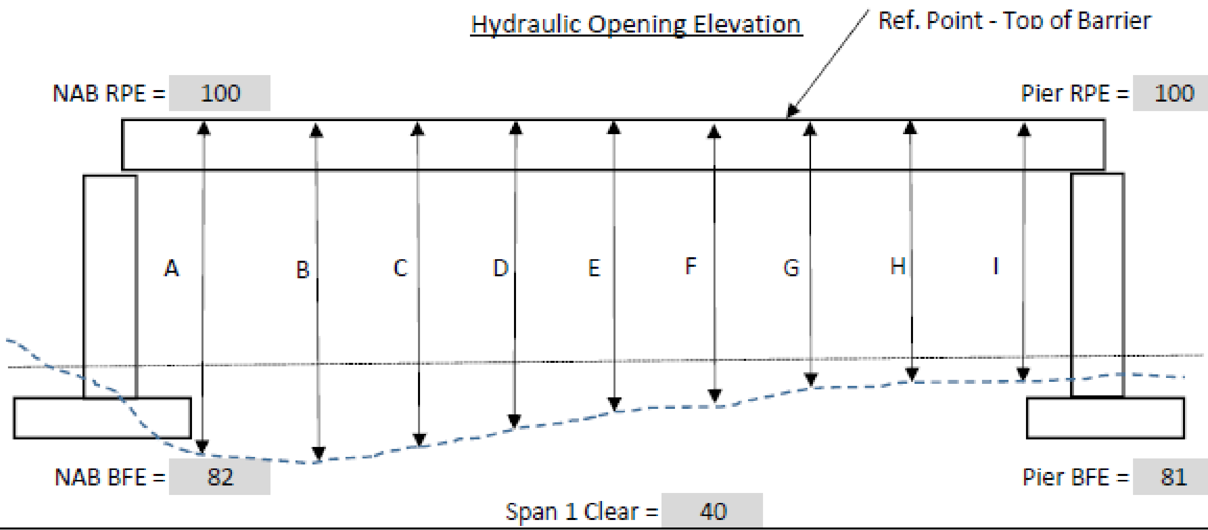

Notes:

Probe Depth (PD) = Total depth probed when in water = water depth + infill
Note: When probed depth is above water, record the distance as (-)
Use additional Probes between the Quarter Point probes as needed.
* SB Elev = Reference Point Elev. - RP Dist - Probe Depth = Streambed Elevation
** Crit Dim= SB Elev - BFE = Distance from probed streambed to the BFE
***Undermine = Horizontal undermining from the front face of the footing (Field Meaured)
RP Dist = Distance between the reference point and water surface.

| Date | PD7 | | PD6 | | PD5 | | PD4 | | PD3 | | PD2 | | PD1 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 4/1/12 | 0.5 | | 1 | | 1.5 | | 1.8 | | 2 | | 2.1 | | 0.5 | Probe Depth |
| | 86.0 | | 85.5 | | 85.0 | | 84.7 | | 84.5 | | 84.4 | | 86.0 | * SB Elev |
| Base | 4 | | 3.5 | | 3 | | 2.7 | | 2.5 | | 2.4 | | 4 | ** Crit. Dim |
| Year | 0.0 | | 0.0 | | 0.0 | | 0.0 | | 0.0 | | 0.0 | | 0.0 | ***Undermine |
| | Notes: Minor Scour, Footing Not Probed | | | | | | | | | | | | RP Dist = | 13.5 |
| 4/1/14 | 1 | | 1.2 | | 1.3 | | 1.5 | | 2.5 | | 2.5 | | 1.8 | Probe Depth |
| | 85.0 | | 84.8 | | 84.7 | | 84.5 | | 83.5 | | 83.5 | | 84.2 | * SB Elev |
| | 3 | | 2.8 | | 2.7 | | 2.5 | | 1.5 | | 1.5 | | 2.2 | ** Crit. Dim |
| | 0.0 | | 0.0 | | 0.0 | | 0.0 | | 0.0 | | 0.0 | | 0.0 | ***Undermine |
| | Notes: Advanced Scour, Footing Probed for 15' and exposed up to 0.5'. | | | | | | | | | | | | RP Dist = | 14.0 |
| 4/1/16 | 1.5 | | 2 | | 2 | | 3 | | 3.9 | 4.2 | 4 | 3.8 | 3 | Probe Depth |
| | 84.0 | | 83.5 | | 83.5 | | 82.5 | | 81.6 | 81.3 | 81.5 | 81.7 | 82.5 | * SB Elev |
| | 2 | | 1.5 | | 1.5 | | 0.5 | | -0.4 | -0.7 | -0.5 | -0.3 | 0.5 | ** Crit. Dim |
| | 0.0 | | 0.0 | | 0.0 | | 0.0 | | 0.5 | 1.5 | 1.0 | 0.5 | 0.0 | ***Undermine |
| | Notes: Serious Scour, Footing undermined up to 18" deep over 15' length. 15% of the length is undermined. | | | | | | | | | | | | RP Dist = | 14.5 |
| | | | | | | | | | | | | | | Probe Depth |
| | | | | | | | | | | | | | | * Elev |
| | | | | | | | | | | | | | | ** Crit. Dim |
| | | | | | | | | | | | | | | ***Undermine |
| | Notes: | | | | | | | | | | | | RP Dist = | 14.5 |
| | | | | | | | | | | | | | | Probe Depth |
| | | | | | | | | | | | | | | * Elev |
| | | | | | | | | | | | | | | ** Crit. Dim |
| | | | | | | | | | | | | | | ***Undermine |
| | Notes: | | | | | | | | | | | | RP Dist = | |

### *III. Bridge Waterway Cross Sections*

**Bridge Waterway Cross Section at Bridge Opening** **BMS: XX-XXXX-XXXX-XXXX**




**Notes:**
BFM = Bottom of Footing Measurement = RPE - BFE
Measurements are from top of barrier to streambed
Elevations are from Design Drawings XXXXX.

**# of Points**
Small Spans (<30') - 3 Points
Med. Spans (30' to 200') - 5 to 9 Points
Large Spans (> 200') - Tenth Points of Span

| U/S READINGS - Span 1 | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Point | A | B | C | D | E | F | G | H | I |
| % Span | 0 | 0.125 | 0.25 | 0.375 | 0.5 | 0.625 | 0.75 | 0.875 | 1.00 |
| Distance (ft) | 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 |
| BFM | 18 | | | | | | | | 19 |
| 4/1/12 | 17.5 | | 18 | | 16 | | 16 | | 15.5 |
| 4/1/14 | 18 | | 18.5 | | 16.5 | | 16 | | 15.5 |
| 5/24/15 Flood | 18.5 | | | | | | | | 15.5 |
| 4/1/16 | 18.5 | | 19 | | 17.5 | | 16 | | 15.5 |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

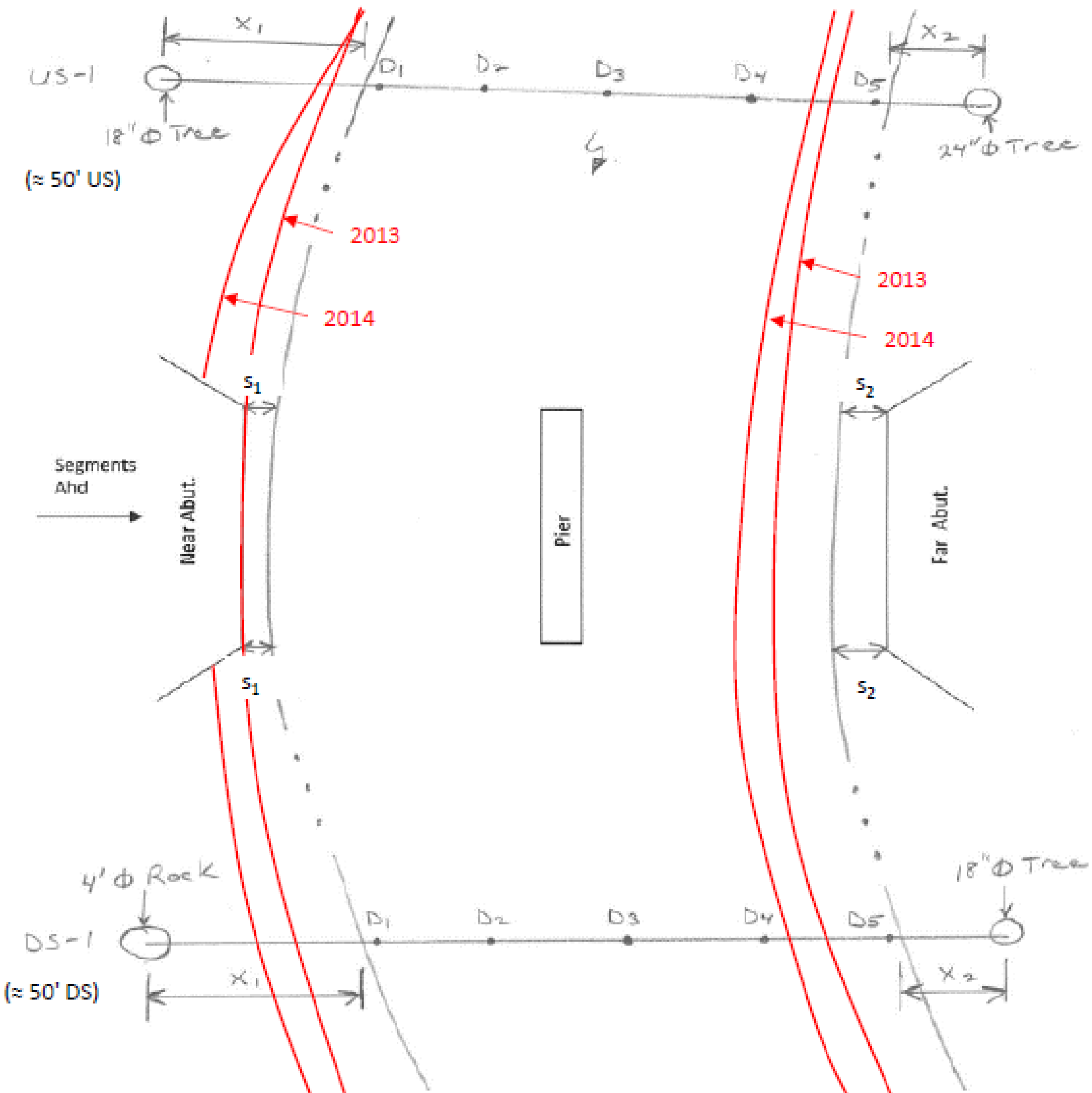
| D/S READINGS - Span 1 | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Point | A | B | C | D | E | F | G | H | I |
| % Span | 0 | 0.125 | 0.25 | 0.375 | 0.5 | 0.625 | 0.75 | 0.875 | 1.00 |
| Distance (ft) | 0 | 5 | 10 | 15 | 20 | 25 | 30 | 35 | 40 |
| BFM | 18 | | | | | | | | 19 |
| 4/1/12 | 14.5 | | 15 | | 16 | | 16 | | 15 |
| 4/1/14 | 15 | | 16 | | 16.5 | | 16 | | 15 |
| 5/24/15 Flood | 16.5 | | | | | | | | 15 |
| 4/1/16 | 16.5 | | 17 | | 17 | | 16 | | 15 |
| | | | | | | | | | |
| | | | | | | | | | |
| | | | | | | | | | |

## *IV. Channel Cross Sections*

### Channel Cross Section

**BMS: XX-XXXX-XXXX-XXXX**

Original Sketch Date: 4/1/12 Channel Cond., IA05 = 3 Monitor Freqency = 12 months




*U/S and D/S Sections*

| Location | Date | Water $x_1$ | $x_2$ | Water Depth $D_1$ | $D_2$ | $D_3$ | $D_4$ | $D_5$ |
|---|---|---|---|---|---|---|---|---|
| US-1 | 4/1/12 | 15 | 10 | 2 | 3 | 3 | 1 | 1 |
| | 4/1/13 | 13 | 12 | 2 | 3.5 | 3.5 | 1 | 0 |
| | 4/1/14 | 11 | 13 | 2.5 | 4 | 4 | 1.5 | 0 |
| | | | | | | | | |
| | | | | | | | | |
| DS-1 | 4/1/12 | 18 | 8 | 1 | 2 | 2 | 2 | 1 |
| | 4/1/13 | 16 | 10 | 1 | 2.5 | 3 | 2 | 0 |
| | 4/1/14 | 14 | 12 | 1.5 | 3 | 3 | 2.5 | 0 |
| | | | | | | | | |
| | | | | | | | | |

*Substructure Corners*

| Location | Date | Dist. To Water $s_1$ | $s_2$ |
|---|---|---|---|
| US Bridge | 4/1/12 | 2 | 3 |
| | 4/1/13 | 0 | 5 |
| | 4/1/14 | -4 | 9 |
| | | | |
| | | | |
| DS Bridge | 4/1/12 | 2 | 3 |
| | 4/1/13 | 1 | 4 |
| | 4/1/14 | -2 | 7 |
| | | | |
| | | | |

# APPENDIX IP 02-F

Action Plan for Emergency
Bridge Closure

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**Action Plan for Emergency Bridge Closure**

| STATE OWNED STRUCTURES | | | |
|---|---|---|---|
| | ***ACTION*** | ***RESPONSIBLE PERSON*** | ***EXPECTED RESULTS*** |
| 1 | Establish need for emergency structure closure. | Inspection Team Leader | Record Basic Information for Step 2<br>- Photograph of deficiency<br>- Location and probable cause of deficiency<br>- Structure type and affected structural member<br>- Do you have required resources to safely close the bridge? |
| 2 | Make notification to PennDOT's District Bridge Unit | Department Inspection Team:<br>Inspection Team Leader<br>Non-Department Inspection Team:<br>Professional Engineer and Inspection Team Leader | Contact the Assistant District Bridge Engineer - Inspection and notify of need for closure<br>- Provide Basic Information from Step 1<br>- Consult on the need for closure<br>- Discuss resources available to close the bridge<br>- Adequate sight distance (Refer to Item 4A02)<br>- Vehicle properly equipped (amber lights) |
| 3 | Make notification to County Maintenance | District Bridge Engineer or Assistant District Bridge Engineer - Inspection | Inform County Maintenance of the incident<br>- Provide location of the structure<br>- Identify a contact person on-site and in the office |
| 4 | Identify the detour route | County Maintenance Manager, Assistant County Maintenance Manager or District Traffic Engineer | Develop detour using State routes |
| 5 | Complete the bridge/road closure and implement detour | Assistant County Maintenance Manager or Highway Foreman | Stop vehicular and pedestrian traffic from using the structure above and below<br>- Place signage and trucks to block traffic at either end of the structure<br>- Implement the detour in the field |
| 6 | Send notifications to the outside public | County /Assistant County Maintenance Manager and Community Relations Coordinator | Make notification to the County 911 center<br>Enter an RCRS Event |
| 7 | Develop a Plan of Action and Bridge Problem Report | District Bridge Engineer or Assistant District Bridge Engineer<br>- Inspection | Complete a Bridge Problem Report and Plan of Action<br>Send POA and BPR to the Assistant Chief Bridge Engineer - Inspection |

| NON-PENNDOT STRUCTURES | | | |
|---|---|---|---|
| | ***ACTION*** | ***RESPONSIBLE PERSON*** | ***EXPECTED RESULTS*** |
| 1 | Establish need for emergency structure closure. | Inspection Team Leader | Record Basic Information for Step 2<br>- Photograph of deficiency<br>- Location and cause of deficiency<br>- Structure type and affected structural member<br>- Do you have required resources to safely close the bridge? |
| 2 | Make notification to the bridge owner and PennDOT's District Bridge Unit | Professional Engineer and Inspection Team Leader | Contact the bridge owner and Assistant District Bridge Engineer - Inspection and notify of need for closure<br>- Provide Basic Information from Step 1<br>- Consult on the need for closure<br>- Discuss the ability to use vehicle to stop traffic<br>- Adequate sight distance (Refer to Item 4A02)<br>- Vehicle equipped with amber lights |
| 3 | Make notification to Municipal/County Maintenance or Traffic Control Provider* | Municipal/County owner or Professional Engineer | Secure Traffic Control for Closure<br>- Provide location of the structure<br>- Identify a contact person on-site and in the office |
| 4 | Identify the detour route | Municipal/County owner | Develop detour using local routes |
| 5 | Complete the bridge/road closure and implement detour | Municipal/County Maintenance Force or Contracted Traffic Control | Stop vehicular and pedestrian traffic from using the structure above and below<br>- Place signage and trucks to block traffic at either end of the structure<br>- Implement the detour in the field |
| 6 | Send notifications to the outside public | Local owner or Professional Engineer | Make notification to the County 911 center<br>Develop a press release |
| 7 | Develop a Plan of Action | Professional Engineer and Inspection Team Leader | Complete a Plan of Action and meet with the Bridge Owner<br>Send the Plan of Action to the Assistant District Bridge Engineer - Inspection |
| 8 | Develop a Bridge Problem Report | Assistant District Bridge Engineer - Inspection | Complete a Bridge Problem Report and send to the Assistant Chief Bridge Engineer - Inspection. Make notification to District Bridge Engineer |

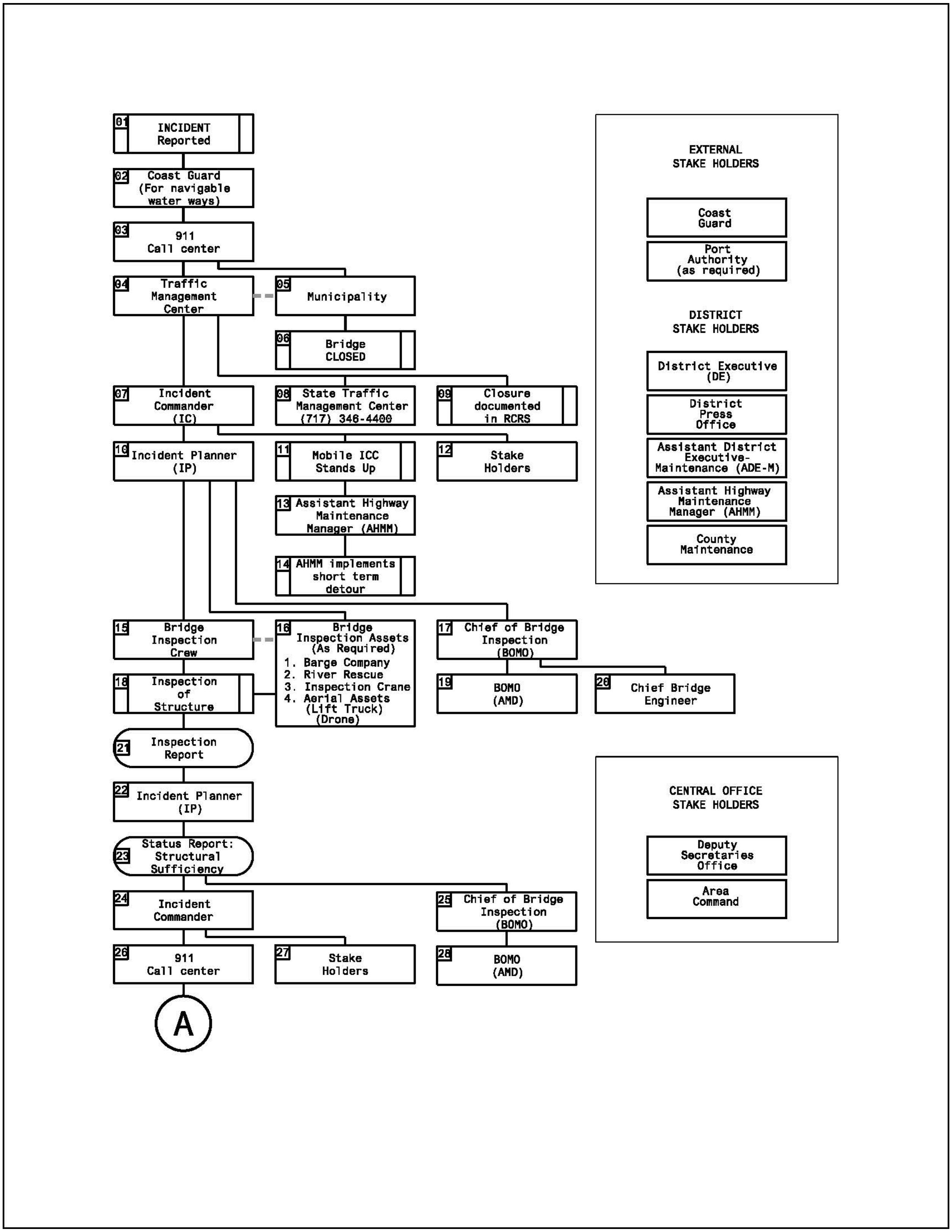
* If local owner is not available or fails to respond, solicit assistance from the District Bridge Engineer as indicated in Publication 238, IP 02.14.

# APPENDIX IP 02-G

Inspection Procedures Following
Emergency Events

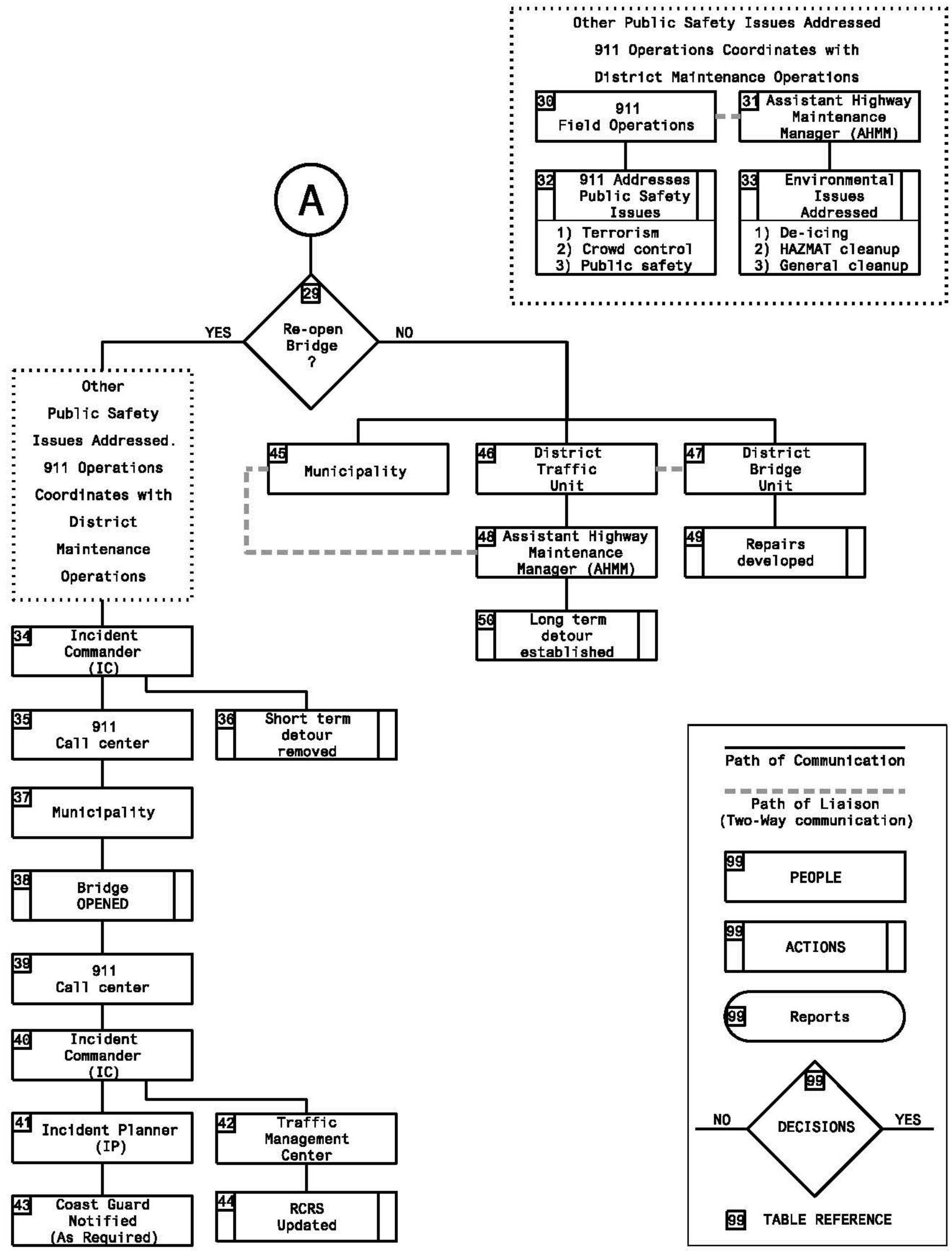
## Sample Communication Flowchart for Emergency Bridge Event (1 of 2)

Example shown is for a barge hit, but the chart can be modified for other events.



## Sample Communication Flowchart for Emergency Bridge Event (2 of 2)

Example shown is for a barge hit, but the chart can be modified for other events.



## Post-Seismic Event Bridge Inspection Guidelines (1 of 2)

Seismic events are complex in nature and it is difficult to develop stringent protocols. With that said, PennDOT has used recent seismic events within our State as well as lessons learned in nearby States to develop guidelines for inspecting bridges after a seismic event has occurred.

### Notification

The Bridge Inspection Section (BIS) receives notifications directly from USGS via their website (https://earthquake.usgs.gov/earthquakes/). Other bridge inspection staff are encouraged to also enroll in notifications. Notifications may also originate from an Emergency Management Center. Upon notification, the BIS will ensure that the following individuals are alerted:

- Deputy Secretary Highway Administration
- Chief Executive
- Director, Bureau of Maintenance and Operations (BOMO)
- Chief Bridge Engineer
- District Executives affected
- District Bridge Engineers affected

### Seismic Response Levels

Pennsylvania has dealt with seismic events in its history; however, a reliable record of bridge damage resulting from seismic events does not exist. Therefore, it is difficult to analyze and set a threshold for when there is a greater likelihood of damage. Utilizing recent seismic events in and near PA as well as other States' guidelines, the four response levels outlined in the table below can be followed.

**Table IP 02-G–1 Seismic Response Levels**

| Response Level | Earthquake Magnitude | Radius of Concern | Description of Response |
|---|---|---|---|
| 1 | < 3.5 | - | No follow-up inspection warranted. Limited inspection scope may be deemed necessary by BIS on a case-by-case basis. |
| 2 | 3.5 to 4.5 | 20 mi | Preliminary inspection of all infrastructure located in the radius of concern. Can be performed by bridge inspection or maintenance personnel. Inspections are cursory in nature. |
| 3 | 4.6 to 5.5 | 50 mi | Thorough inspection by qualified NBIS inspector of structures located in the radius of concern. Similar in scope to a damage inspection. Access means shall be coordinated if required. |
| 4 | > 5.5 | 100 mi* | Thorough inspection by qualified NBIS inspector of structures located in the radius of concern. Similar in scope to a damage inspection. Access means shall be coordinated if required. Closures are expected due to higher likelihood of infrastructure damage. |

*For higher magnitude earthquakes the radius of concern may be adjusted on a case by case basis as determined by the Assistant Chief Bridge Engineer - Inspection.

## Post Seismic Event Bridge Inspection Guidelines (2 of 2)

Bridge Inspection Criteria

Once the response level has been determined, the Bridge Inspection Section will distribute the list of affected structures to the District Bridge Engineers. These lists are assembled using the following criteria:

- Bridges with rocker bearings
- Bridges with any individual span >200’
- Bridges with pier heights > 60’
- NSTM bridges
- Curved bridges
- Other bridges based on intensity, location, and depth
- Tunnels, sign structures, retaining walls and high mast lights may be included for response levels 3 and 4.

Districts may be asked to provide information on bridges in staged construction or shored state.

Inspections will be considered Damage Inspections and shall only be performed during daylight hours to allow for clear visibility and hands-on inspections if necessary.

During the inspections, focus should be on the follow areas of concern:

- Deck joints for lateral movements or unusual longitudinal movements
- Bearings for excessive tilt
- Unusual cracking in deck, superstructure or substructure
- Settlement of approach roadway pavement that could indicate global instability
- Damage to pin and hanger assemblies
- Damage to attached utilities or supports (e.g. gas or water lines)

In the instance of a Response Level 4 affecting a large population of bridges, prioritization of inspection will be as follows:

1. Interstates
2. NHS Routes
3. Non-NHS

If seismic related damage is found, the District Bridge Engineer should be notified immediately. The Chief Bridge Engineer and Assistant Chief Bridge Engineer - Inspection should then be notified as soon as possible. If closure is warranted, it shall be done immediately.

## Post Fire Evaluation Guide

A fire event associated with a bridge, the Incident Command Organizational Structure is implemented. In support of the Incident Commander, the District Bridge Engineer shall:

1. Assess the safety of the bridge regarding safety of vehicles both civilian vehicles and fire equipment on the bridge

2. Assess the safety of the bridge to determine if the bridge can be reopened to traffic

Some types of damage are obvious, such as large deflections, bearing damage, collision damage, buckling of members, and other visually detectable degradation. There is also a possibility of hidden damage due to degradation of material properties.

This Guide provides information on fire effect to steel and concrete properties, inspection guidance, material testing guidance and results of investigations into 2 fire damaged steel bridges.

### 1. Overview of Structural Behavior from the Effects of Fire

Understanding the high temperature structural response of bridges is useful for predicting behavior prior to fire events. The maximum deflection at high temperatures is determined by thermal expansion effects, reduced material strength, the reduced modulus of materials at high temperatures, and the effects of creep. When the structure cools after the fire event, a substantial amount of this deflection recovers. It is even possible in some cases to have some positive residual camber if localized yielding occurs during the fire. If the deflection recovers, the geometry of the bridge is still suitable for its intended traffic use. Any effect on load rating needs to be determined based on a survey of localized damage and post-fire material properties. Predicting the maximum high temperature deflection of structures is interesting, but the most important aspect is to determine/measure the presence of permanent deformation after the fire.

An engineering assessment is required to evaluate the post-fire strength and serviceability of the bridge structure. Any permanent deflections will be obvious, and their impact can be assessed without the need for high temperature modeling. However, high temperature modeling can be useful to develop a better understanding of material temperatures during the fire. There is substantial information available in the literature that can be used to predict the post-fire material properties based on the temperature reached during the fire event. Fire simulation modeling provides a benchmark for predicting the material temperatures that occurred during the fire. The fire simulation modelling is a specialized analysis and is warranted in certain circumstances as determined by District Bridge Engineer.

Thermal expansion may be the most important thermal property for structural response prediction at high temperatures. Large thermal strains and strain incompatibilities between different parts of the structure are a primary cause of global deflection and localized damage. The coefficient of expansion of steel increases at elevated temperatures. The thermal conductivity of steel is about 30 times greater than concrete. Therefore, concrete tends to heat up internally much slower than steel when exposed to the same surface temperatures. This effect is very beneficial to prevent or delay strength loss in concrete beams and bridge decks.

Fire suppression efforts can worsen bridge damage during fire events. Water applied directly to hot steel or concrete members can sometimes cause more damage than letting the members cool slowly. For steel members, quenching localized areas with water can induce large temperature gradients that may create local buckling and distortion. There is also the possibility that water may act as a quenching agent on the material. Quenching can be expected to alter the properties of some steels that may impair their fitness for continued service in the structure. Water spray on hot concrete members can accelerate cracking and spalling during the fire event. The strength of concrete members depends in part on the integrity of the reinforcing steel. Having concrete cover protects the steel from direct exposure to heat. Rapid quenching with water can cause cracking and spalling of the concrete cover thereby exposing the reinforcement to higher temperatures. In general, fire fighters should be directed to avoid spraying water directly on the bridge superstructure. Obviously, there may be more important issues than preventing bridge damage during the fire event. For fires occurring underneath a bridge the suppression should be directed to the base of the fire. It may not be possible to avoid spraying the bridge when fires are located on the bridge deck. In this case, the best approach is to avoid, as much as possible, spraying the beams underneath the bridge.

Infrared photography may also be useful to assess bridge temperature during the fire. The presence of smoke and flames during the peak fire times may block the effectiveness of this method. However, infrared information at the beginning of the cooling phase would be very useful.

### 1.1 *Fire Location*

The location of the fire has a significant effect on the deflection response of the bridge. Since the strength and stiffness of the members is proportional to their temperature and the fire location determines how the different members are heated, the fire location directly determines the distribution of weakened members in the structure.

Studies have shown that it is inaccurate to assume that the surface temperature of bridge members equals the gas temperature of the fire. On an average, the severe fire simulations performed in the NCHRP 12-85 study show gas temperatures around 1400°C (2552°F) for a typical gasoline tanker fire while the maximum steel temperature only reached about 700°C (1292°F) and held relatively steady at that level.

For bridges without structural members above the deck level, fires contained on bridge decks do not cause any significant heating of the bridge members below the deck. Most of the heat is directed upward and the deck concrete has relatively low thermal conductivity. This protects the underlying members from heat. It is important to recognize that fires involving fuel spills may also extend underneath the bridge. Liquid fuel will follow the normal drainage path for rain water and can cause significant fire underneath the bridge.

Bridges with support structure above deck level, such as through trusses and cable supported structures, may have structural damage from fires confined to the roadway deck. Any structural member that is exposed to direct flames for more than a few minutes may be vulnerable to fire damage. Guidance developed from girder temperature in NCHRP 12-85 Research Report for the 14 case studies may be useful to estimate temperatures versus time for truss members exposed to flames. Cables are not directly covered by this Guide, but some information is available in the NCHRP 12-85 Research Report.

Fires located in the center underneath of the bridge, both longitudinally and transversely cause the largest vertical deflection. For this case, the maximum heating occurs at the point where the girders have maximum moment due to loading. The exterior girders receive less heating since the fire typically does not wrap around the exterior face of the girders. There is a general reluctance for flames to spread transversely across the bridge due to the projection of the girders. It is much easier for flames to spread longitudinally. The center location causes strength reduction in most of the interior girders and the two exterior girders will carry most of the load in the latter fire stages. The bridge width and number of girders will have a substantial effect on transverse fire spread. Wider bridges will have even less heating on the exterior girders and more redundant load paths to support the weakened girders.

## 2. Post Fire Inspection

1. Development of an overall flame spread map is the first step to developing a plan to proceed with evaluation of the bridge. See Figure 1.
2. In general, the overall flame spread mapping should define regions of the structure into at least three categories relative to fire exposure:
   a. regions with no direct contact with flames;
   b. regions with intermittent or lapping flame contact and
   c. regions with continuous flame contact.
3. A significant fact that will aid post-fire strength evaluation is knowing the maximum temperature of the bridge members but is difficult to quantify. The primary variables are full source and vertical offset. See Table 1 for data from NCHRP 12-85.

   The purpose of this mapping is to focus the post-fire inspection on the regions with the highest potential for material damage. Regardless of fire exposure, physical damage may exist in other parts of the structure due to thermal expansion and distortion of members. Therefore, a detailed structural inspection of the entire structure is suggested, regardless of the flame spread map. See Section 5 for instructions of components to be inspected post fire. The presence of physical cracking and distortional damage can be readily assessed by detailed visual inspection and the consequences of such damage shall be evaluated by the District Bridge Engineer.

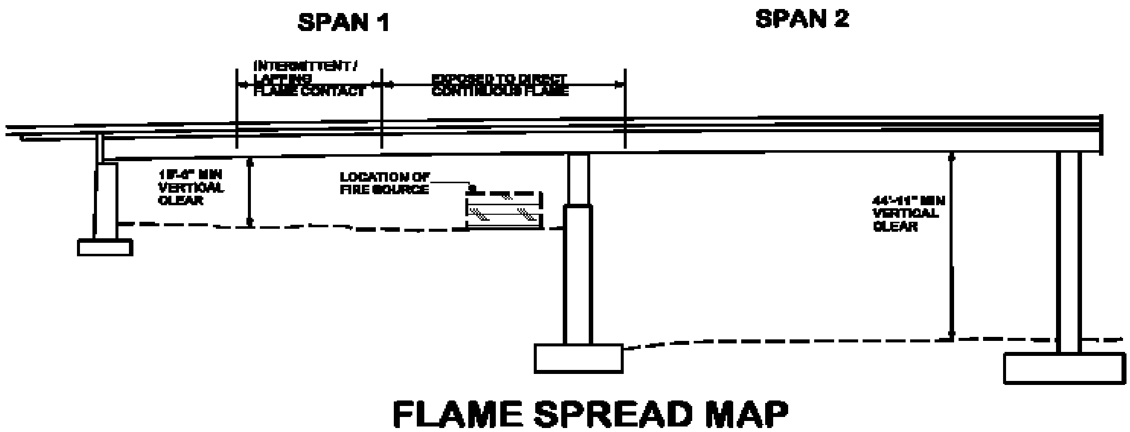



**Figure 1 – Flame Spread Map**

**Table 1 – NCHRP 12-85 Benchmark Fire Simulations**

| Case No. | Vehicle Type | Fire Location | Beam Material | Vertical Clearance (ft) | Maximum Heat Release Rate (MW) | Event Duration Heating Phase (seconds) | Event Duration Cooling Phase (seconds) |
|---|---|---|---|---|---|---|---|
| 1 | | A | Steel | 16 | | | |
| 2 | Bus | B | Steel | 16 | 40 | 5000 | 3600 |
| 3 | | C | Steel | 16 | | | |
| 4 | HGV | A | Steel | 16 | 210 | 2500 | 6100 |
| 5 | | A | Steel | 16 | | | |
| 6 | Half | B | Steel | 16 | 105 | 2500 | 6100 |
| 7 | HGV | C | Steel | 16 | | | |
| 8 | | A | Steel | 16 | | | |
| 9 | Tanker | B | Steel | 16 | | 3000 | 5600 |
| 10 | | C | Steel | 16 | | | |
| 11 | Tanker | I-65 | Steel | 16 | 295 | | |
| 12 | | A | Steel | 24 | | | |
| 13 | Tanker | A | Concrete | 16 | | 2650 | 5950 |
| 14 | | A | Steel | 32 | | | |

## 3. Steel

### 3.1 *Carbon and Low Alloy structural steels:*

Assuming a structure does not collapse in a fire event, the residual properties of steel must be understood to evaluate the post-fire safety of the structure. It is possible for steel to sustain damage in the fire event that will alter its residual properties. However, damage is only expected for cases with extreme fire exposure. The extent of damage depends on the intensity and duration of the fire, the geometry of the structure, the type of material, and the type of load on the members. The best way to assess structural integrity after a fire is to measure the material properties. However, this requires destructive testing that may not be warranted or practical in most cases. This is because common structural steel grades generally do not show much, if any, post-fire strength reduction unless the reach very high temperatures approaching the melting point.

The most common steel grades present in bridges are A709 Grades 36, 50, and 50S. These steels are typically heated to around 1000°C during the rolling process and allowed to air cool at the mill. The steels have relatively low hardenability indicating that the mechanical properties are relatively insensitive to heat treatment. As appoint of reference, the phase transformation temperature is around 725°C (1337°F) where the grain structure may be changed through heating and cooling.

Most fire events involving small vehicles or limited size fires in trucks are not expected to cause temperatures that reach the threshold of possible steel damage. Studies have shown that there is little change in the post-fire strength properties when heating is kept below about 700°C (1292°F). A PennDOT study, Effects of Fire Damage on the Structural Properties of Steel Bridge Elements, April 30, 2011, confirms that the post-fire properties of A709 Grade 50 structural steel are unaffected following heating to 650°C (1202°F). See Section 3.6 for hyperlink to the study report.

Statistically, the average yield strength of undamaged structural plate is about 8% higher than the nominal strength used for design. Therefore, even if there is a slight post-fire strength reduction, most steel members will still exceed their nominal design strength also still meet the required load rating with a slightly reduced yield strength, depending on what limit states govern the design.

The presence of distortion is another piece of evidence that can indirectly indicate the post-fire properties of steel. An AISC report indicates that members heated beyond 700°C will usually show large deflection or localized distortion when heated in a structural system. Such distortion may require remediation or repair that lessens the need to evaluate strength. The models run in the NCHRP 12-85 research showed substantial web buckling when the steel temperature exceeded about 600°C (1112°F).

Typical severe bridge fire, that it is difficult to reach steel temperatures exceeding 700°C (1292°F) without large deformation or collapse.

Based on the information generated in the NCHRP 12-85 project and that available in the literature, the following recommendations are made for evaluating the strength of structural steel with Fy ≤ 50 ksi:

- Steel heated to temperatures at or below 700°C (1292°F) can be considered to have no reduction in strength following fire exposure.

- Steels subjected to temperatures exceeding 700°C (1292°F) but less than 1000°C (1832°F) can be conservatively estimated to have a 10% reduction in both tensile and yield strength. If this is a cause for concern after load rating the post-fire bridge, mechanical property testing may be indicated to determine a more refined estimate of steel strength.

The available evidence shows that steels heated to temperatures below 650°C (1202°F) will not have any significant loss in CVN toughness. Therefore, post-fire evaluation generally should not include CVN testing. A thorough hands-on visual inspection of heat affected weldments is required. Distortion and high forces due to thermal expansion can cause cracking or distress of welds. However, some bridge members, such as those classified as NSTMs, may present a higher concern for CVN toughness evaluation. In these cases, destructive CVN testing is the only option available to determine fitness for service.

### 3.2 *Heat Treated Steels*

Heat treated steels require extra caution in the post fire strength evaluation process. ASTM A709 grades HPS 50W, 50W, HPS 70W, and HPS 100W have higher hardenability compared to low-alloy structural steels. This indicates that the mechanical properties are more dependent on the heating and cooling rate history. Grades HPS 70W and HPS 100W rely on some form of heat treatment in the manufacturing process to develop mechanical properties.

Grades HPS 50W and 50W are not classified as heat treated steels since no heat treatment is used in the manufacturing process. However, the chemistry is very similar to heat treated grades (70W, and HPS 70W) that add heat treatment to boost properties. From a strength perspective, grades 50W and HPS 50W can be expected to have little or no strength loss or mechanical property loss following fire exposure. They can be expected to perform similar to low alloy steels. However, rapid cooling during the fire process, such as from direct exposure to water from fire hoses, may have a quenching effect on the steel resulting in strength elevation. In general, HPS 50W and 50W steels should be evaluated as low alloy steels for strength and toughness. If there is evidence that the steel was subjected to high cooling rates from heavy application of water, further evaluation of toughness is required.

In general, the AWS D1.5 Bridge Welding Code allows heat forming operations for quenched and tempered steels as long as the heating temperature falls below the tempering temperature used in manufacturing. AWS conservatively limits the maximum temperature during heat forming operations to 600°C (1112°F) for the HPS grades.

Based on the information gathered in the 12-85 project and that available in the literature, the following recommendations are made for evaluating the strength of heat-treated structural steel with Fy > 50 ksi:

• Heat treated steel heated to temperatures at or below 600°C (1112°F) can be considered to have no reduction in strength following fire exposure.

• Heat treated steels subjected to temperatures exceeding 700°C (1292°F) are expected to have a significant loss of strength and require further evaluation.

• Heat treated steels that reach temperatures between 600°C (1112°F) and 700°C (1292°F) may experience strength loss or toughness degradation and require further evaluation.

### 3.3 *High Strength Bolts*

High strength bolts should be classified as heat-treated products for strength evaluation. Grade A325 and A490 structural bolts are heat treated during manufacture to obtain their mechanical properties. Heating above 600°C may alter the tensile strength and ductility of the bolts. Any bolted connections designed for strength should be subjected to further evaluation if it is suspected that the girder temperature exceeded 600°F (1112°F) at the connection location. Unlike steel girder plate material, bolt strength is relatively easy to evaluate by removing a sampling of bolts from the connection and testing the bolts.

The results show that it is inaccurate to assume that the surface temperature of bridge members equals the gas temperature of the fire. On an average, the severe fire simulations performed in the NCHRP 12-85 study show gas temperatures around 1400°C (2552°F) for a typical gasoline tanker fire while the maximum steel temperature only reached about 700°C (1292°F) and held relatively steady at that level.

### 3.4 *Fatigue and Fracture*

Fatigue life can be divided into two phases, a crack initiation phase and a crack propagation phase. Most of the life is spent in the initiation phase where no finite size cracks are present at the detail. Fire events cause very large thermal expansion of some steel members. This results in very high forces in members that are interconnected. In some cases, these forces can overstress fillet welds and introduce localized cracking. If such cracks are present, this shortens the initiation phase and can lead to premature fatigue failure. It is therefore important to perform a "hands-on" inspection of fillet welds in steel bridges that have substantial exposure to fire. The same procedures used for NSTM inspection should be followed. The overstressing effects of thermal expansion may extend beyond the heating area of the fire. Therefore, any parts of the bridge that may be affected by thermal expansion should be inspected. This includes regions in compression that are not normally evaluated for fatigue. If it can be verified that no cracks were introduced during the fire, then fatigue life should not be reduced.

Research on heat straightening of steel members does not show any special concerns for CVN toughness in grade 36, 50, 50W, or 50S steel. Therefore, for typical girder bridges with redundant members there is no evidence to suggest that fire will impair fracture toughness.

### 3.5 *Structural Analysis*

The structural analysis of a bridge due to fire damage can vary in the level of complexity. A structural analysis if warranted shall be conducted per direction of the District Bridge Engineer. The structure can be modeled as a line girder with the reduced properties of steel at 700°C (1292°F) in the appropriate regions determined by observation of the fire event. In many cases, the mechanism may predict structural collapse. This can be used as a relatively quick reality check that may invalidate or validate temperature prediction from other methods. A structural "reality check" is useful to prevent over-estimation of temperatures that can lead to over-estimation of structural damage. For an advanced analysis for fire effects describing Fire Modelling, Heat Transfer Modeling and Structural Modeling please refer to report ATLSS-18-03 NUMERICAL EVALUATION OF A SAMPLE STEEL GIRDER BRIDGE FOR A CONSTRUCTION TRAILER FIRE UNDERNEATH.

### 3.6 *Paint Condition:*

The condition of the paint on steel girder bridges can provide a rough estimate to the intensity of the temperature that the girders were exposed. A general study was performed by PennDOT looked at several different coating systems in a more controlled furnace environment. A series of photographs show the condition of coatings heated to different temperatures that can serve as samples for use by bridge inspectors. Practical application of this approach may be hampered by smoke and soot from other combustible material in the fire event. The report is at:

https://docs.penndot.pa.gov/Public/Bureaus/Bridge/BMS/Post%20Fire%20Inspection%20Guide%20For%20Steel%20Bridges.pdf

### 3.7 *Destructive Testing*

Destructive testing is the most accurate way to determine the residual properties of steel after fire. Performing tension tests according to ASTM E6 procedures provides an accurate assessment of the post-fire strength and ductility of steel. However, this requires removing samples of material from the web and flange plates, often in critical structural areas. The cost of testing therefore may be large since structural repairs may be required following removal of test specimen. Sometimes it may be possible to test stiffeners, or other secondary members without causing damage that is costly to repair. Consideration needs to be given if the secondary member material is different than the primary member material of concern. It is also possible to test sub-size specimens that can be machined from a 4 in core drilled from the material. Sub-size specimens can introduce a size effect that must be considered when comparing results to the standard size specimens used for structural steel testing. Refer to case study 2 for material testing performed on the Dauphin County SR 322 bridge.

## 4. Concrete

The post-fire strength of concrete does not recover like the post fire strength of steel when the material cools down. This indicates that strength loss in concrete due to heating is a permanent after effect of fire.

In general, spalling will be either visually obvious following a fire event. This will not be a hidden problem; therefore, the strength consequences can be addressed in terms of section loss and effects on the development capacity of the reinforcing steel.

An indicator of the level of heating that the concrete has experienced is the color of the aggregates.

500 F spalled/exposed concrete surface will appear to have a pink/red color.

900 F spalled/exposed concrete surface will appear to have a purple-gray color

The response of mild steel reinforcement in concrete is similar to structural steels. No post-fire reduction in strength is expected as long as the temperature does not exceed 700°C (1292°F). There will also be severe concrete damage at this temperature it is unlikely that there will be any "hidden" reinforcing bar strength loss that needs to be considered in post-fire strength evaluation.

Estimation of temperature in concrete members is more important than steel members as the expected concrete damage is permanent. The prestress loss and the post fire strength of strands is also very dependent on temperature. An analysis performed under NCHRP 12-85 shows the predicted surface temperatures and predicted temperatures within the concrete beam. Table 2 shows the internal departures decrease at greater depths.

**Table 2 – Predicted internal temperatures of PS Concrete beam**

| Girder | Surface Temperature | | Internal Temperature at 2" Cover | | Internal Temperature at 4" Cover | | Internal Temperature at 6" Cover | |
|---|---|---|---|---|---|---|---|---|
| | (°C) | (°F) | (°C) | (°F) | (°C) | (°F) | (°C) | (°F) |
| 1 | 830 | 1526 | 425 | 797 | 200 | 392 | 180 | 356 |
| 4 | 1050 | 1922 | 525 | 977 | 270 | 518 | 180 | 356 |

For these beams, the bottom strand layer is located 2 in. above the bottom surface. The exterior girder (girder 1) is not expected to have much strand strength reduction in the bottom strand layer located 2 in. above the bottom surface. The interior girder (girder 4) might be expected to have about a 30% strength reduction in the bottom strands. No strength reduction is predicted in the upper strand layers in either girder. Therefore, even for the most severely exposed interior girder, most of the strands are expected to retain their pre-fire capacity.

A prestressed girder bridge that was damaged by a train fire was the Puyallup River Bridge in Washington State reported by Stoddard (Stoddard, 2004). The prestressed girders were engulfed by a large ethanol fire for a fire duration of about one hour. Post fire testing indicated there was no detectable loss of prestressing force in the exposed tendons at the bottom of the beam.

### 4.1 *Non-Destructive Testing/Observation*

Degradation of the concrete cross section due to delamination and/or spalling can be easily detected by a detailed post-fire inspection. Spalling where the concrete has fallen off is easy to visually detect. Sounding the concrete with a hammer can detect delaminations that indicate the early stages of spalling. Data is available in the literature relating the color of concrete to post-fire compression capacity. A pinkish hue generally serves as an indication of altered material properties. Tests such as rebound hammer may also be useful to evaluate concrete strength.

Cracking in tension areas of prestressed concrete members provides a strong indication that there is a loss of effective prestressing force. Open cracks will be accompanied by measurable vertical deflection and are a clear sign that the prestressing force has been compromised. Tight cracks with no apparent vertical deflection are more difficult to evaluate since they may have been formed by thermal expansion at high temperature. In this case, visual crack observations should be corroborated with other assessment means such as temperature estimation.

### 4.2 *Destructive Testing*

If concrete strength loss is suspected, destructive testing is the preferred approach to accurately quantify this effect. This typically involves coring cylinders from the affected members and performing standard compression tests. This can determine if the concrete meets the required strength capacity. The available locations for coring may be limited in prestressed members in regions with close strand and reinforcement spacing. For design, concrete strength is measured at the 28-day point in the curing process and it is well known that strength continues to increase with age. It is typical for tests performed on older age concrete to show strength exceeding f'c. Even with a slight reduction in capacity relative to the pre-fire strength, the concrete strength may still exceed the nominal design requirements. Refer to case study 2.

## 5. Post Fire Inspection Considerations

Steel members recover most of their strength when they cool following a fire event. This also applies to concrete reinforcing steel and prestressing strands. The compression strength of concrete does not recover following heating and may be slightly decreased below the high temperature strength. However, the relatively short duration of most bridge fires and the thermal properties of concrete usually results in minimal internal material damage for concrete members. The tension capacity of steel generally will have the most effect on strength, therefore structures can be expected to have higher strength after the fire than during the fire event. Structures that appear to have only minor deflections and damage will usually be safe to allow access for inspection. This does not necessarily apply to structures that have collision or other severe damage to concrete supporting or bearing elements.

### 5.1 *Composite Action*

Visual examination of the structure should be performed to detect any evidence of slip at the beam to deck interface. If such evidence exists, further evaluation may be specified by the District Bridge Engineer. Observation of behavior underneath the structure when live loads are on the bridge for evidence composite action slip. Sometimes movement, either vertical or horizontal can be detected as a truck crosses the bridge. If problems are still suspected, a field instrumentation study may be indicated. This can detect bridge stiffness and the effective neutral axis location for the composite girders. In general, there is probably no reason to perform such a study unless there is a strong indication of problems based on simple observations.

### 5.2 *Web Distortion*

AASHTO LRFD Bridge Design Specifications allows consideration of tension field action when evaluating shear strength and the capacity based on current codes may be greater than that calculated in the original design based on allowable stress design. This provides some relief when calculating the shear capacity of sections with web distortion. It is highly likely that large web distortions will be present following a severe fire event, particularly for welded plate girders. Analysis of large web distortions would require a 3D FEA of the girder. Thus, a practical solution is to add stiffeners instead of performing an advanced analysis.

Distortion of transverse stiffeners, cross frames, diaphragms, and other secondary members need to be assessed as required. It may be possible to ignore some deformation of transverse stiffeners if they are still capable of performing their web stiffening and connection functions. The welds near any such deformations need to be carefully inspected. Axial force members such as those used in cross frames or diaphragms often show effects of buckling or distortion following a severe fire event. The distortion typically occurs at peak heating when differential thermal expansion is at a maximum and the member strength is reduced. Some members may undergo a force reversal between the heating and cooling phases of a fire event. A member that is normally in tension may go into compression at high temperatures and revert back to tension when the structure cools.

### 5.3 *Bearings and Expansion Joints*

It is common to see bridges that have bearing damage and possible contact damage with abutment walls following fire events. The thermal expansion (longitudinal and transverse) can be much higher than anticipated by normal bridge temperature changes. Expansion at the bridge ends reflects the integration of the thermal expansion occurring at every section of the bridge. It is therefore possible to see bearing and joint problems away from the locations directly affected by the fire. Therefore, all expansion joints and bearings, regardless of location, require thorough inspection and evaluation following a significant fire event. Much of the bridge expansion present at peak temperature will recover as the bridge cools. Bearings may have suffered damage at the peak expansion point that may not be apparent after the bridge contracts. This is a particular concern for elastomeric or other non-mechanical bearing types.

Expansion joints require thorough inspection, again irrespective of their location relative to the fire. The large forces present at peak expansion may have caused compression damage to the joints and their attachment connections to the concrete. Bridges that experience any significant vertical deflection at high temperatures that recover as the bridge cools may cause tensile damage to the joints.

### 5.4 *Connections*

For strength evaluation, A325 and A490 structural bolts are heat-treated products that depend on controlled heating to obtain their properties during manufacturing. Lacking more precise information, it is recommended that bolts from any connections in primary members that may have experienced temperatures exceeding 200°C (392°F) should be tested. This can easily be done by removing a few bolts from the connection and testing them in a Skidmore device or in direct tension. Direct tension testing may be easier if the threads are distorted from the initial tensioning. If the bolts are found to have insufficient strength, it is relatively easy to replace the bolts incrementally to restore the connection capacity.

One factor that needs to be considered when evaluating primary connections at STRENGTH I is the 75% rule present in the AASHTO design specifications. Most girder splices built prior to the 9th Edition of the LRFD Bridge Design Specifications are located at locations with low moment in the girders. However, the connections are designed for at least 75% of the moment capacity of the member. The bolts in the connection may therefore have much higher shear strength than required for the actual loads. The engineer should consider this fact before replacing bolts if only a slight reduction in bolt strength is suspected.

It is a bit more difficult to determine the slip capacity for SERVICE II. Bolts are pretensioned to a large portion of their tension capacity during installation to provide frictional clamping force. Depending on the temperature and duration of the fire event, creep may be expected to allow relaxation of the initial bolt tension.

### 5.5 *Bridge Decks*

A detailed cracking survey should be performed following a significant fire event. Results should be interpreted using the same analysis that is used on non- fire exposed decks. Crack width can be evaluated concerning the expected effect on permeability. If concrete material changes are suspected, the permeability can be evaluated through destructive testing.

### 5.6 *Summary*

In summary of list of items to be inspected for steel girders, concrete girders and bridge decks as listed below.

Steel Girders:

Visual Inspection:

- Permanent deflection
- Paint condition
- Web distortion
- Welds – web to flange, stiffener and connection plate welds
- Bolts in bolted connections
- Bearings
- Joints

NDE and Testing:

- Specimen cores for mechanical properties including CVN
- Bolted connections – bolt removal for mechanical testing of high strength bolts

Concrete Girders:

Visual Inspection:

- Permanent deflection
- Spalling and delaminations
- Cracking
- Exposed reinforcing steel – mild and/or prestressing
- Bearings
- Joints

Testing:

- Concrete Cores for compression testing
- Rebound hammer
- Ultrasonic impact echo

Bridge Decks:

Visual Inspection:
- Cracking
- Spalling
- Delaminations
- Joints

Testing:
- Concrete core for compression testing
- Concrete core for permeability testing
- Rebound hammer
- Ultrasonic impact echo

## 6. Examples

Examples of test results for two bridges are presented to use as reference.

### 6.1 *Liberty Bridge Fire*

Through an experimental study by Lehigh University the fire temperature exceeded $1000^{0}$C (1832°F). At this elevated temperature white deposits formed on the steel as illustrated below.

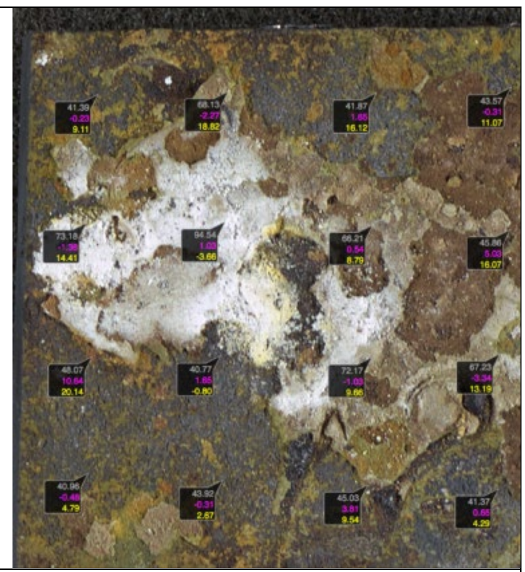
| | |
|---|---|
|  |  |
| White Deposits on lacing | White deposit from experiment at $1000^{0}$C (1832°F) |

**Figure 2 – Lacing member depicting white deposits.**

**Table 3 - Mechanical Property Testing Results of Samples from Liberty Bridge**

| SAMPLE ID | (ksi) TENSILE STRENGTH | (ksi) YIELD STRESS (0.2% OFFSET) | (%) ELONGATION IN 4D (MANUAL) | (%) REDUCTION OF AREA |
|---|---|---|---|---|
| DS Web West - 1 | 59.5 | 29.9 | 38 | 62 |
| DS Web West - 2 | 60.5 | 30.5 | 38 | 60 |
| DS Web West - 3 | 59.5 | 30.0 | 39 | 58 |
| DS Web East - 1 | 57.0 | 29.2 | 41 | 66 |
| DS Web East - 2 | 56.5 | 29.0 | 39 | 66 |
| DS Web East - 3 | 58.0 | 28.4 | 39 | 63 |
| US Web West - 1 | 58.5 | 28.8 | 39 | 62 |
| US Web West - 2 | 58.5 | 29.3 | 39 | 60 |
| US Web West - 3 | 58.5 | 28.3 | 41 | 63 |
| US Web East - 1 | 60.0 | 28.4 | 38 | 62 |
| US Web East - 2 | 60.0 | 28.8 | 38 | 63 |
| US Web East - 3 | 60.5 | 28.5 | 39 | 63 |
| Mid Web West - 1 | 65.5 | 30.9 | 34 | 56 |
| Mid Web West - 2 | 64.0 | 31.0 | 34 | 59 |
| Mid Web West - 3 | 66.0 | 30.8 | 28 | 40 |
| Mid Web East - 1 | 56.0 | 29.2 | 42 | 65 |
| Mid Web East - 2 | 56.5 | 29.6 | 42 | 68 |
| Mid Web East - 3 | 56.0 | 28.2 | 41 | 64 |

**Procedures/Methods:** 86-TT-2, Rev. 15, Room Temp. Tensile Testing for Metallic Materials


**Figure 3 – Location of 4-inch diameter Cores**

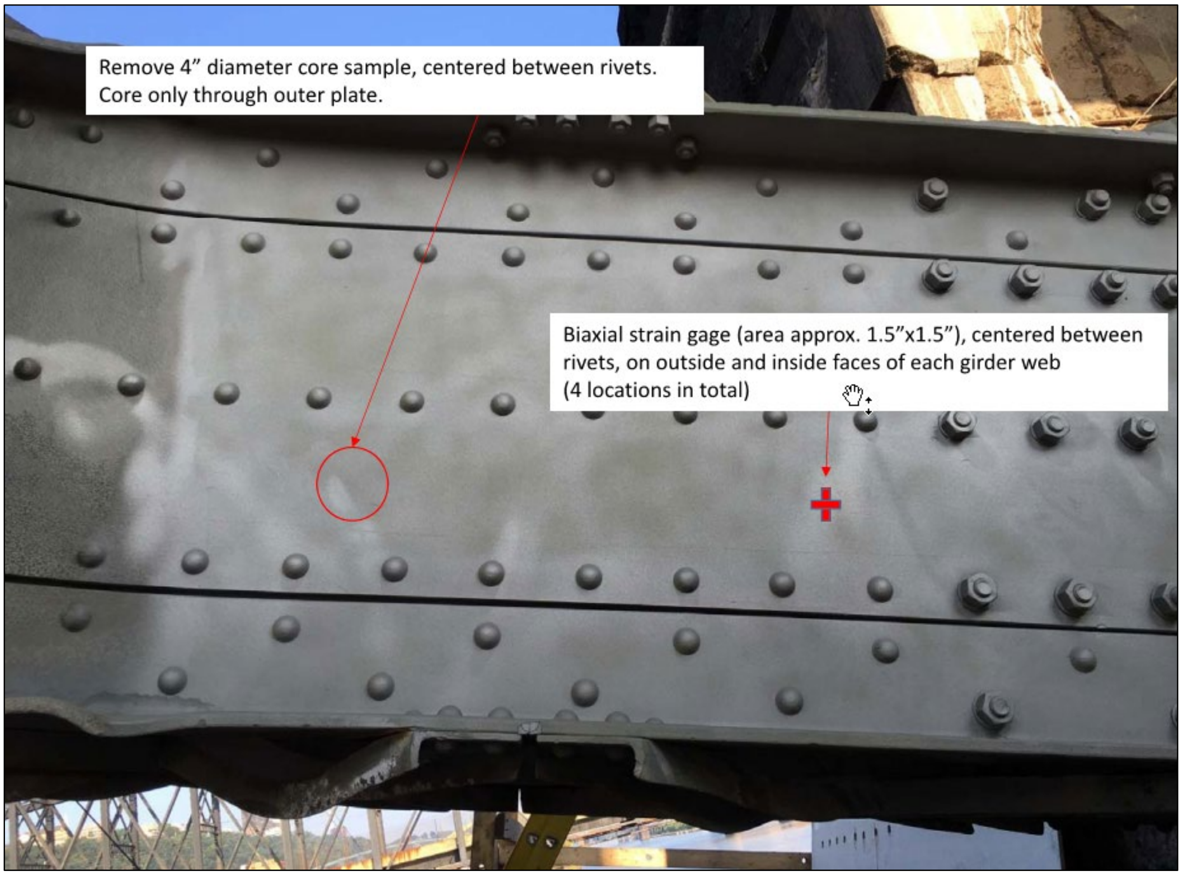



**Figure 4 – Location of 4-inch diameter Cores**

### 6.2 *Dauphin County SR 322*

The results of concrete and steel testing results and locations are presented below.

Concrete test results and locations:

**Table 4 – Concrete Testing Results**

| May 2013: I-81 and Rt-22/322 Ramp Project -Diesel Tanker Damage | | | | | | | DCB - 5/19/13 | |
|---|---|---|---|---|---|---|---|---|
| Core # | Arrival at Lab Date | Structure | Good/ Bad | Test | 40 hr Date | 40 hr Time | Strength PSI | Lab No. |
| 1 | 5/16/2013 | S-12184 Abutment Wing A Side Ramp B | Bad | Petro | | | | 13-48163-1 |
| 2 | 5/16/2013 | S-12184 Abutment Wing A Side Ramp B | Bad | Comp. Str | 18-May | 6:30 AM | 4310 | 13-48163-2 |
| 3 | 5/16/2013 | S-12184 Abutment Wing A Side Ramp B | Bad | Comp. Str | 18-May | 6:30 AM | 4800 | 13-48163-3 |
| 4 | 5/16/2013 | S-12184 Wing A Side Backwall Ramp B | Good | Petro | | | | 13-48166-1 |
| 5 | 5/16/2013 | S-12184 Abutment Wing B Side Ramp B | Good | Petro | | | | 13-48171-1 |
| 6 | 5/16/2013 | S-12184 Abutment Wing B Side Ramp B | Good | Comp. Str | 19-May | 10:50 AM | 4190 | 13-48171-2 |
| 7 | 5/16/2013 | S-12184 Abutment Wing B Side Ramp B | Good | Comp. Str | 19-May | 10:50 AM | 4460 | 13-48171-3 |
| 8 | 5/16/2013 | S-12184 Wing B Side Backwall Ramp B | Good | Petro | | | | 13-48166-2 |
| 9 | 5/16/2013 | S-12188 Flyover Pier #2 SR 22 | Good | TBD - Small | | | | 13-48167-1 |
| 10 | 5/16/2013 | S-12188 Flyover Pier #2 SR 22 | Good | Comp. Str | 19-May | 10:30 AM | 4800 | 13-48167-2 |
| 11 | 5/16/2013 | S-12188 Flyover Pier #2 SR 22 | Good | Petro | | | | 13-48167-3 |
| 12 | 5/16/2013 | S-12188 Flyer Pier #2 SR 22 | Good | TBD - Small | | | | 13-48167-4 |
| 13 | 5/16/2013 | S-12188 Flyover Bridge - Pier Caps | Bad | Petro | Polished | | 5/17/2013 | 13-48165-1 |
| 14 | 5/16/2013 | S-12188 Flyover Bridge - Pier Caps | Bad | Petro | Polished | Polished | 5/17/2013 | 13-48165-2 |
| 15 | 5/16/2013 | S-12188 Flyover Bridge - Pier Caps | Bad | Petro | Polished | Polished | 5/17/2013 | 13-48165-3 |
| 16 | 5/16/2013 | S-12188 Flyover Bridge - Pier Caps | Bad | Comp. Str | 18-May | 7:20 AM | 3690 | 13-48165-4 |
| 17 | 5/16/2013 | S-12188 Flyover Bridge - Pier Caps | Bad | Comp. Str | 18-May | 7:20 AM | 4250 | 13-48165-5 |
| 18 | 5/16/2013 | S-12184 Piers SR 22 Ramp B | Bad | Petro | Polished | | 5/17/2013 | 13-48168-1 |
| 19 | 5/16/2013 | S-12184 Piers SR 22 Ramp B | Bad | Comp. Str | 18-May | 7:30 AM | 5140 | 13-48168-2 |
| 20 | 5/16/2013 | S-12184 Piers SR 22 Ramp B | Bad | Comp. Str | 18-May | 7:30 AM | 5750 | 13-48168-3 |
| 21 | 5/16/2013 | S-12184 Pier #1 Pier Caps Ramp B | Bad | Petro | Polished | | 5/17/2013 | 13-48169-1 |
| 22 | 5/16/2013 | S-12184 Pier #1 Pier Caps Ramp B | Bad | Comp. Str | 18-May | 7:30 AM | 4380 | 13-48169-2 |
| 23A | 5/16/2013 | S-12184 Pier #1 Pier Caps Ramp B | Bad | Comp. Str | 18-May | 7:30 AM | 4970 | 13-48169-3A |
| 23B | 5/16/2013 | S-12184 Pier #1 Pier Caps Ramp B | Bad | Comp. Str | 18-May | 7:30 AM | 4610 | 13-48169-3B |
| 24 | 5/16/2013 | S-12184 Pier #1 Pier Caps Ramp B | Bad | Comp. Str | 18-May | 7:30 AM | 5020 | 13-48169-4 |
| 25 | 5/16/2013 | S-12184 Pier Caps Pier #1 | Good | Petro | | | | 13-48170-1 |
| 26 | 5/16/2013 | S-12184 Pier Caps Pier #1 | Good | Comp. Str | 19-May | 11:00 AM | 4040 | 13-48170-2 |
| 27 | 5/16/2013 | S-12184 Pier Caps Pier #1 | Good | Comp. Str | 19-May | 11:00 AM | 4190 | 13-48170-3 |
| 28 | 5/17/2013 | S-12184 Pier Ramp B | Good | Comp. Str | 19-May | 11:05 AM | 5620 | 13-48172-1 |
| 29 | 5/17/2013 | S-12184 Pier Ramp B | Good | Petro | | | | 13-48172-2 |
| 30 | 5/17/2013 | S-12184 Pier Ramp B | Good | Comp. Str | 19-May | 11:05 AM | 5040 | 13-48172-3 |
| 31 | 5/17/2013 | S-12188 Flyover Pier #1 Pier Cap | Bad | Petro | | | | 13-48173-1 |
| 32 | 5/17/2013 | S-12188 Flyover Pier #1 Pier Cap | Bad | TBD - Small | | | | 13-48173-2 |
| 33 | 5/17/2013 | S-12188 Flyover Pier #1 Pier Cap | Bad | TBD - Small | | | | 13-48173-3 |
| 34 | 5/17/2013 | S-12188 Flyover Pier #1 Pier Cap | Bad | Comp. Str | 19-May | 11:05 AM | 4780 | 13-48173-4 |
| 35 | 5/17/2013 | S-12188 Flyover Pier #1 Pier Cap | Bad | Comp. Str | 19-May | 11:05 AM | 4440 | 13-48173-5 |
| 36 | 5/17/2013 | S-12188 Flyover Pier #1 | Good | Petro | | | | 13-48174-1 |
| 37 | 5/17/2013 | S-12188 Flyover Pier #1 | Good | Comp. Str | 19-May | 11:05 AM | 4250 | 13-48174-2 |
| 38 | 5/17/2013 | S-12188 Flyover Pier #1 | Good | Comp. Str | 19-May | 11:05 AM | 3860 | 13-48174-3 |

**Table 5 – Summary Results at various substructure locations**

| Ramp B | Range of Compressive Strengths (psi) | | Avg. Compressive Strength (psi) | |
|---|---|---|---|---|
| | Good Areas | Bad Areas | Good Areas | Bad Areas |
| Pier Caps | 4040-4190 | 4380-5020 | 4115 | 4745 |
| Piers | 5040-5620 | 5140-5750 | 5330 | 5445 |
| Abutment | 4190-4460 | 4310-4800 | 4325 | 4555 |

**Figure 5 – Location of Concrete Cores**

Steel test results and locations:

**Table 6 – Steel Mechanical Property Results**

## I-81 Ramp B Steel Test Results

| ID# | Thickness | CVN (avg) NFC Zone 2 | | Yield ksi (lowest) | | Tensile ksi (lowest) | | Elongation % (lowest) | |
|---|---|---|---|---|---|---|---|---|---|
| | | primary | verification | primary | verification | primary | verification | primary | verification |
| | | PRL | *PennDOT* | PRL | *PennDOT* | PRL | *PennDOT* | PRL | *PennDOT* |
| **A36** | | 15ft-lbs@40°F | | 36 ksi min | | 58-80 ksi | | 23% min | |
| G1-2-BF-1 | 1½" | 93.0 | *90.7* | 43.9 | *37.9* | 70.0 | *70.7* | 33.6 | *34* |
| G1-2-W-1 | ⅜" | 54.7 | *72.7* | 40.0 | *37.9* | 70.1 | *72.4* | 35.9 | *47* |
| G2-2-BF-1 | 1½" | 61.7 | *68.3* | 35.5 | * | 70.3 | *70.6* | 33.6 | *33* |
| G5-2-W-1 | ⅜" | 61.0 | *65.2* | 44.9 | *44.2* # | 76.0 | *74.5* | 40.6 | *34* |
| G5-2-W-2 | ⅜" | 53.0 | *54.6* | 42.9 | *44.2* | 77.3 | *79.3* | 39.1 | *36* |
| G5-2-BF-1 | 2¼" | 62.0 | *68.4* | 37.3 | *37.7* | 69.6 | *70.4* | 35.2 | *36* |
| **A572** | Lab Testing | | | 50 ksi min | | 65 ksi min | | 21% min | |
| G3-2-TF-1 | 1½" | 46.0 | *39.6* | 52.5 | *54.0* | 83.0 | *85.0* | 27.0 | *26* |
| G4-2-W-1 | ⅜" | 39.0 | *44.6* | 54.5 | *54.9* | 77.0 | *78.4* | 32.0 | *31* |
| **A588** | | 20ft-lbs@40°F>2" thickness | | 50 ksi min | | 70 ksi min | | 21% min | |
| G4-2-TF-1 | 2¼" | 119.0 | *162.5* | 47.3 | *46.9* | 74.0 | *74.9* | 30.0 | *29* |
| G4-2-BF-1 | 1⅞" | 99.0 | *93.0* | 51.0 | *52.0* | 78.5 | *79.7* | 29.0 | *28* |

* invalid yield due to test grip/specimen incompatibility

\# one of the initial 2 samples had a low yield at 32.2 ksi, so PennDOT prepared and tested 2 additional samples which both had yield results greater than 36 ksi and is considering the 32.2 ksi as an outlier due to the preparation and subsequent compromise in testing.

**Figure 6 – Location of steel coupon**

# APPENDIX IP 02-H

Fatigue & Fracture Plan

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# Fatigue and Fracture (F&F) Plan

**Structure ID (5A01):** ______ **Original F&F Plan Date:** ______
**Structure Name:** ______ **Review Date:** ______
**District:** ______ **BRKEY (5A03):** ______ **Update Date (if changed):** ______

*Note: This F&F plan is in accordance with PennDOT Pub 238 IP 2.4.5.1. This plan shall be reviewed prior to each NSTM inspection and updated if needed. A copy of the latest version of the F&F Plan shall be uploaded to BMS2.*

| **1. QC Req'd Check: (If either criteria applies, a NSTM QC Checklist must be completed)** | *Yes/No* |
|---|---|
| *1) NSTM condition rating has changed* | |
| *2) Field conditions and/or required NSTM details (Cat. D - E') are not accurately reflected within the plan* | |

**2. Bridge Conditions:**

Deck (1A01) = ______ Super (1A04) = ______ Sub (1A02) = ______ NSTM (1A15) = ______
Posting (VP02) = ______ Do NSTM's require postings? ______
Notes:

**3. NSTM Inspection Scope and Interval:**

*Note: Indicate which portions of the structure require a hands-on NSTM Inspection and the interval required.*

**Routine Inspection:**

**Special Inspection:**

**4. Access Equipment and Special Testing Needs:**

*Note: List any access equipment necessary to complete the NSTM inspection. Also, list any special testing equipment required in addition to the standard magnifying glass, dye penetrant, and lighting for a NSTM inspection (i.e. ultrasonic testing equipment for testing of pins).*

**Routine Inspection:**

**Special Inspection:**

**5. Approval for Limited Scope Inspection (If Required, check the approved item)**

*Note: Approval is required only for the following cases: Interim inspection is a limited inspection (Does not include all NSTMs), a less than full hands on Routine inspection of the NSTMs is proposed for concrete encased NSTMs, or NSTMs don't control the superstructure rating and an interval longer than required by Pub 238 Table IP 2.3.2.4-1 is scheduled. For locally owned bridges, Limited scope must be approved by a Professional engineer working for the owner or their consultant.*

______ The proposed Limited scope Interim F&F plan is satisfactory to meet NSTM inspection requirements.

______ The proposed less than full hands-on NSTM Routine inspection of the concrete encased NSTMs is satisfactory to meet FC inspection requirements.

______ The proposed inspection interval, which is longer than required by Pub 238 Table 2.3.2.4-1, is satisfactory to meet NSTM inspection requirements due to NSTM not controlling the Superstructure Rating.

| District Bridge Engineer or Local Owner Engineer | ______ Signature | ______ Date |
|---|---|---|

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# Fatigue and Fracture (F&F) Plan

Structure ID (5A01): ____________ BRKEY (5A03): ____________

Structure Name: ____________

## 6. Fatigue/Fracture Prone Members and Details:

*Note: The following Fatigue/Fracture prone members and details have been identified. Inspection notes for these members and details shall be recorded on the IF Screen in BMS2. See the attached sketches for locations of the members and details.*

| NSTM Location | NSTM Type | NSTM Description | NSTM Detail | AASHTO Fatigue Category | Member Inspection Procedures (Note Inspection guidance, special access/equipment needs, retrofits, specific problematic details, etc) |
|---|---|---|---|---|---|
| IF01 | IF02 | IF03 | IF04 | IF05 | IF10 |
| | | | | | |
| | | | | | |
| | | | | | |
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# APPENDIX IP 02-I

Uncoated Weathering Steel

Bridge Safety Inspection and

Maintenance Manual

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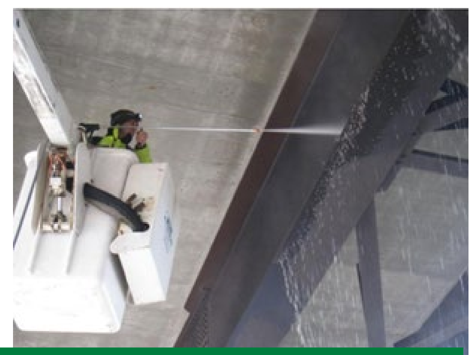
PENNSYLVANIA DEPARTMENT OF TRANSPORTATION

**ASTM Weathering Steels**
A242
A588
A709 50W
A709 HPS 50W/
70W/100W



# Uncoated Weathering Steel Bridge Safety Inspection and Maintenance Manual

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2-24

Photographs, Figures, and Table Sources:

- Cover: Crampton et al., 2013 – Bottom Right; NSBA/AISC 2022 - Top Right and Bottom Middle; PennDOT – All Others
- Chapters 1, 2, and 3: Except where cited otherwise, PennDOT

**Abbreviations**

| | |
|---|---|
| AISC | American Institute of Steel Construction |
| AASHTO | American Association of State Highway and Transportation Officials |
| BIRM | FHWA Bridge Inspector's Reference Manual |
| BMS2 | Bridge Management System 2 |
| BMS3 | Bridge Management System 3 |
| FHWA | Federal Highway Administration |
| IowaDOT | Iowa Department of Transportation |
| MBE | AASHTO Manual for Bridge Evaluation |
| MBEI | AASHTO Manual for Bridge Element Inspection |
| MnDOT | Minnesota Department of Transportation |
| NSBA | National Steel Bridge Alliance |
| NSTM | Non-Redundant Steel Tension Member |
| NYSDOT | New York State Department of Transportation |
| PennDOT | Pennsylvania Department of Transportation |
| UWS | Uncoated Weathering Steel |
| UWS-SME | Uncoated Weathering Steel - Subject Matter Expert |

**Glossary**

**Appraisal Rating**: An evaluation of a functionality of a bridge characteristic or component in comparison to current standards for the highway system the bridge serves.

**Bridge:** A structure, including supports, erected over a depression or an obstruction, such as water, highway, or railway, and having a track or passageway for carrying traffic or other moving loads.

**Bridge Management System (BMS)**: A system designed to optimize the use of available resources for the inspection, maintenance, rehabilitation, and replacement of bridges.

**Bridge Owner:** An organization or agency responsible for the inspection, load rating, and maintenance of highway bridges.

**Condition Rating:** An evaluation of the physical condition of a bridge component in comparison to its original as-built condition.

**Condition State Rating:** The evaluation of the physical condition of bridge elements (pieces of the bridge), classifying the extent and severity of individual defects as defined in the AASHTO MBEI and PennDOT Publication 100A BMS 2 Coding Manual. The extent of the defect is captured by rating defined portions (quantities) of the element, such as square feet or lineal feet. The severity of the defect is defined by element and material, using four condition states: CS-1 (good), CS-2 (fair), CS-3 (poor), and CS-4 (severe, with an effect on structural capacity or serviceability).

**Load Rating:** The determination of the live load carrying capacity of an existing bridge using existing bridge plans supplemented by information gathered from a field inspection**.**

**National Bridge Inspection Standards (NBIS)**: Federal regulations establishing requirements for inspection procedures, frequency of inspections, qualifications of personnel, inspection reports, and preparation and maintenance of bridge inventory records. The NBIS apply to all structures defined as bridges located on or over all public roads.

**National Bridge Inventory:** Inventory containing SI&A information for the nation's NBIS bridges.

**Structure Inventory and Appraisal Sheet (SI&A):** A summary sheet of bridge data required by NBIS.

# Table of Contents

**Table of Contents**

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# Introduction and Purpose

This *Uncoated Weathering Steel - Bridge Safety Inspection and Maintenance Manual* has been developed to provide bridge safety inspection and maintenance requirements and guidance related to the specific needs of uncoated weathering steel (UWS) bridges.

UWS bridge safety inspection and maintenance requirements, recommendations, and guidance reside in many documents from a multitude of sources including but not limited to those from PennDOT, FHWA, AASHTO, NCHRP, NSBA/AISC, and various research projects. The purpose of this manual is to consolidate select and relevant information from these sources into a single document for use by practicing bridge safety inspectors and maintenance coordinators.

A graphical summary of information covered in this manual, entitled *Roadmap to Achieving Better Uncoated Weathering Steel Bridges,* is provided in Figure 1 on the following page.

The information in this manual focuses on UWS only and is not intended to be inclusive of all requirements and actions required for the full coverage of NBIS bridge safety inspections and bridge maintenance operations.

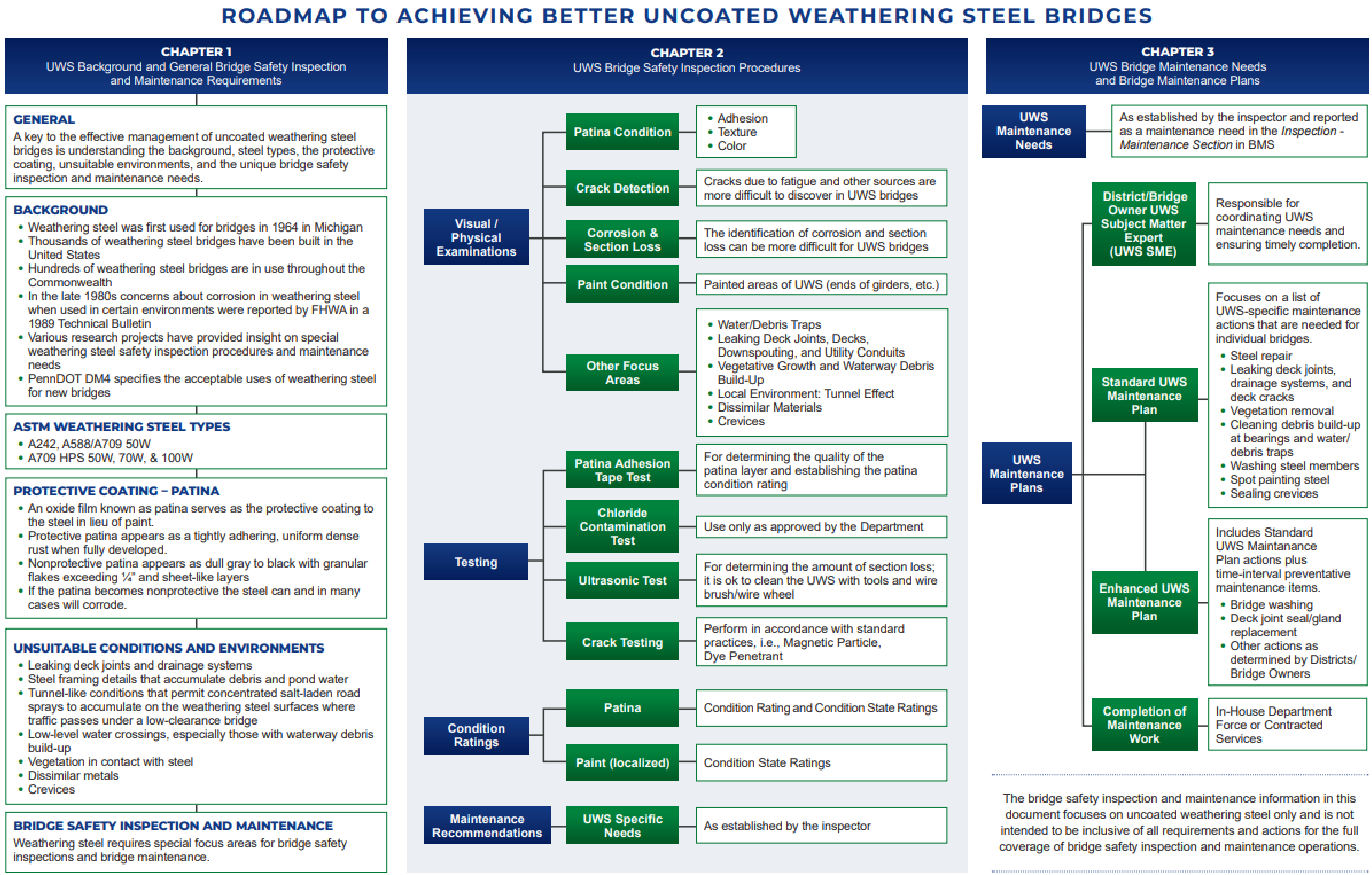



**Figure 1 - Roadmap to Achieving Better Uncoated Weathering Steel Bridges**

# Chapter 1 – UWS Background and General Bridge Safety Inspection and Maintenance Requirements

## 1.1 Background

Weathering steel was first used for bridges in 1964 in Michigan (FHWA BIRM 2022). Since then, thousands of bridges have been constructed of Uncoated Weathering Steel (UWS) in the United States, including hundreds in Pennsylvania. See Figures 2, 3 and 4.

In the proper environments, weathering steel does not necessitate painting but produces its own protective coating. When exposed to the atmosphere, weathering steel develops a protective oxide film, which seals and protects the steel from further corrosion. This oxide film, known as patina, is actually an intended layer of surface rust which protects the member from further corrosion and loss of material thickness.


**Figure 2 – UWS Delta K-Frame Bridge**

**Figure 3 – UWS Deck Truss Bridge**

**Figure 4 – Views of Multi- Configuration UWS Bridge and Rebuild of Girder at Pinned Joint**

The frequency of surface wetting and drying cycles determines the patina's texture and protective nature. The wetting cycle includes the accumulation of moisture from rainfall, dew, humidity, and fog, in addition to the spray of water from traffic. The drying cycle involves drying by sun and wind. Alternate cycles of wetting and drying are essential to the formation of the protective patina. Patina formation time will vary according to the many factors as indicated and may take 2-3 years or more to form completely. The protective film may not form if UWS remains wet for long periods of time.

The early successes of UWS in bridges led to the use of this steel in locations where the steel could not attain and/or retain the protective patina, and where corrosion progressed beyond the intended layer of surface rust, especially in wet and chloride contaminated environments.

In 1989 the Federal Highway Administration issued a Technical Advisory, *T 5140.22: Uncoated Weathering Steel in Structures*. This document recognizes the need to consider locations and conditions when using uncoated weathering steel in highway structures. Effective maintenance programs are also noted as essential, especially for uncoated weathering steel bridges. Guidelines are provided for the use of weathering steel in structures with four focus areas: Environment, Location, Design Details, and Maintenance Actions.

The 1989 FHWA Technical Advisory was issued several months following the publication of the National Cooperative Highway Research Program (NCHRP) *Report 314 Guidelines For The Use Of Weathering Steel In Bridges* which recognizes long-term performance concerns with the use of unpainted weathering steel under certain conditions.

In response to recommendations specified in the FHWA Technical Advisory, many transportation agencies implemented new design and maintenance requirements for UWS bridges. These include, but are not limited to, overpass clearance restrictions and application of a coating system, such as paint, to the ends of weathering steel members near deck expansion joints and at other steel component locations. See Figure 5.


**Figure** 5 **- Painted Steel Box Cross Girder, Painted End of Girder**

As of late 1989, PennDOT had implemented uncoated weathering steel policy in Publication 15, *Design Manual Part 4 - Structures*. This policy included the following:

- Prohibited use of weathering steel in: acidic or corrosive environments; locations subject to salt spray or fog; depressed roadway sections with less than 20-foot of underclearance; low underclearance situations where the steel is either 5 feet from normal water elevation or continuously wet; and where the steel may be buried in soil.
- Prohibited weathering steel for deck expansions dams, and for stringers and other members under open steel decking.
- Added design criteria for use of weathering steel, including: minimization of deck expansion joints; avoid details that retain water and debris; paint steel at least 5 feet on each side of deck expansion joints; add drip plates (bars); and protect substructure units against staining.
- Added criteria for painting specific areas of existing uncoated weathering steel bridges, including: beam ends up to 5 feet from leaking joints; and where the steel is exposed to or subject to salt water spray.

In 2022 the National Steel Bridge Alliance (NSBA) in conjunction with the American Institute of Steel Construction (AISC) issued the *Uncoated Weathering Steel Reference Guide.* This document provides guidance to bridge owners and designers on when it is appropriate to utilize UWS in bridge construction, and how to design, detail, fabricate, construct, inspect, preserve, maintain, and repair uncoated weathering steel bridges. Much of the content builds upon the information contained in NCHRP *Report 314* and the FHWA Technical Advisory. The document also adds information and recommendations from other UWS research sponsored by state departments of transportation, including but not limited to a report entitled *Assessment of Weathering Steel Bridge Performance in Iowa and Development of Inspection and Maintenance Techniques* prepared for the Iowa Department of Transportation and the FHWA in 2013 by Wiss, Janney, Elstner Associates, Inc.

Through research funding at the FHWA Office of Infrastructure Research and Development, FHWA has sponsored Project 11-0046 *Better Understanding of Weathering Steel Performance.* The project is defined as having a goal to better understand the macroenvironments or microenvironments and the geometric design variables that influence the corrosion performance of weathering steel. As of early-2024 the final report is in the editing process. It is hopeful that this research project will provide more insight into the design, inspection, and maintenance needs for the “tunnel-effects” at overpass bridges.

## 1.2 General Bridge Safety Inspection Requirements

The inspection of uncoated weathering steel bridges differs from and can be more difficult than the inspection of ordinary painted steel bridges. Unlike painted structures where rust is undesirable and its appearance serves as a warning of incipient paint failure, in uncoated weathering steel bridges the entire structure, or the portion that is unpainted, is covered with rust, also known as patina. The inspector must distinguish between a protective and non-protective patina with consideration of knowing that the patina will form only if the steel is able to cycle between being completely wet and then completely dry. The inspector must also establish the existence and limits of corrosion and section loss.

If the patina has not properly formed, the steel will continue to corrode. Non-protective patina will appear as either continuous flaking of the plates and/or by plate delamination. The plate delamination will appear as open cracks along the vertical edges of the flange plates or by blistering (bulging) on flat surface areas. See Figure 6.

**Figure 6 - Large, Thick Loose Flakes Covering the Entire Flange Surface (Source: Crampton et al., 2013)**

Therefore, it is important for the inspector to distinguish between the protective layer of rust and advanced corrosion that can lead to section loss. It is also important to note that fatigue cracks initiating from areas of section loss and rust pits of UWS can be difficult to detect visually.

The presence of chlorides and other deleterious substances on and sometimes just near the uncoated weathering steel members will severely and negatively affect the formation of the protective patina. These include: salt-laden deck drainage due to leaking scuppers and leaking joints, cracks through the deck that allow deck drainage to penetrate and seep onto the weathering steel, accumulated debris on steel surfaces, encroaching vegetation that prevents air circulation, and pooling water due to poor detailing. These conditions, even though they may not yet affect the member rating, must be reported for maintenance actions. Failure to address these conditions in a timely manner can significantly reduce the performance and lifespan of the uncoated weathering steel.

Uncoated weathering steel bridges built in Pennsylvania prior to 1990 are of special concern since these bridges were likely not designed and built with the restrictions and features, as indicated in Section 1.1, which promote and help ensure the formation and longevity of the patina on weathering steel surfaces.

Even with the implementation of these restrictions and features, some bridges in Pennsylvania designed and built after 1989 have been found to have localized problematic conditions where the patina has become non-protective. See Figure 7.

**Figure 7 – Extensive Rust Laminations Along the Lower Chord of a Modern Deck Truss Bridge**

Bridges passing over roadways that are heavily treated with deicing agents for winter roadway maintenance can be susceptible to a severe local environment (see Figure 8). This is because airborne salt-laden road spray from the underlying roadway can collect on the superstructure. The combination of vertical and horizontal clearances combined with variation in roadway elevation can create what is known as a tunnel effect, which may also create a more aggressive micro-climate (NSBA/AISC 2022). The width of the overpassing structure has been cited as a possible variable influencing the tunnel effect.

**Figure 8 – UWS Overpass Bridges over Interstate and Major State Route**

## 1.3 General Bridge Maintenance Requirements

While proper maintenance is important for all bridges, the unique characteristics of UWS bridges create the need for special and timely attention to bridge maintenance actions. Delays in providing routine and condition-based maintenance actions can lead to extensive, difficult, and costly repairs and/or a significant reduction in the lifespan of the structure.

# Chapter 2 - UWS Bridge Safety Inspection Procedures

## 2.1 General

In addition to bridge safety inspection procedures that are common and essential to all bridges, uncoated weathering steel bridges require special inspection knowledge and special attention to key areas of the steel and other conditions to ensure the bridge can provide the expected life-span without the need for unexpected and extensive repair and maintenance actions.

The following items are considered as special focus areas for UWS bridges, each of which is specifically covered in this chapter:

Inspection

- Oxide Film (Patina)
  - Adhesion
  - Texture
  - Color
- Crack Detection
- Corrosion and Section Loss
- UWS Paint Condition
- Other Focus Areas
  - Water/Debris Traps
  - Leaking Deck Expansion Joints, Decks, and Deck Drainage Systems
  - Vegetative Growth and Waterway Debris Build-Up
  - Local Environment – Salt-laden Moisture Exposure and Tunnel Effect
  - Dissimilar Materials
  - Crevices

Testing

- Patina Tape Adhesion Test
- Chloride Contamination Testing
- Ultrasonic Testing
- Crack Testing

Condition and Condition State Ratings

- Condition Rating, Oxide Film (Patina)
- Condition State Rating, Steel Protective Coatings
- Condition Rating, UWS Paint
- Condition State Rating, UWS Paint

## 2.2 Inspection Procedures

### 2.2.1 Visual/Physical Examinations

#### 2.2.1.1 Patina

The patina must be examined for three characteristics – adherence, texture, and color. These examinations are to be used to determine if the patina is protective or non-protective, and for use in establishing patina condition ratings.

Examinations must be performed up-close at arm's length to the steel to establish the micro aspects of the steel surface. This need is illustrated by Figure 9 which shows the contrast in appearance when a UWS superstructure is viewed from ground level compared to a similar elevation as the superstructure. While from a distance the patina appears adequate, from a closer distance there appears to be large flakes indicating the initiation of a non-protective patina.

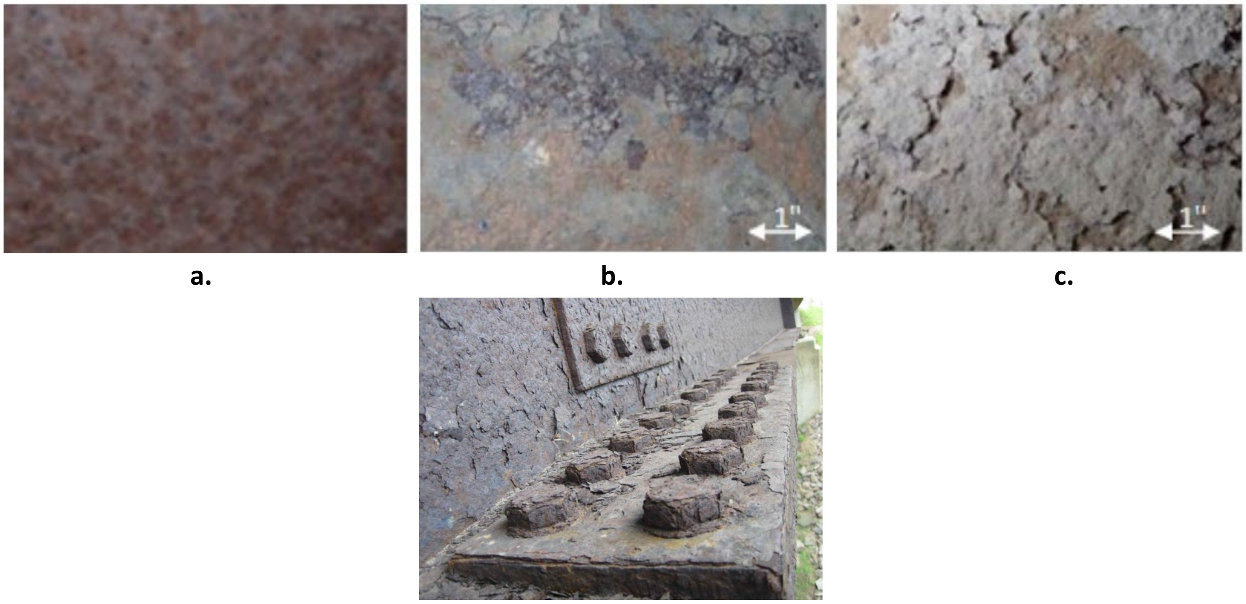
a. b.

**Figure 9 – Views of Patina Up Close and from a Distance Showing Different Results (Source: NSBA/AISC 2022)**

- Adherence

The protective layer should be tightly adhering. This metric is most reliable for determining UWS performance and can be confirmed by no change in surface condition occurring due to pounding with a rubber mallet or the inability of the patina to be rubbed off or pried loose by hand tools (e.g., putty knife). Examples of the surface texture of UWS meeting this criterion are shown in Figure 10. In contrast, poor performing patina can be scraped loose with a putty knife, crushed under light impact and/or pried loose with a knife or fingernail as shown in Figure 11.  There are two exceptions to adherence being indicative of performance. One is that in the first few years of service or in very benign environments where patina development occurs slowly, fine (< 1/32") particles that are easily removed from the surface are not of concern. The second is that if the mill scale was not removed at the time of fabrication/construction, the mill scale will likely be easily removed from the surface. This condition should be noted in the initial bridge safety inspection. For Pennsylvania bridges, generally only the fascia side of exterior beams or girders are blast cleaned as specified in Publication 408, Specifications.


**Figure 10 – Views of good-performing UWS showing a smooth surface of varying colors (Source: NSBA/AISC 2022)**




a. b. c.

d.

**Figure 11 - Views of poor-performance of UWS showing various textures: (a.) rough “freckled surface, (b.) a relatively smooth surface that was easily crushed by tapping, and (c.) and ( d.) course surface with exfoliating rust layers (Sources: NSBA/AISC 2022, a., b., c.; NYSDOT, d.)**

The quality of adhesion should also periodically be based on a patina tape adhesion test (2013 Crampton et. al. 2013) in accordance with the procedures in Section 2.2.2.1 Patina Adhesion Tape Test. This tape test is a physical assessment of the patina. The tape will remove samples of the oxide layer that should be compared to the rust flake size and spatial distributions. This information will provide supplemental data for establishing the condition rating of the patina coating as addressed in Section 2.3.1.

Patina adhesion tape tests shall be performed in conjunction with Routine inspections as follows:

- New UWS Bridges: First tape test within four years after opening of the bridge, then at an interval not to exceed 48-months for bridges on a 48-month inspection cycle, and 72-months for bridges on a 24-month inspection cycle.
- In-Service Bridges: First tape test by the end of calendar year 2028, then at an interval not to exceed 48-months for bridges on a 48-month inspection cycle, and 72-months for bridges on a 24-month inspection cycle.
- Perform sufficient tests to adequately represent the patina condition of overall steel members, but no less than two tests per span. Additional tape tests shall be performed at specific locations where there is an indication that the patina is non-protective.
- For overpass bridges with less than 25 feet of vertical clearance and/or 20 feet of lateral clearance (edge of lane to face of abutment or pier) perform the tape tests on the uncoated weathering steel within the region of the lanes or shoulder areas of each applicable span to provide sufficient splash zone exposure changes over time. See Figure 12 for a graphic of tape test locations.

In order to document the schedule and other inspection information for patina adhesion tape test, an In-Depth inspection type should be added to the inspection record. The In-Depth inspection interval and next date should be adjusted to schedule the next tape test. See Figure 13 for a screenshot of a sample BMS3 Schedule Screen.

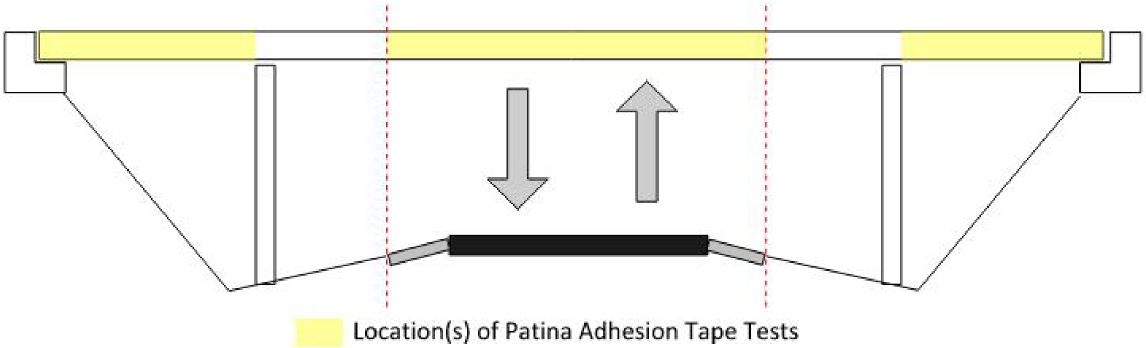
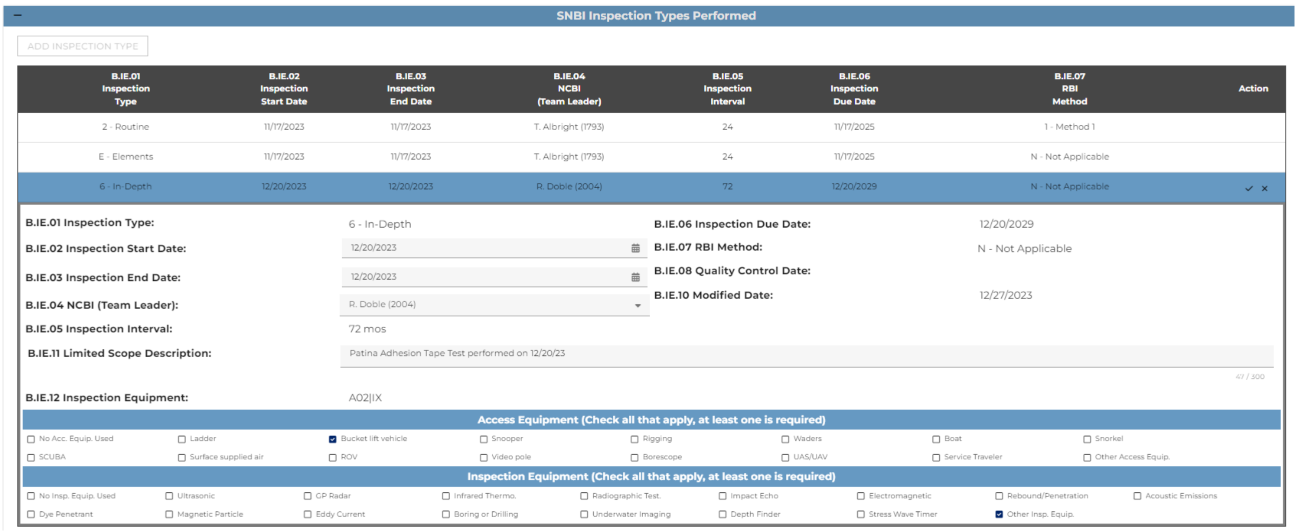



**Figure 12 – Locations of Patina Tape Tests for Overpass Bridges**



SNBI Inspection Types Performed

ADD INSPECTION TYPE

| B.IE.01 Inspection Type | B.IE.02 Inspection Start Date | B.IE.03 Inspection End Date | B.IE.04 NCBI (Team Leader) | B.IE.05 Inspection Interval | B.IE.06 Inspection Due Date | B.IE.07 RBI Method | Action |
|---|---|---|---|---|---|---|---|
| 2 - Routine | 11/17/2023 | 11/17/2023 | T. Albright (1793) | 24 | 11/17/2025 | 1 - Method 1 | |
| E - Elements | 11/17/2023 | 11/17/2023 | T. Albright (1793) | 24 | 11/17/2025 | N - Not Applicable | |
| 6 - In-Depth | 12/20/2023 | 12/20/2023 | R. Doble (2004) | 72 | 12/20/2029 | N - Not Applicable | ✓ ✕ |

B.IE.01 Inspection Type: 6 - In-Depth
B.IE.02 Inspection Start Date: 12/20/2023
B.IE.03 Inspection End Date: 12/20/2023
B.IE.04 NCBI (Team Leader): R. Doble (2004)
B.IE.05 Inspection Interval: 72 mos
B.IE.06 Inspection Due Date: 12/20/2029
B.IE.07 RBI Method: N - Not Applicable
B.IE.08 Quality Control Date:
B.IE.10 Modified Date: 12/27/2023
B.IE.11 Limited Scope Description: Patina Adhesion Tape Test performed on 12/20/23
47 / 300
B.IE.12 Inspection Equipment: A02|IX

Access Equipment (Check all that apply, at least one is required)

☐ No Acc. Equip. Used ☐ Ladder ☑ Bucket lift vehicle ☐ Snooper ☐ Rigging ☐ Waders ☐ Boat ☐ Snorkel
☐ SCUBA ☐ Surface supplied air ☐ ROV ☐ Video pole ☐ Borescope ☐ UAS/UAV ☐ Service Traveler ☐ Other Access Equip.

Inspection Equipment (Check all that apply, at least one is required)

☐ No Insp. Equip. Used ☐ Ultrasonic ☐ GP Radar ☐ Infrared Thermo. ☐ Radiographic Test. ☐ Impact Echo ☐ Electromagnetic ☐ Rebound/Penetration ☐ Acoustic Emissions
☐ Dye Penetrant ☐ Magnetic Particle ☐ Eddy Current ☐ Boring or Drilling ☐ Underwater Imaging ☐ Depth Finder ☐ Stress Wave Timer ☑ Other Insp. Equip.

**Figure 13 – Sample BMS3 Schedule Screen, In-Depth Inspection for Patina Adhesion Tape Test**

- Texture

A properly formed patina has tight mill scale or a tight granular consistency which will not be adversely affected by vigorous brushing with a wire brush. An improperly formed patina will generally have flakes and/or delaminations which can be removed with a hammer tap, a wire brush or chipping hammer.

Particle sizes of ⅛” or less are not of concern. Such particles sizes result in relatively smooth surfaces, manifesting in many different appearances as shown at close range in Figure 10. Granular rust flakes exceeding ¼” diameter are indications of a non-protective patina as shown in Figure 11 a. Sheet-like layers of rust are clear indications of a non-protective patina as shown in Figures 11 b., c., and d.

Table 1 represents a correlation between texture of UWS and the degree of protection.

**Table 1 - Correlation Between Weathering Steel Texture and Condition (Source: FHWA BIRM 2022)**

| Appearance | Degree of Protection |
|---|---|
| Tightly adhered, capable of withstanding hammering or vigorous wire brushing | Protective oxide |
| Dusty | Early stages of exposure; should change after a few years |
| Granular | Possible indication of problem, depending on length of exposure and location of member |
| Small flakes, ¼ inch in diameter | Initial indication of non-protective oxide |
| Large flakes, ½ inch in diameter or greater | Non-protective oxide |
| Laminar sheets or nodules | Non-protective oxide, severe condition |

- Color

The color of the surface of UWS is an indicator of the protective patina. The color changes as the patina matures to a fully protective coating.

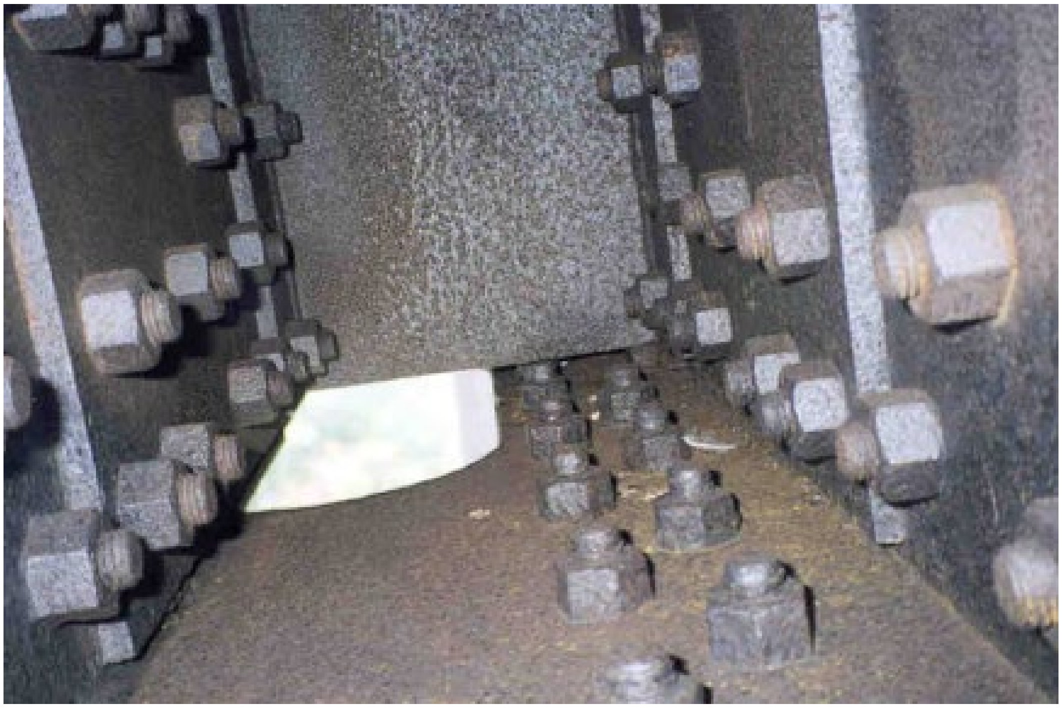
In general, in good performing UWS, the color of new UWS begins as orangish brown after the initial stage of exposure, then becomes reddish brown and finally dark brown (often with a purplish hue). The specifics of these colors and the time scale over which these changes occur vary significantly in different environments. Non-protective oxides generally appear dull gray to black in typical U.S. environments. The dull gray color can often be found on poorly performing horizontal face-up surfaces where debris collects and mixes with the patina. See Figure 14.

Vertically-oriented variations in color are usually indicative of condensation patterns as depicted in Figure 15.

**Yellow Orange – Early Development of Patina**

**Light Brown – Early Development of Patina**

**Chocolate Brown to Purple Brown – Fully Developed Patina**



**Black – Non-Protective Patina**

**Figure 14 - Views of Colors associated with Patina Development Stages (Source: FHWA BIRM 2022)**

**Figure 15 – View of Good Patina with Vertical Variations in Color Due to Condensation (Source: NSBA/AISC 2022)**

#### 2.2.1.2 Crack Detection

While fatigue cracking is not expected in steel bridges designed in accordance with modern fatigue provisions, such cracking may occur and be present in bridges designed prior to the implementation of these provisions.

Cracks due to fatigue or other sources may be more difficult to discover in a UWS structure compared to a painted structure unless the crack is active and bright visible staining is evident from fretting corrosion or moisture in the crack. This is because in a coated structure, a fatigue crack typically causes a defect in the paint, which in turn causes localized rusting that has a visually obvious color contrast to the adjacent paint.

As with all steel bridges, early detection is important to the effective management of cracks in steel members.

Refer to Section 2.2.2.3 *Crack Testing* for crack testing procedures.

#### 2.2.1.3 Corrosion and Section Loss

The identification of corrosion and determination of section loss is vital to the safety inspection process of all bridges, but both tasks can be more difficult at UWS bridges due to the presence of the patina.

When the appearance (color and texture) indicate that the patina has become non-protective, the condition of the underlying steel should be assessed to determine the need to check for section loss.

When checking for section loss the steel should be properly cleaned and measurements should be established using a digital caliper or an ultrasonic thickness gauge (also known as a D-meter). See Figures 16 and 17.

Appropriate and good cleaning practices are well-defined in applicable bridge safety inspection manuals including, but not limited, to the AASHTO MBE Section 4.3.5.2 and the FHWA BIRM.

Inspectors should not be concerned with removing the patina during the cleaning process. A new patina is expected to form on the cleaned steel surfaces in the same manner that it forms after the initial construction.[1]

Refer to Section 2.2.2.4 *Ultrasonic Testing* for base metal preparation and ultrasonic testing procedures.

When section loss has been determined to be present, the need for a new load rating must be considered in accordance with Publication 100A, Section 2.5.6 *Load Rating* and Publication 238, IP Section 3.6 *Live Load Capacity Rating Methods*. Additionally, the need for maintenance actions to repair and/or stem the advancement of section loss must be considered.

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[1] A review of UWS technical documents did not reveal any study results or a formal position related patina reformation after cleaning, nor did the technical documents cite any concerns with cleaning. One reference (Ault and Dolph 2018) cited a state agency's experience of reestablished patina on weathering steel structures through a 3,500-psi hot water or 5,000-psi cold water wash. The presence of detrimental environmental and other local conditions would be expected to affect or inhibit the patina formation process.


**Figure 16 – View of Thickness Measurement with a Digital Caliper**


**Figure** 17 **– Views of Cleaned Surface and Ultrasonic Testing Gauge Measurement**

### 2.2.1.4 UWS Paint Condition

Where UWS is painted, typically at beam/girder ends and other select locations, the paint must be inspected using the same procedures as non-UWS bridges. Paint inspection is important since the localized paint areas provide protection to the uncoated weathering steel where corrosion is known to initiate, rapidly advance, and may be difficult and costly to repair.

BMS does not provide a field for reporting the component condition of localized paint areas. Therefore localized paint conditions should be recorded in the inspection notes and in the element level data in BMS. Associated paint maintenance needs should be established in the Inspection – Maintenance Section in BMS.

#### 2.2.1.5 Other Areas of Focus

Six focus areas are established to determine if certain conditions are present at the bridge. The six focus areas include:

- Water/Debris Traps
- Leaking Deck Expansion Joints, Decks, Drainage Systems, and Utility Conduits
- Vegetative Growth and Waterway Debris Build-up
- Local Environment – Salt-laden Moisture Exposure and Tunnel Effect
- Dissimilar Metals
- Crevices.

Bridge inspectors shall determine and record if the patina and weathering steel are adversely affected by these conditions when present.

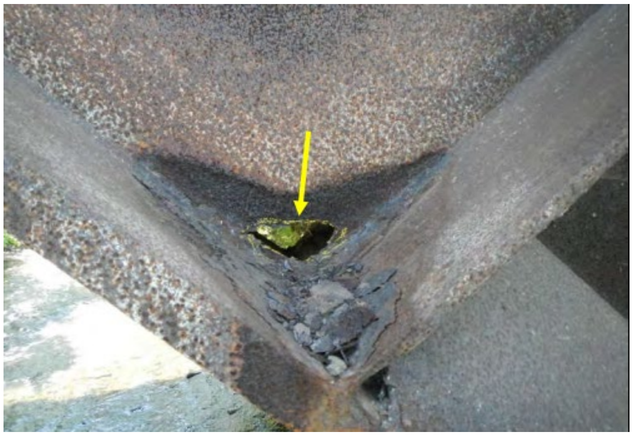
As with the safety inspection of all bridges, regardless of the material type, debris and other deposits must be removed to facilitate the proper inspection.

- Water/Debris Traps

The identification, inspection, and maintenance of water and debris traps is especially important for UWS since traps have a well-documented history of creating an adverse environment that leads to and accelerates steel corrosion and section loss. Many changes to design details in response to the 1989 FHWA Technical Advisory have served to eliminate these trap conditions, but many past designs contain a multitude of locations where rust and debris can collect and create small reservoirs or wet areas. These conditions are especially prevalent at the bottom of K-frame columns, the bottom of transverse stiffeners where corner clips (chamfers) were not provided, and at the top side bottom splice plates of the exterior face of fascia beams. See Figures 18 and 19 for photographs of example conditions, and Figure 20 for current-standard corner chamfer details.

Effective maintenance recommendations to address these conditions can include power washing and/or spot painting. In most cases a Maintenance Priority 2 should be established for cleaning water and debris traps in accordance with the maintenance priority examples listed in Field IM05 – Maintenance Priority of Publication 100A. Recommendations for spot painting, retrofits to eliminate the traps, or full painting of the steel member may be appropriate for some conditions and bridges. A recommendation for the installation of drip bars on the bottom flanges should be considered where these devices could prevent water/debris from entering trap conditions and prevent substructure staining. See BC-753M for current-standard drip bar details.

Debris Build-Up at Bottom of Frame Leg



Debris Build-Up and Full Depth Web Corrosion at Frame Leg, Intersection of Flange/Web/Transverse Stiffener

Debris Build-Up at Bottom of Frame Leg

**Figure 18 – Problematic Water and Debris Traps**

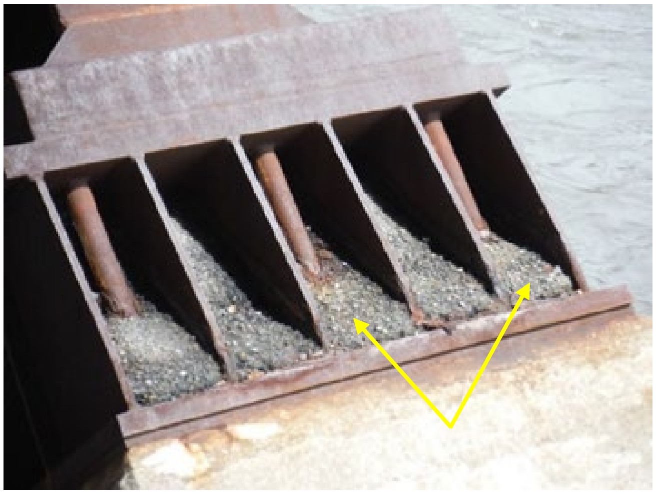


**Figure 19 – Bearing Anchorage of UWS Frame Bridge: Views of Ineffective Asphalt Filling and Effective Watertight Steel Panel Retrofit**

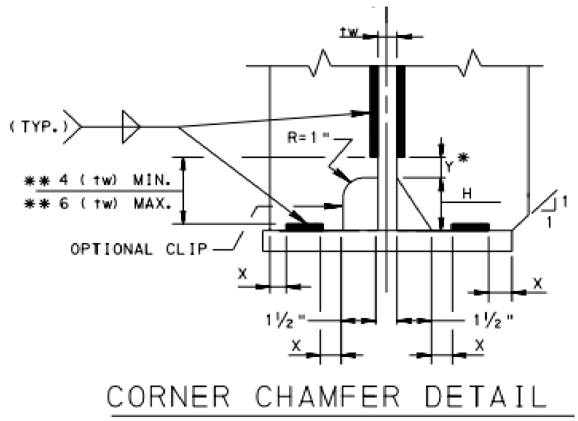



**Figure 20 – Corner Chamfer Detail from BC-753M**

- **Leaking Deck Expansion Joints, Decks, Drainage Systems, and Utility Conduits**

The inspection of deck expansion joints, underside cracking of concrete decks, and drainage systems is especially important for UWS bridges since the leakage of chloride contaminated drainage onto steel member surfaces can quickly initiate the conversion of a protective patina to a non-protective patina. See Figures 21 and 22.

Leaking utility conduits can create a wetting environment that can also impact the protective patina.

When leaking conditions are identified, it is especially important for inspectors to recommend maintenance repairs to eliminate leakages onto uncoated weathering steel. A Maintenance Priority 1 or 2 should be established for leakages onto weathering steel in accordance with the maintenance priority examples listed in Field IM05 – Maintenance Priority of Publication 100A.

**Figure 21 – Advanced Corrosion and Section Loss Due to Leaking Drainage Systems or Leaking Deck Joints**

**Figure 22 – Top Flange Deterioration Beneath Concrete Deck Crack (Source: Crampton et al., 2013)**

- Vegetative Growth and Waterway Debris Build-Up

The presence of vegetative growth on or near UWS surfaces and waterway debris build-up can create conditions that prevent the natural drying of steel member surfaces, which is a condition that can convert a protective patina to non-protective. See Figure 23.

Vegetative growth and waterway debris build-up on or near UWS should therefore be identified during bridge safety inspections, and maintenance recommendations should be established for prompt removal. In most cases a Maintenance Priority 2 should be established for removal of vegetative growth and waterway debris build-up in accordance with the maintenance priority examples listed in Field IM05 – Maintenance Priority of Publication 100A.

**Figure 23 - Vegetative Growth and Waterway Debris Build-Up at UWS Bridges**

- **Local Environment - Salt-laden Moisture Exposure and Tunnel Effect**

Bridges passing over roadways that are heavily treated with deicing agents for winter roadway maintenance can be susceptible to a severe local environment. This is because airborne salt-laden road spray from the underlying roadway can collect on the superstructure.

The combination of small vertical and horizontal clearances combined with variation in roadway elevation can create what is known as a “tunnel effect,” which may also create an aggressive micro-climate. The width of the overpassing structure has also been cited as a possible variable influencing the tunnel effect. Bridges with small vertical clearances (less than 25 feet) and/or small horizontal clearances (less than 20 feet from the edge of lane to the face of substructure unit, e.g., abutment or pier) may be most vulnerable.

While design changes for UWS bridges as implemented in 1989 have been beneficial to UWS performance, inspectors must carefully inspect all overpass bridges for possible impacts due to the “tunnel effect.” Such impacts should be recorded in the inspection notes in BMS.

Dual multi-span interstate overpass bridges built in Pennsylvania in the early 2000’s were found to show signs of localized non-protective patina and corrosion at the exterior girder bottom splice location. It is thought that this condition may be influenced by salt-laden spray. See Figure 24.

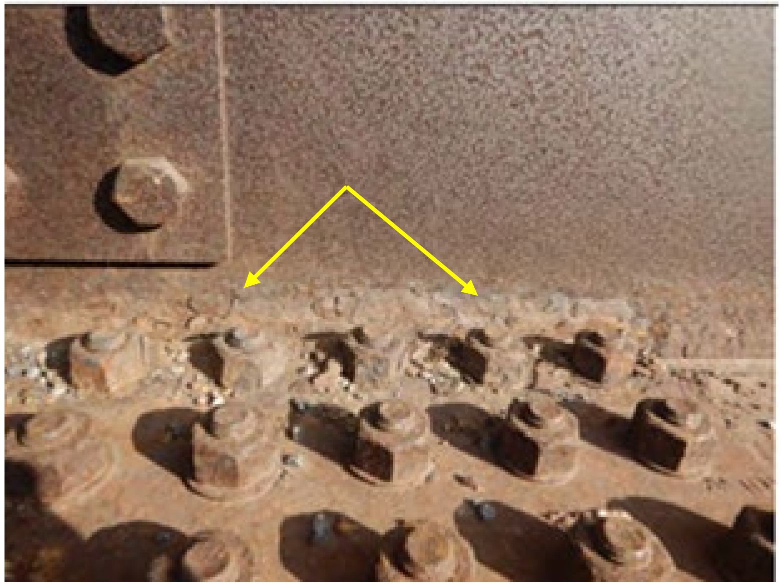


**Figure 24 – Exterior Girder Web Corrosion at Top Plate of Bottom Splice**

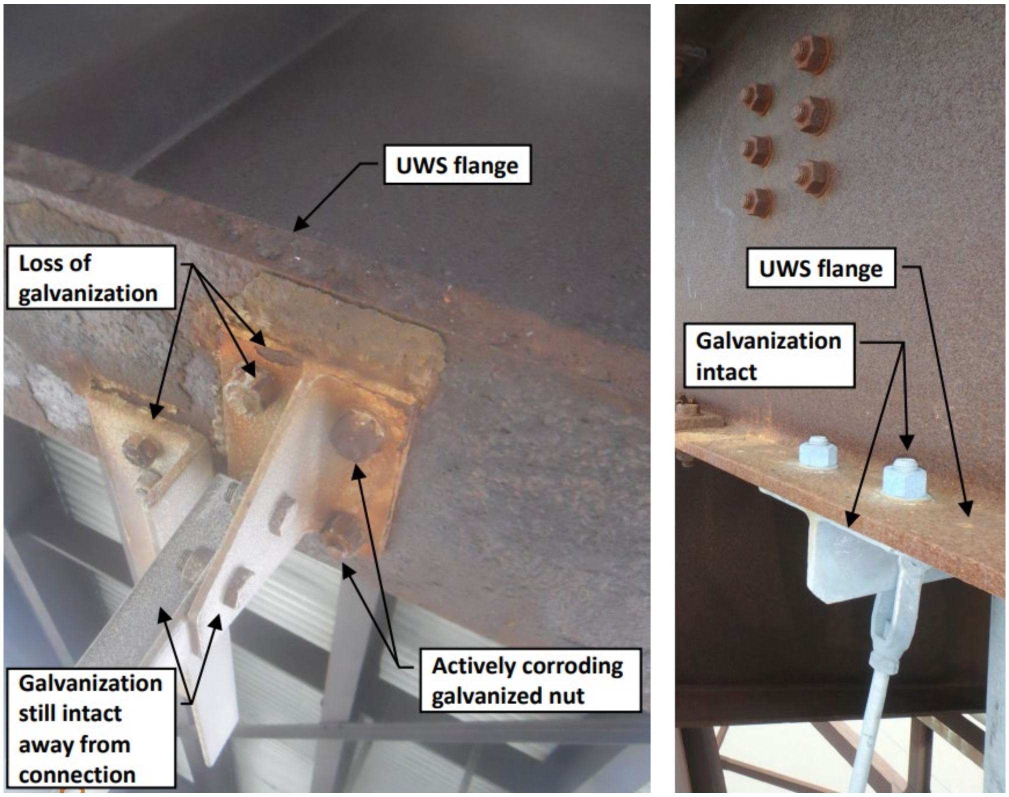
Recommendations for appropriate maintenance actions for the tunnel effect conditions should be made and reported in BMS. For example, the corrosion as shown in Figure 24 may be remediated with blast cleaning and painting the bottom splice region of the girder. Such action in the early stages of corrosion can prevent more complex and costly repairs in the future.

- Dissimilar Metals

Galvanic, or bimetallic corrosion, can occur between metals with electrical differences that are in contact and exposed to an electrolyte source. In the case of galvanized supports attached to uncoated weathering steel, moisture can act as the electrolyte and corrosion can be of concern if the contact surfaces frequently become wet and an insulation material was not used between the two metals (UWS and galvanized metal).

Examples of dissimilar metal contact in attachments to UWS are provided in Figure 25. Both photos in the figure show the same metals in contact, UWS and galvanized steel, but with different corrosion performance. The connection in Figure 25 a. is experiencing galvanic corrosion, while the connection in Figure 25 b. shows no sign of galvanic corrosion. This most likely can be attributed to the lack of moisture exposure at the non-corroded connection.

According to recent information from FHWA, there is not a high likelihood of galvanic corrosion where the patina is protective (sometimes referred to as stable patina).




a. b.

**Figure 25 - Example of Corrosion at Dissimilar Metal Appurtenance Attachment to UWS (Source: NSBA/AISC 2022)**

The use of conventional mild steel fasteners with UWS can result in corrosion of the bolt/nut. Figure 26 shows a corroded nut likely inadvertently used during construction. Similar conditions can occur at bearings where dissimilar metals are used. In these cases galvanized anchor bolt assemblies (anchor rod, nut, and washers) that come in contact with UWS may incur corrosion where there is a potential for moisture.

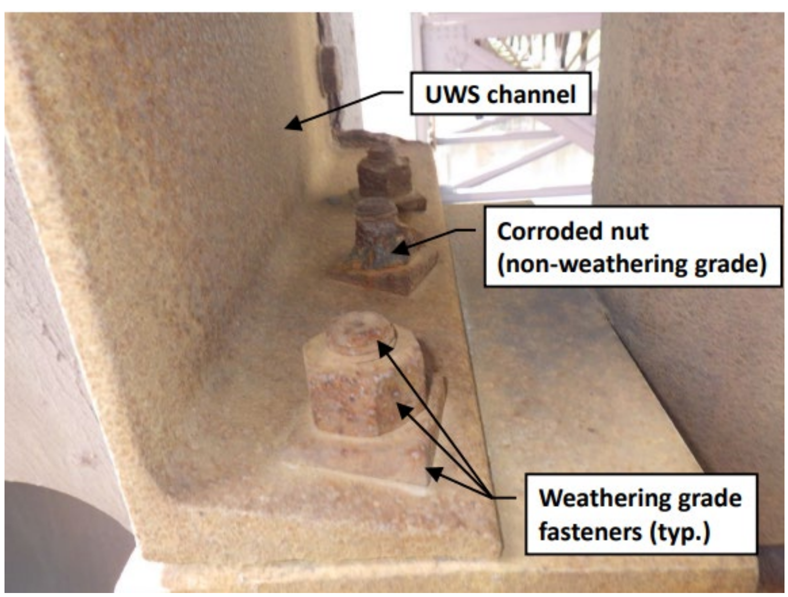



**Figure 26 - Advanced Corrosion of Non-Weathering Steel Fastener (Source: NSBA/AISC 2022)**

- Crevices

Crevice corrosion can be a deterioration problem on steel bridges, including UWS bridges, due to the tendency to trap debris and hold moisture (e.g., increased time of wetness) in these areas. This is more common in older existing bridges that utilized rivets or were designed during a time with more relaxed bolt spacing requirements. Bridges designed to modern sealing requirements generally do not suffer from this form of deterioration.

Where crevice corrosion has occurred, the sealing of crevices can be a viable maintenance solution depending on the connection type and extent of corrosion. See Figure 27. The need for crevice sealing should be reported as a needed maintenance action in the Inspection – Maintenance Section in BMS.

**Figure 27 - View of Sealed Crevice at Bolted Cover Plate**

### 2.2.2 Testing Procedures

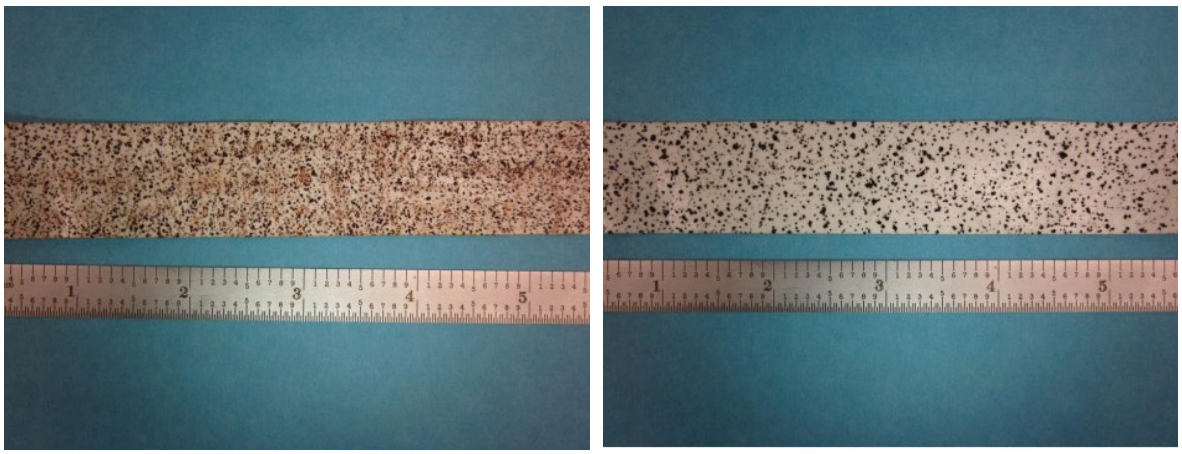
#### 2.2.2.1 Patina Adhesion Tape Test

Patina adhesion tape test locations and frequencies are in accordance with Section 2.2.1.1.

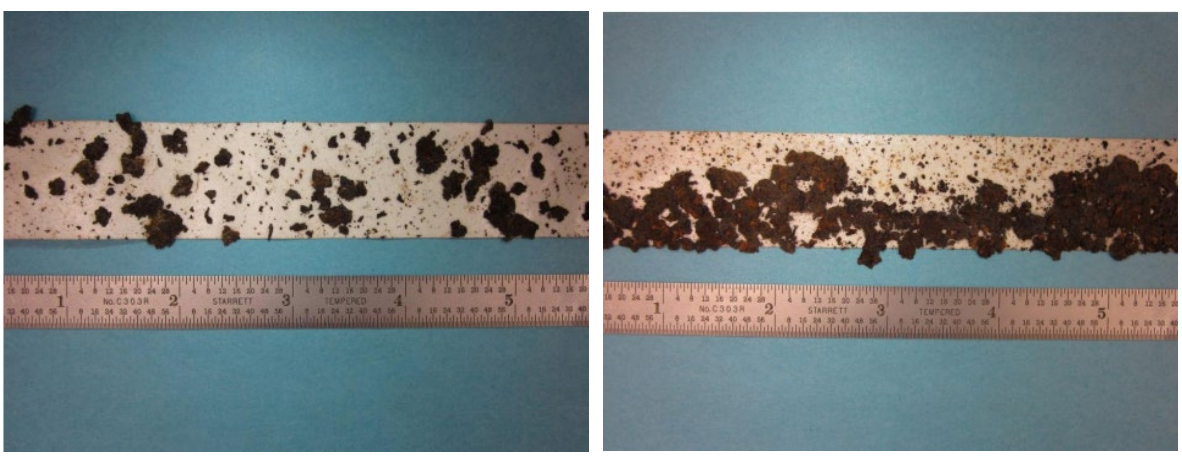
Use the following procedures for the patina adhesion tape test.

- The tape should be 1” wide white cross-hatch tape manufactured by the SEMicro Division of M.E. Taylor Engineering, Inc., an approved equal, or as provided by the Bureau of Bridge. Remove two wraps of tape before cutting the 8 to 12 in. long strip used for the test.
- Use a strip of tape 8 to 12 inches long and press firmly onto the patina surface.
- After waiting for a period of at least one minute, peel the tape off surface quickly and smoothly at a steep angle between 90 and 150 degrees from the patina surface. The tape will remove samples of the oxide layer.
- The sample material stuck on the tape should be compared to the rust flake size and spatial distribution for use in establishing the Patina Condition Rating. See Figure 28 for sample photographs of tape test results with Protective and Non-Protective Patina, and Section 2.3.1 for the Patina Condition Rating procedure.

- Documentation
    - Provide one photograph of each patina adhesion tape test.
    - Provide one photograph showing a close-up view of the UWS member patina at each patina adhesion tape test location.
    - Provide a simple graphic showing the specific locations of all tape tests.
    - Label the photographs at each tape test location with the BMS ID or BRKEY, date taken, tape test location, and patina condition rating assigned to the sample. See Figure 29 for the format.
- Tape tests for each test location shall be stored in a sealable clear plastic bag, labeled in accordance with the information described above for photographs.
- Image Analysis of Tape Tests
    - Rust particle sizes and corresponding particle size distributions from patina tape tests can be established with scanners and image processing software or other means. The need for this advanced method of patina assessment should be coordinated with the Bureau of Bridge.



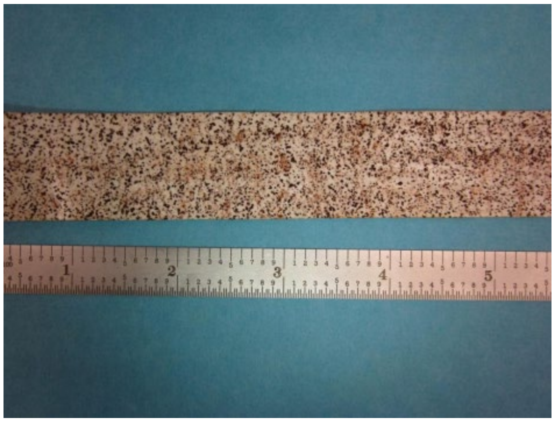
**Protective Patina**



**Non-Protective Patina**

**Figure 28 – Photos of Patina Adhesion Tape Test Results for Protective to Non-Protective Patina (Source: Crampton et al., 2013)**

**Close-Up View of Patina**



**Patina Adhesion Tape Test**

**BRKEY: 99999 | Date Taken: 12-26-23**
**Location: Span 2, Girder 1, Right Face of Web,**
**20 feet from C.L of Bearing at Pier 1**
**Patina Condition Rating: 7**

**Figure 29 – Sample Format for Patina Photographs**

An electronic document shall be created to include the patina tape test locations graphic and patina tape test photographs. The photographs shall be arranged in the document by span. Subsequent to the initial tape tests, photographs from the most recent past tape tests shall also be included and placed adjacent to the current photographs to allow for condition comparisons and assessment of any changes to the patina. Where a current tape test is a new location and has no past tape test for comparison, identify that tape test as a "New Test Location."

The electronic document shall use the following file naming format for consistency and searchability: "BRKEY xxxx_Patina Tape Test Photographs_ Calendar Year of Tape Test" or as an example, "BRKEY 9999_Patina Tape Test Photographs_2024."

The patina tape test photographs document shall be uploaded to the Document Screen in BMS using the *Coating Test Results* Document Type.

#### 2.2.2.2 Chloride Contamination Testing

Research indicates that there is a correlation between chloride contamination within a patina layer and the size of the loose rust flakes on the surface and patina deterioration can be a result of chloride exposure in areas that are not sufficiently cleaned by natural washing. Research, however, also indicates that chloride testing did not yield sufficient information for evaluating patina performance and predicting long term performance (Crampton et al. 2013, Ault and Dolph 2018). Therefore chloride contamination testing is not recommended for routine inspections and should only be performed when approved by the District and/or the Bureau of Bridge.

#### 2.2.2.3 Crack Testing

With consideration of the unique characteristics of UWS as stated in Section 2.2.1.2, the inspection for cracks should follow Publication 238 *Bridge Safety Inspection Manual*, the FHWA *Bridge Inspector's Reference Manual,* and the AASHTO *Manual for Bridge Element Inspection*. Crack detection is typically performed visually, but nondestructive techniques may be used as well (e.g., magnetic particle testing, dye penetrant testing).

#### 2.2.2.4 Ultrasonic Testing

Perform ultrasonic testing as discussed in Section 2.2.1.3.

Use the following procedure for ultrasonic testing to determine remaining steel sections in UWS.

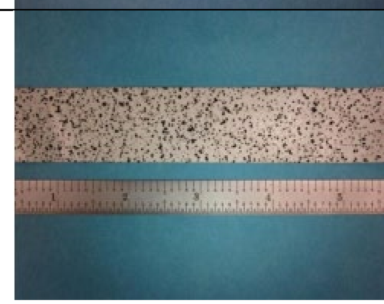
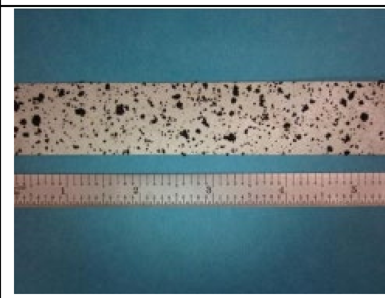
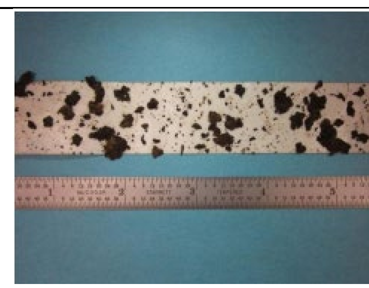
1. Ensure the ultrasonic thickness testing device is properly calibrated.
2. With a handheld wire brush or mechanical wire brush fitted onto a drill or similar device, remove the oxide (patina) off a small area on one side of the plate until the bare metal is exposed. Bare metal should be exposed only on the highest points of the corroded surface, leaving any depressions filled with oxide and ensuring the probe makes good contact with the steel surface. Approximately 30 percent of the ground surface should have a metallic (e.g., shiny) appearance. The prepared surface area needs only to be as large as the probe of the measurement device, which is typically circular with a diameter less than 1 inch. Note that an inspection hammer or rotary percussion hammer may be used in advance of wire brush operations as appropriate.
3. Apply a liquid biodegradable coupling agent to the prepared surface area.
4. Place the ultrasonic testing probe flush on the steel surface and move the probe of the ultrasonic thickness gauge around the ground area and retain the smallest reading. This peak-to-valley reading is a reasonable estimate of the remaining plate thickness.
5. Take multiple readings to account for sensitivity of the probe and record the minimum reasonable thickness.
6. Subtract the measured plate thickness from the original plate thickness specified on the as-built drawings to determine the section loss.

## 2.3 Condition and Condition State Ratings

### 2.3.1 Condition Rating, Oxide Film (Patina)

The oxide film condition rating, as recorded in BMS Field 6B36 – Protective Coating Condition Rating, is to typically represent the average condition of all uncoated weathering steel members based on visual observations and, when performed, the patina tape adhesion test results. Where section loss has occurred, the overall rating should be representative of the worst case patina location.

Refer to Figure 30 for establishing the Patina Condition Rating which ranges from 7 (Good) to 2 (Critical). Note that the condition rating shall consider the patina adhesion tape tests when tape tests are performed. Also note that this condition rating scale is different from the 9-0 scale for coatings (most commonly paint) for Non-UWS.

| Patina Rating | Condition Description | Example Condition in Field | Example Patina Adhesion Tape Test Specimen |
|---|---|---|---|
| **7 Good** | - Uniform color pattern, generally dark brown with some lighter reddish-brown, metallic, and purple-brown spots. May be difficult to see small rust product clusters.<br>- Texture may be dimpled or rough but uniform in pattern. Patina layer is thin but dense and very adherent, indicative of very good protective properties. Superior adherence; tape test sparse with only very small flakes (< 1 mm). |  |  |
| **6 Satisfactory** | - Uniform color pattern, generally dark brown with some lighter reddish-brown, metallic, and purple-brown spots. Individual rust product clusters visible.<br>- Texture is dimpled or rough but uniform in pattern. Patina layer is thin but dense and adherent, indicative of good protective properties. Tape test easily removes very small (< 1 mm) flakes. |  |  |
| **5 Fair** | - Dark brown coloration, but begins to show minor variation. Flakes up to ¼" loose on surface, easily removed with tape test.<br>- Underlying layer adherent, still relatively dense, thin, and protective. Texture more granular and loose flakes may be less protective, holding water and salts.<br>- Chalky poultice layer* may be present, but not significantly affecting performance (e.g., flake size). |  |  |
| **4 Poor** | - Dark brown with black and some color variation. Blotchy with some salty or rusty stains. Medium (¼" to ½") flakes over most of area loose and non-protective, easily removed with tape test.<br>- Layer beneath flakes thicker and more permeable, with some pitting beginning. Non-protective; contaminants penetrating.<br>- Elements with poultice may show significant associated flaking. |  |  |
| **3 Serious** | - Color is dark brown and black but non-uniform, with widespread blotchiness and staining. Non-protective.<br>- Large (> ½") flakes, or layered delamination beginning in some areas. Thickness/permeability of rust increased, with pitting and section loss possible.<br>- Poultice* areas have thin delamination sheets or very large flakes. Layer below loose poultice may appear similar, but still somewhat adherent. |  |  |

| Patina Rating | Condition Description | Example Condition in Field | Example Patina Adhesion Tape Test Specimen |
|---|---|---|---|
| **2**<br>**Critical** | - Blackish, stained, blotchy appearance.<br>- Formation of laminar sheets with deeply pitted semi-adherent layer beneath; chunks and sheets of rust product removable by hand.<br>- Aggressive advancement of pitting and section loss; can be up to 50%. Complete failure of patina to protect base steel. |  |  |

* Poultice layer is defined as a layer with dust, dirt, rust flakes, bird droppings, and/or other debris that can hold water and may be contaminated with chlorides. Most commonly associated with the top surface of horizontal elements such as bottom flanges or bracing members.

**Figure 30 – Uncoated Weathering Steel - Patina Condition Rating Scale (Source: Crampton et al., 2013, modified by PennDOT)**

### 2.3.2 Condition State Rating, Steel Protective Coatings (Element 515)

The condition state rating and associated quantities for the patina of uncoated weathering steel should be established in Steel Protective Coatings (Element 515), Defect 3430 Oxide Film Degradation and Defect 3440 Effectiveness, and recorded in BMS Fields 1A11 – Condition State Quantities and 1A12 – Element Condition Notes for the portions of each applicable steel element that are not locally painted. The patina condition state and associated quantities should be based on visual observations, and as supplemented by information from the patina tape adhesion test results. See Figure 31 for example condition state ratings for Defect 3430.

| Condition State 1 | Condition State 2 | Condition State 3 | Condition State 4 |
|---|---|---|---|
| Color is yellow-orange or light brown for early development. Chocolate-brown to purple-brown for fully developed.<br><br>Tightly adhered, capable of withstanding hammering or vigorous wire brushing. Smooth particles < 1/8”. | Color is dark brown with some variation.<br><br>Granular texture. Some small flakes up to ¼”. | Color is dark brown and black and some variation.<br><br>Small flakes, less than ½ in. diameter.<br><br>Section loss | Color is dark brown and black but may not be uniform.<br><br>Large flakes ½ in. diameter or greater, or laminar sheets of nodules.<br><br>Section loss |
| <br>**Weathering steel patina is uniform and tightly adhered to steel beam** | <br>**Weathering steel patina is slightly uneven – surface is granular and dusty** | <br>**Weathering steel patina has flaking (less than ½” diameter) along bottom flange** | <br>**Weathering steel patina has failed – large areas of surface layer flaking off** |

**Figure 31 – Example Condition State Ratings for Steel Protective Coatings (Element 515) - Defect 3430 Oxide Film Degradation (Source: Photos - MnDOT 2023)**

### 2.3.3 UWS Paint Condition Rating

As addressed in Section 2.2.1.4, BMS does not provide a field for reporting the component condition for localized paint areas, such as beam or girder ends.

### 2.3.4 UWS Paint Condition State Ratings

The condition state ratings and associated quantities for the localized paint areas, such as girder ends, should be established in Steel Protective Coatings (Element 515) Defect 3410 Chalking, Defect 3420 Peeling/Bubbling/Cracking, and Defect 3440 Effectiveness, as applicable, and should be recorded in BMS Fields 1A11 and 1A12 for the respective painted portion(s) of each applicable steel element.

## 2.4 Unsuitable and Other Reportable UWS Conditions Form

In a future release of BMS, adverse conditions and other reportable conditions specifically related to UWS will be coded via new BMS fields and extracted from current BMS fields and reported in new UWS Sections within the Superstructure and Substructure Screens. These UWS Sections will assist Districts and bridge owners with understanding the conditions and other important aspects of UWS for their respective bridges within BMS.

Until these changes to BMS are completed, adverse and reportable condition information shall be recorded on *Form A: Unsuitable and Other Reportable UWS Conditions* as provided in Appendix B. A fillable PDF version of this form is available on the BMS home screen under the PennDOT Bridge Inspection Forms and Templates link. Form A must be completed during routine inspections.

Completed forms must be uploaded to the Document Screen in BMS using the *Uncoated Weathering Steel Documents* Document Type. For consistency and searchability, the electronic document file name of Form A shall include the BRKEY and calendar year the form was completed; Example: "BRKEY 99999_Form A_ Unsuitable and Other Reportable UWS Conditions Form_2024."

## 2.5 Maintenance Action Recommendations

### 2.5.1 General

Proposed maintenance actions should be established in the Inspection – Maintenance Section in BMS.

### 2.5.2 Washing and Cleaning

Washing and cleaning of bridge components is an essential practice of bridge management and preservation. Generally, the methods involve two categories: dry methods (i.e., cleaning) and wet methods (i.e., washing).

Cleaning/washing is expected to have many positive effects including: visibly reducing the indicators of corrosion, reducing chloride levels, improving long-term performance, and/or reducing life-cycle cost.

Highway overpasses over the most heavily salted roadways, and bridges such as K-frames that were constructed with web stiffeners within the girder/leg regions and at leg bearings where debris and other material deposits are common in their need for washing. The benefits of washing can be significant for these structures.

Bridges with leaking deck expansion joints, leaking deck drainage systems, and other focus area conditions as previously indicated in this manual are of significant concern and also merit a high consideration of the need for washing. Maintenance actions to eliminate these conditions can, in many cases, relieve the need to clean and wash these bridges.

#### 2.5.2.1 Frequency

The frequency at which cleaning and washing should occur depends on bridge conditions. Funding limitations, however, may dictate the need for prioritization. Regardless, it is expected that inspectors recommend cleaning and washing actions based on the bridge safety inspection findings.

## 2.6 UWS Field Reference Guide for Bridge Safety Inspections

An *Uncoated Weathering Steel - Field Reference Guide for Bridge Safety Inspections* has been developed to assist bridge safety inspectors with safety inspections. This three-page document provides summary information specifically related to UWS bridges. See Appendix A for the reference guide.

# Chapter 3 - UWS Bridge Maintenance

## 3.1 General

UWS bridges require ongoing care and maintenance just like all steel and concrete bridges. However, some maintenance actions for weathering steel are different than non-weathering steel. For example, weathering steel is typically not painted and relies on the naturally formed oxide coating (patina) to protect the steel members against corrosion. It is therefore important to ensure the patina remains protective. Adverse conditions that can adversely affect patina such as sustained water or moisture, chlorides, debris build-up, and vegetation must be addressed effectively and timely.

The use of maintenance plans for UWS bridges is a concept that has been suggested by the steel industry (NSBA/AISC 2022). From PennDOT's perspective, the overall purpose of a maintenance plan is to: 1) ensure the timely completion of UWS maintenance actions, and 2) provide Districts and bridge owners the opportunity to plan and program preventative maintenance actions that can mitigate or prevent maintenance problems and improve UWS performance.

## 3.2 UWS Bridge Maintenance Needs

Maintenance needs are expected to be established by the inspector during each bridge safety inspection. For UWS bridges, bridge washing, spot or zone painting, repair/elimination of leaking deck expansion joints and leaking drainage systems, installation of drip bars, removal of vegetation and waterway debris, and sealing of crevices are possible action items that can be expected.

While most maintenance needs should generally be established based on conditions discovered during bridge safety inspections, some maintenance needs, considered as preventative maintenance, should be established as follows:

- Routine pressure washing of the steel framing to remove chloride contamination can be an effective method of extending the service life of uncoated weathering steel structures. However, the traffic control complexity and the resources for pressure washing can make this action impractical and cost-prohibitive as a system-wide objective. Therefore the need and frequency for pressure washing at individual bridges should be established by Districts and bridge owners. For example, cleaning for a bridge with leaking deck expansion joints and unpainted ends of girders may require annual cleaning to prevent the initiation and/or advancement of member corrosion until the deck expansion joint is repaired.
- Routine pressure washing at known problematic locations, such as water traps or low-clearance overpasses, can be an effective long-term maintenance action or performed until other actions such as member painting is performed. Districts and bridge owners should establish the need and frequency for time-interval pressure washing at specific locations for individual bridges based on need and/or performance expectations.
- Routine replacement of deck expansion joint seals, glands, and troughs at a specified time-interval can help ensure chloride-contaminated deck drainage does not penetrate onto the steel framing. Districts and bridge owners should decide on the need and use of time-interval replacement of deck joint seals/glands/troughs for individual bridges. The use of this maintenance action is expected to be very limited and established only for certain major bridges on Interstates or other critical networks.

- Other time-interval maintenance actions, as applicable, should be established based on need and performance expectations by Districts and bridge owners.

## 3.3 UWS Bridge Maintenance Plans

### 3.3.1 Standard and Enhanced UWS Bridge Maintenance Plans

Two UWS bridge maintenance plans have been developed, one of which must be established by the District for each UWS bridge. Other bridge owners are also encouraged to establish maintenance plans for their UWS bridges.

The plans consist of a Standard UWS Bridge Maintenance Plan and an Enhanced UWS Bridge Maintenance Plan. The Standard Plan encompasses maintenance actions that are needed in response to adverse bridge conditions identified during safety inspections. The Enhanced Plan encompasses preventative maintenance actions in addition to the Standard Plan actions.

<u>Standard UWS Bridge Maintenance Plan</u>

- Focuses on ensuring the following maintenance actions as recommended in the Inspection – Maintenance Section of BMS during bridge safety inspections are promptly performed:
  - repair/replace corroded weathering steel members
  - repair leaking deck expansion joints
  - repair deck drainage systems that are leaking onto UWS surfaces
  - repair conditions where utility conduits are leaking on UWS surfaces
  - repair deck cracks that allow deck drainage to seep onto UWS surfaces
  - remove vegetation in contact with UWS steel
  - remove and clean debris build-up at bearings areas and water/debris trap locations such as vertical stiffener to beam flange intersections and frame legs at bearing locations
  - wash uncoated weathering steel members
  - spot paint corroded sections as recommended
  - seal crevices
  - install drip bars

<u>Enhanced UWS Bridge Maintenance Plan</u>

- Focuses on the Standard UWS Bridge Maintenance Plan actions and the following additional time-interval, preventative maintenance items as established by Districts and bridge owners:
  - wash the entire uncoated weathering steel framing at a specified time-interval
  - wash specific uncoated weathering steel members, or locations of members, at a specified time-interval
  - replace deck expansion joint seals/glands/troughs at a specified time-interval regardless of condition
  - other time-interval maintenance actions as applicable

The District, and as applicable other bridge owners, shall establish the desired maintenance plan. The chosen maintenance plan shall be recorded on *Form B: Uncoated Weathering Steel Maintenance Plan Form* as provided in Appendix B. A fillable PDF version of this form is available on the BMS home screen under the PennDOT Bridge Inspection Forms and Templates link.

When the Enhanced UWS Maintenance Plan is chosen, time-interval, preventative maintenance item(s) shall be established and then recorded on *Form B.*

Completed forms must be uploaded to the Document Screen in BMS using the *Uncoated Weathering Steel* Document Type. For consistency and searchability, the electronic document file name of Form B shall include the BRKEY and calendar year the form was completed; Example: "BRKEY 99999_Form B_ Uncoated Weathering Steel Maintenance Plan Form_2024."

A maintenance action item for each time-interval, preventative maintenance item shall also be added to the Inspection – Maintenance Section of BMS.

*Form B* shall be updated when the District desires to modify the Maintenance Plan and/or the time-interval preventative maintenance actions.

It is anticipated that information provided on *Form B* will be integrated into a future release of BMS.

### 3.3.2 UWS Bridge Maintenance Plan Coordination

In accordance with Publication 238, Section 2.13.1.1 *UWS Bridge Maintenance Plan Coordination,* the District, and as applicable bridge owner, the UWS Subject Matter Expert (UWS-SME) is responsible for coordinating needed maintenance actions for UWS bridges. To facilitate coordination and completion of these maintenance actions, the District UWS-SME will receive a notification from BMS after a bridge safety inspection report has been accepted in BMS and when the safety inspector has reported certain maintenance needs associated with any of the following conditions in the bridge safety inspection report of a UWS bridge:

- repair/replace corroded uncoated weathering steel members
- repair leaking deck expansion joints
- repair deck drainage systems that are leaking onto UWS surfaces
- repair conditions where utility conduits are leaking on UWS surfaces
- repair deck cracks that allow deck drainage to seep onto UWS surfaces
- remove vegetation in contact with UWS steel
- remove waterway build-up in contact with UWS
- remove and clean debris build-up at bearings areas and water/debris trap locations such as vertical stiffener to beam flange intersections and frame legs at bearing locations
- wash uncoated weathering steel members
- spot paint corroded sections as recommended
- seal crevices

Maintenance actions associated with the above conditions are expected to be included on the Inspection - Maintenance Section in BMS under the following FlexActions:

- CLEAN/FLUSH
  - 1-B743101 - Scuppers and Downspouting
  - 8-C743102 - Bearing/Bearing Seat
  - 34-D743102 - Steel Horizontal Surface
- DECK
  - 6-D744303 – Concrete Deck Repair
  - 97-New PPC Overlay – Install New PPC Overlay
  - 98-New Epoxy Overlay – Install New Epoxy Overlay
- DECK JOINTS:
  - 2-A743301 - Reseal
  - 4-A744101 - Repair/Reseal
  - 33-B744102 - Compression Seal (Rep/Rehab)
  - 53-C744102 - Modular Dam (Rep/Rehab)
  - 20-D744102 - Steel Dams (Rep/Rehab)
  - 9 - E744102 - Other Types (Rep/Rehab)
- DECK DRAIN
  - 14-C744402 - Down spouting (Rep/Repl)

| | | |
|---|---|---|
| o | STEEL | 25-A744602 - Stringer (Rep/Repl) |
| | | 50-B744602 - Floorbeam (Rep/Repl) |
| | | 49-C744602 - Girder (Repair) |
| | | 54-D744602 - Diaph/Lat. Bracing (Rep/Repl) |
| | | 92-BRSHCLR - Brush Clearing |
| | | 89-E744602 – UWS Members (Repair)* |
| o | PAINTING | 57-A743201 - Superstructure - Spot |
| | | 16-B743201 - Substructure – Spot |
| | | 65-C743201 – Superstructure - Full |
| | | 79-D743201 – Substructure – Full |
| | | * Use 89-E744602 for misc. weathering steel repairs to include crevice sealing |

District UWS-SME's will also receive the BMS notices for local owned UWS bridges. At the local bridge owner's discretion, the respective District UWS-SME can assist the local owner with developing appropriate UWS maintenance actions.

## 3.4 UWS Bridge Maintenance Actions

### 3.4.1 General

Many UWS maintenance actions can be performed in accordance with standard Department maintenance Flexactions. Examples of such actions include scupper and downspouting repair/replacement and deck expansion joint repair. Other UWS maintenance actions require special requirements. These requirements are addressed in Section 3.4.2 Special UWS Requirements.

### 3.4.2 Special UWS Maintenance Action Requirements

#### 3.4.2.1 Painting

Localized UWS member regions, and in some cases the entire bridge, may be require application of an appropriate protective paint system. There are significant differences for painting uncoated weathering steel once the bridge has been in service. Some key differences include:

- Dry blast cleaning of all surfaces is necessary. Due to the rough surface and pitting, it may be difficult to economically obtain a high-quality finish and therefore the specifications should address the expected final finish.
- The paint system selected must be able to accommodate large dry film thickness variations resulting from the rough surface of the steel substrate. This is notably applicable to the primer, as a larger quantity will be required to fill the dry blasted surface profile. To achieve a smooth surface finish, this can amount to as much as four times as much primer as needed in a typical application.
- The primer system should be resistant to rust residue or chemical residue, both of which may be too difficult to entirely remove from numerous pits in the substrate surface.
- The paint system must have a low water vapor transmission rate to prevent blistering of the paint film.

Until standard special provisions for UWS painting are developed for use, Districts should coordinate UWS paint specifications with the Bureau of Bridge UWS-SME.

See Figure 32 for examples of localized painted members.

**Figure 32 - Painted Gusset Plate and Girder Splice (Source: NSBA/AISC 2022)**

#### 3.4.2.2 Sealing

Where the sealing of connections (e.g., girder splices) is to be provided, a penetrating sealer should be applied to displace moisture, all edges should be caulked with a compatible sealant (e.g., epoxy), and a stripe coat should be applied to the connection with a compatible coating. See Figure 33 for examples of sealing UWS members.

a. b.

**Figure 33 – Examples of Sealing: a. Penetrating Sealer Applied to the Interior Surface of a UWS Box Column Base; b. Caulking Applied Around the Edges of a Gusset Plate Connection. (Source: NSBA/AISC 2022)**

#### 3.4.2.3 Strengthening

Where repair of corroded sections is required and supplemental material is added, it is important to perform proper surface preparation prior to installing the new section. The needed actions may include one or more of the following:

- Cleaning (e.g., hand tool cleaning, power tool cleaning, solvent cleaning) or blast cleaning to remove lost section rust residue.
- Applying an approved rust resistant, waterproof mastic metal putty to provide a smooth interface between the damaged material and the new plating.
- Priming the contact surfaces between the damaged material and new plating.

See Figure 34 for examples of UWS repairs.

**Figure 34 - Bolted Repair to Lower Section of UWS Girders**

#### 3.4.2.4 Washing and Cleaning

##### 3.4.2.4.1 General

Bridge components should be cleaned and washed in accordance with the maintenance actions established in the bridge safety inspection report and in accordance with the UWS Maintenance Plan.

Maintenance personnel should not be concerned with removing the patina during power washing operations. It is expected that a new patina, if removed during power washing, will form on cleaned steel surfaces in the same manner that it forms after the initial construction of the bridge.[2]

##### 3.4.2.4.2 Methods and Equipment

<u>Dry Methods</u>

Dry methods include:

- Sweeping
- Compressed air blowing
- Vacuuming
- Shoveling
- Brushing, scraping, and other mechanical cleaning methods
- Vegetation removal

Typical equipment needed for dry methods includes street sweepers, brooms, air compressors, industrial vacuums, shovels, wheelbarrows, brushes, scrapers, and mowers.

<u>Wet Methods (Washing)</u>

Wet methods include:

- Flushing
- Low pressure washing (< 1000 psi)
- High pressure washing (between 1200 and 6000 psi)

[2] A review of UWS technical documents did not reveal any study results or a formal position related patina reformation after cleaning, nor did the technical documents cite any concerns with cleaning. One reference (Ault and Dolph 2018) cited a state agency's experience of reestablished patina on weathering steel through a 3,500-psi hot water or 5,000-psi cold water wash.

Associated equipment includes a large capacity water tank, water pump, hoses and nozzles, and a pressure washer.

A number of variables can influence the effectiveness of pressure washing including the horizontal and vertical distance between the nozzle and target area, and the angle between the water stream and washing surface. The following are good rules of thumb when washing UWS bridges:

- Keep the spray nozzle at or above the target area elevation.
- Keep the spray nozzle within a reasonable distance from the target area, such as 1 to 5 ft.
- Use a nozzle with a 0 to 15° spray angle.

Site and access limitations may hinder these recommendations in certain scenarios. Examples of wet methods in use are shown in Figure 35.

Additional equipment to that listed above will be needed to carry out both dry and wet cleaning methods, such as mobilization and access equipment.

a. b. (Source: Crampton et al., 2013)

**Figure 35 - Examples of Wet Methods of Bridge Cleaning (a) Deck Cleaning; (b) Girder Pressure Washing**

### 3.4.3 Work by In-House Department Forces

Districts should establish the method of completion of the maintenance work, namely by either In-House Department Forces or by contracted services with consideration that the type and extent of maintenance action(s) may be beyond the capacities of In-House Department Forces.

### 3.4.4 Maintenance Work by Contracted Services

Maintenance work by contracted services should be initiated and performed using PennDOT's ECMS (Engineering and Construction Management System) contracting procedures as specified in Publication 51 *Plans, Specifications and Estimate Package – Delivery Process Policies & Preparation Manual*.

Some Districts utilize On-Demand Maintenance contracts for contracted maintenance services.

## References


AASHTO (2017), LRFD Bridge Construction Specifications, 4th Edition with 2020 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2018), Manual for Bridge Evaluation, 3rd Edition with 2020 Interim Revisions, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO (2019), Manual for Bridge Element Inspection, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C.

Albrecht, P., and Naeemi, A.H. (1984), NCHRP Report 272: Performance of Weathering Steel in Bridges, Transportation Research Board, National Research Council, Washington, D.C

Albrecht, P., Coburn, S.K., Wattar, F.M., Tinklenberg, G.L. and Gallagher, W.P. (1989), NCHRP Report 314: Guidelines for the Use of Weathering Steel in Bridges, Transportation Research Board, National Research Council, Washington, D.C

Ault, J.P., and Dolph, J.D. (2018), NCHRP Synthesis 517: Corrosion Prevention for Extending the Service Life of Steel Bridges, Transportation Research Board, National Academies of Sciences, Engineering, and Medicine, Washington, D.C.

Crampton, D.D, Holloway, K.P., and Fraczek, J. (2013), “Assessment of Weathering Steel Bridge Performance in Iowa and Development of Inspection and Maintenance Techniques,” Final Report SPR 90-00-RB17-012, Iowa Department of Transportation, Ames, Iowa, and Federal Highway Administration

FHWA (1989), Uncoated Weathering Steel in Structures, Technical Advisory 5140.22, Washington, D.C.

FHWA (2022) Bridge Inspector’s Reference Manual (BIRM), Federal Highway Administration, Vienna, VA

Iowa DOT (2015), Bridge Inspection Manual, Office of Bridges and Structures, Iowa Department of Transportation, Ames, Iowa

MnDOT (2023), Bridge and Structure Inspection Program Manual, Minnesota Department of Transportation, St. Paul, MN

NSBA/AISC (2022), Uncoated Weathering Steel Reference Guide, Chicago, IL

NYSDOT (2017), Bridge Inspection Manual, New York State Department of Transportation, Albany, NY

PennDOT (2023), Bridge Management System 2

PennDOT (2022), Publication 55 Bridge Maintenance Manual

PennDOT (2022), Publication 100A Bridge Coding Manual

PennDOT (2023), Publication 238 Bridge Safety Inspection Manual

# Appendix A

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# Appendix A - Uncoated Weathering Steel - Field Reference Guide for Bridge Safety Inspections

## UNCOATED WEATHERING STEEL

### Field Reference Guide for Bridge Safety Inspections

*An Overview of Key Aspects of the Safety Inspection of Weathering Steel Bridges*

Note: The information in this document focuses on uncoated weathering steel only and is not intended to be inclusive of all requirements and actions for the full coverage of bridge safety inspections.

## WHAT INSPECTORS NEED TO KNOW

A key to effective safety inspections of uncoated weathering steel (UWS) bridges is understanding the steel types, the steel protective coating, unsuitable environments, and the unique safety inspection needs.

### GENERAL

The inspection of UWS steel bridges differs from and can be more difficult than the inspection of ordinary painted steel bridges primarily due to the presence of the rust-based protective coating.

### ASTM WEATHERING STEEL TYPES

- A242, A588/A709 50W
- A709 HPS 50W, 70W, & 100W

### PROTECTIVE COATING – PATINA

- An oxide film known as patina serves as the protective coating to the steel in lieu of paint.
- Protective patina appears as a tightly adhering, uniform dense rust when fully developed.
- Nonprotective patina appears as dull gray to black with granular flakes exceeding ¼" and sheet-like layer
- If the patina becomes nonprotective the steel can and in many cases will corrode.

### UNSUITABLE CONDITIONS AND ENVIRONMENTS

- Leaking deck joints and drainage systems
- Steel framing details that accumulate debris and pond water
- Tunnel-like conditions that permit concentrated salt-laden road sprays to accumulate on the weathering steel where traffic passes under a low-clearance bridge
- Low-level water crossings with waterway debris build-up
- Vegetation in contact with steel
- Dissimilar metals
- Crevices

### UWS BRIDGE SAFETY INSPECTIONS

**Unique inspection information and requirements are as follows:**

#### VISUAL / PHYSICAL EXAMINATIONS

- Patina Condition
  - » Inspect patina for Adhesion, Texture, and Color to establish the quality
- Crack Detection
  - » Cracks in UWS bridges are more difficult to discover
- Corrosion/Section Loss:
  - » The identification of corrosion and section loss can be more difficult for UWS bridges
- Paint Condition
  - » Some surfaces of UWS such as the ends of girders are painted; this paint must be inspected
- Other Focus Areas
  - » Water/Debris Traps
  - » Leaking Deck Joints, Decks, Downspouting, and Utility Conduits
  - » Vegetative Growth and Waterway Debris Build-Up
  - » Local Environment: Tunnel effect at low overpasses and narrow clearances from air-borne salt-laden moisture
  - » Dissimilar Materials
  - » Crevices

#### TESTING

- Patina Adhesion Tape Test
  - » For determining the quality of the patina layer
- Chloride Contamination Testing
  - » Use only as approved by the Department
- Ultrasonic Testing
  - » For determining the amount of section loss; it is ok to grind the UWS with a wire wheel for cleaning
- Magnetic Particle, Dye Penetrant, Etc.
  - » For crack testing; use standard practices

pennsylvania
DEPARTMENT OF TRANSPORTATION

## COLORS OF PATINA

**Yellow Orange** - Early Development of Patina

**Light Brown** - Early Development of Patina

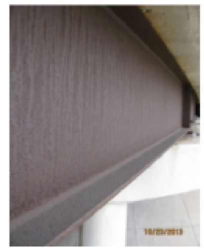
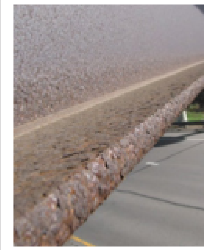
**Chocolate Brown to Purple Brown** - Fully Developed Patina

**Black** - Non-Protective Patina

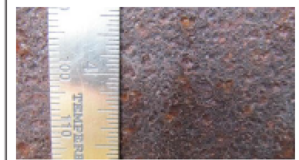
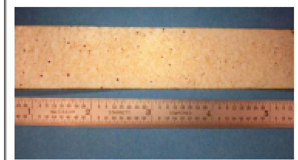
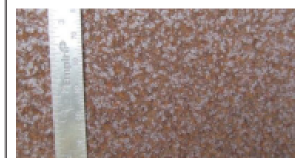
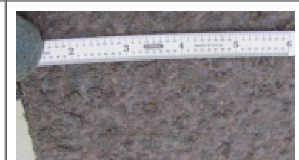
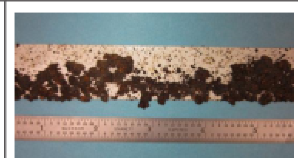
## CORRELATION BETWEEN WEATHERING STEEL TEXTURE AND CONDITION

| APPEARANCE | DEGREE OF PROTECTION |
|---|---|
| **Tightly adhered, capable of withstanding hammering or vigorous wire brushing** | Protective oxide (patina) |
| **Dusty** | Early stages of exposure; should change after a few years |
| **Granular** | Possible indication of problem, depending on length of exposure and location of member |
| **Small flakes, ¼ inch in diameter** | Initial indication of non-protective oxide |
| **Large flakes, ½ inch in diameter or greater** | Non-protective oxide |
| **Laminar sheets or nodules** | Non-protective oxide, severe condition |

## CONDITION STATES FOR STEEL PROTECTIVE COATINGS (ELEMENT 515) – DEFECT 3420 OXIDE FILM DEGRADATION

| CONDITION STATE 1 | CONDITION STATE 2 | CONDITION STATE 3 | CONDITION STATE 4 |
|---|---|---|---|
| Color is yellow-orange or light brown for early development. Chocolate-brown to purple-brown for fully developed.<br>Tightly adhered, capable of withstanding hammering or vigorous wire brushing. Smooth particles < 1/8". | Color is dark brown with some variation.<br>Granular texture. Some small flakes up to ¼". | Color is dark brown and black and some variation.<br>Small flakes, less than ½ in. diameter.<br>Section loss | Color is dark brown and black but may not be uniform.<br>Large flakes ½ in. diameter or greater, or laminar sheets of nodules.<br>Section loss |
|  |  |  |  |
| **Weathering steel patina is uniform and tightly adhered to steel beam** | **Weathering steel patina is slightly uneven – surface is granular and dusty** | **Weathering steel patina has flaking (less than ½" diameter) along bottom flange** | **Weathering steel patina has failed – large areas of surface layer flaking off** |

## PATINA CONDITION RATING

| PATINA RATING | CONDITION DESCRIPTION | EXAMPLE CONDITION IN FIELD | EXAMPLE TAPE TEST SPECIMEN |
|---|---|---|---|
| 7<br>Good to Very Good | • Uniform color pattern, generally dark brown with some lighter reddish-brown, metallic, and purple-brown spots. May be difficult to see small rust product clusters.<br>• Texture may be dimpled or rough but uniform in pattern. Patina layer is thin but dense and very adherent, indicative of very good protective properties. Superior adherence; tape test sparse with only very small flakes (< 1 mm). |  |  |
| 6<br>Satisfactory | • Uniform color pattern, generally dark brown with some lighter reddish-brown, metallic, and purple-brown spots. Individual rust product clusters visible.<br>• Texture is dimpled or rough but uniform in pattern. Patina layer is thin but dense and adherent, indicative of good protective properties. Tape test easily removes very small (< 1 mm) flakes. |  |  |
| 5<br>Fair | • Dark brown coloration, but begins to show minor variation. Flakes up to ¼" loose on surface, easily removed with tape test.<br>• Underlying layer adherent, still relatively dense, thin, and protective. Texture more granular and loose flakes may be less protective, holding water and salts.<br>• Chalky poultice layer* may be present, but not significantly affecting performance (i.e., flake size). |  |  |
| 4<br>Poor | • Dark brown with black and some color variation. Blotchy with some salty or rusty stains. Medium (¼" to ½") flakes over most of area loose and non-protective, easily removed with tape test.<br>• Layer beneath flakes thicker and more permeable, with some pitting beginning. Non-protective; contaminants penetrating.<br>• Elements with poultice may show significant associated flaking. |  |  |
| 3<br>Serious | • Color is dark brown and black but non-uniform, with widespread blotchiness and staining. Non-protective.<br>• Large (> ½") flakes, or layered delamination beginning in some areas. Thickness/permeability of rust increased, with pitting and section loss possible.<br>• Poultice* areas have thin delamination sheets or very large flakes. Layer below loose poultice may appear similar, but still somewhat adherent. |  |  |
| 2<br>Critical | • Blackish, stained, blotchy appearance.<br>• Formation of laminar sheets with deeply pitted semi-adherent layer beneath; chunks and sheets of rust product removable by hand.<br>• Aggressive advancement of pitting and section loss; can be up to 50%. Complete failure of patina to protect base steel. |  |  |

# Appendix B

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## Appendix B: Uncoated Weathering Steel - Inspection and Maintenance Plan Forms

*Note: Fillable PDF versions of Forms A and B are available on the BMS home screen under the PennDOT Bridge Inspection Forms and Template link.*

### Form A. Unsuitable and Other Reportable UWS Conditions Inspection Form

District: ________ Inspected By: ________

BMS #: ____________________ Inspection Date: __________

BRKEY: ____________________

---

**Unsuitable UWS Conditions**

1. Are there any leaking deck joints?
   Yes ☐ or No ☐
2. Is deck drainage leaking onto UWS due to:
   a. Scuppers?
      Yes ☐ or No ☐
   b. Downspouting?
      Yes ☐ or No ☐
   c. Deck Cracks?
      Yes ☐ or No ☐
3. Is water/liquid leaking onto UWS due to utility conduits?
   Yes ☐ or No ☐
4. Are water/debris traps present within the steel framing members?
   Yes ☐ or No ☐
   i. If Yes,
      1. Is moisture/debris present in the trap(s)?
         Yes ☐ or No ☐
      2. Is the patina at the trap non-protective?
         Yes ☐ or No ☐
      3. Is the UWS corroded with section loss at the trap?
         Yes ☐ or No ☐
5. Is there any vegetative growth in contact with UWS?
   Yes ☐ or No ☐
6. Is there any waterway channel buildup on UWS?
   Yes ☐ or No ☐
7. Is there any UWS member corrosion with section Loss?
   Yes ☐ or No ☐

Comments:
______________________________________________________________________________
______________________________________________________________________________
____________________________________________________

**Other Reportable UWS Conditions**

1. UWS Overpass Bridge With Limited Clearances
    a. Is this an overpass bridge
    Yes ☐ or No ☐
        i. If Yes,
            1. Is the vertical clearance (Field 4A17) less than 25 feet?
            Yes ☐ or No ☐
            2. Is the horizontal clearance (Field 4A19/4A20) less than 20 feet?
            Yes ☐ or No ☐
2. Painted Ends of UWS Beams/Girders at Deck Joints
    a. Does the bridge have bridge deck joints (Field 6A41): Yes ☐ or No ☐
    b. If Yes,
        i. Are all ends of the beams/girders at bridge deck joints painted: Yes ☐ or No ☐

---

## Form B. Uncoated Weathering Steel Maintenance Plan Form

District: ________ Established By: ________

BMS #: _____________________ Initial Plan Date: __________

BRKEY: _____________________ Revised Plan Date: _________

**This form establishes one of two maintenance plans for UWS bridges.**

**Description:**

This form records the uncoated weathering steel maintenance plan to be used for bridges with uncoated weathering steel. Two maintenance plan options are available: Standard UWS Maintenance Plan and Enhanced UWS Maintenance Plan. Districts, and as applicable other bridge owners, shall select one of these two plans for each UWS bridge.

Standard UWS Maintenance Plan

If the Standard Plan is chosen, the following ten focus areas for UWS are the desired level of maintenance which focuses on ensuring maintenance actions, as recommended in the Inspection – Maintenance Section in BMS, are performed within the timeframe of the assigned Maintenance Priority. These maintenance actions include:

- repair/replace corroded weathering steel members
- repair leaking deck expansion joints
- repair deck drainage systems that are leaking onto UWS surfaces
- repair conditions where utility conduits are leaking on UWS surfaces
- repair deck cracks that allow deck drainage to seep onto UWS surfaces
- remove vegetation in contact with UWS steel
- remove and clean debris build-up at bearings areas and water/debris trap locations such as vertical stiffener to beam flange intersections and frame legs at bearing locations
- wash weathering steel members
- spot paint corroded sections as recommended
- seal crevices

Enhanced UWS Maintenance Plan

If the Enhanced UWS Maintenance Plan is chosen, Standard UWS Maintenance Plan actions plus time-interval, preventative maintenance actions are the desired maintenance plan actions. Time-interval, preventative maintenance actions can include: power washing the entire superstructure, power washing specific steel members, replacing deck joints, and other maintenance actions.

**Procedure:**

Check one of the two boxes to indicate the desired UWS maintenance Plan.

☐ Standard UWS Maintenance Plan

☐ Enhanced UWS Maintenance Plan

For the Enhanced UWS Maintenance Plan, establish the time-interval, preventative maintenance action(s) to be used.

Check one or more boxes.

☐ Power Wash Entire Steel Framing

☐ Power Wash Specific Steel Members

☐ Replace Deck Joint Seals

☐ Other Maintenance Actions

For each checked maintenance action, provide additional data as follows:

<u>Power Wash Entire Steel Framing *</u>

When time-interval power washing the entire steel framing is chosen:

Establish the time-interval in years: ____ years

Establish the start year: 20__

<u>Power Wash Specific Steel Members *</u>

When time-interval power washing of specific steel members is chosen:

Specify the specific steel members to be power washed:
______________________________________________________________________________________
______________________________________________________________________________________
___________________________________________________________

Establish the time-interval in years: ____ years

Establish the start year: 20__

Replace Deck Expansion Joint Seals *

When time-interval replacement of deck expansion joint seals is chosen:

Specify the location(s) and type of the deck expansion joints to be replaced. i.e., Abutment No.1 strip seal gland, etc.:

______________________________________________________________________________________________
______________________________________________________________________________________________
________________________________________________________________

Establish the time-interval in years: ____ years

Establish the start year: 20__

Other Maintenance Actions: Item 1 *

When the Other Maintenance Action Items is chosen.

Establish the name of the maintenance action: ________________________________________

Establish the time-interval in years: ____ years

Establish the start year: 20__

Comments:

______________________________________________________________________________________________
______________________________________________________________________________________________
________________________________________________________________

Other Maintenance Actions: Item 2 *

When the Other Maintenance Action Items is chosen.

Establish the name of the maintenance action: ________________________________________

Establish the time-interval in years: ____ years

Establish the start year: 20__

Comments:

______________________________________________________________________________________________
______________________________________________________________________________________________
________________________________________________________________

Other Maintenance Actions: Item 3 *

When the Other Maintenance Action Items is chosen.

Establish the name of the maintenance action: ________________________________________

Establish the time-interval in years: ____ years

Establish the start year: 20__

Comments:

______________________________________________________________________________________________

______________________________________________________________________________________________

________________________________________________________________

* The time-interval and start year, as well as relevant comments, should be specified in the Notes field, IM15a, in BMS for each time-interval, preventative maintenance item. The time-interval maintenance action(s) must be re-created in BMS after completion of that maintenance work.

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# APPENDIX IP 03-A

PA Bridge Posting Vehicles
Table of Live Load Effects on Simple Spans
(No Impact Included)

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**PA Bridge Posting Vehicles Table of Live Load Effects on Simple Spans**
**(No Impact Included)**

| SPAN | LIVE LOAD MOMENT Ft.-Kips per Wheel Line | | | | LIVE LOAD SHEAR Kips per Wheel Line | | | |
|---|---|---|---|---|---|---|---|---|
| | NON-COMBINATION VEHICLE | | | COMBINATION | NON-COMBINATION VEHICLE | | | COMBINATION |
| Feet c/c | H20 | ML-80 (1) | TK-527 (1) | HS20 | H20 | ML-80 (1) | TK-527 (1) | HS20 |
| 8 | 32.0 | 23.2 | 23.2 | 32.0 | 16.0 | 15.5 | 15.5 | 16.0 |
| 9 | 36.0 | 27.8 | 27.8 | 36.0 | 16.0 | 17.2 | 16.5 | 16.0 |
| 10 | 40.0 | 36.0 | 33.0 | 40.0 | 16.0 | 18.5 | 17.3 | 16.0 |
| 11 | 44.0 | 43.4 | 40.0 | 44.0 | 16.0 | 19.7 | 18.0 | 16.0 |
| 12 | 48.0 | 51.5 | 45.3 | 48.0 | 16.0 | 20.6 | 18.5 | 16.0 |
| 13 | 52.0 | 59.0 | 52.3 | 52.0 | 16.0 | 21.4 | 19.3 | 16.0 |
| 14 | 56.0 | 67.0 | 57.7 | 56.0 | 16.0 | 22.1 | 20.0 | 16.0 |
| 15 | 60.0 | 74.5 | 64.6 | 60.0 | 16.3 | 22.7 | 20.6 | 17.1 |
| 16 | 64.0 | 82.4 | 70.0 | 64.0 | 16.5 | 23.2 | 21.1 | 18.0 |
| 17 | 68.0 | 90.0 | 77.3 | 68.0 | 16.7 | 23.6 | 21.8 | 18.8 |
| 18 | 72.0 | 97.8 | 84.5 | 72.0 | 16.9 | 24.0 | 22.4 | 19.6 |
| 19 | 76.0 | 105.5 | 91.7 | 76.0 | 17.1 | 24.8 | 23.0 | 20.2 |
| 20 | 80.0 | 113.3 | 98.9 | 80.0 | 17.2 | 25.4 | 23.5 | 20.8 |
| 21 | 84.0 | 121.0 | 106.2 | 84.0 | 17.3 | 26.0 | 24.1 | 21.3 |
| 22 | 88.0 | 128.7 | 113.3 | 88.0 | 17.5 | 26.5 | 24.7 | 21.8 |
| 23 | 92.0 | 136.4 | 120.6 | 92.0 | 17.6 | 27.0 | 25.3 | 22.3 |
| 24 | 96.0 | 144.7 | 129.2 | 96.3 | 17.7 | 27.5 | 25.8 | 22.7 |
| 25 | 100.0 | 151.6 | 136.8 | 103.7 | 17.8 | 27.9 | 26.2 | 23.0 |
| 26 | 104.0 | 160.1 | 145.5 | 111.1 | 17.8 | 28.3 | 26.6 | 23.4 |
| 27 | 108.5 | 167.8 | 153.2 | 118.5 | 17.9 | 28.6 | 27.0 | 23.7 |
| 28 | 113.4 | 177.5 | 161.9 | 126.0 | 18.0 | 28.9 | 27.4 | 24.0 |
| 29 | 118.4 | 186.5 | 169.6 | 133.5 | 18.1 | 29.2 | 27.7 | 24.4 |
| 30 | 123.3 | 196.1 | 178.6 | 141.0 | 18.1 | 29.5 | 28.0 | 24.8 |
| 31 | 128.3 | 205.3 | 187.6 | 148.6 | 18.2 | 29.8 | 28.3 | 25.2 |
| 32 | 133.2 | 214.9 | 197.0 | 156.2 | 18.3 | 30.0 | 28.6 | 25.5 |
| 33 | 138.2 | 224.0 | 206.0 | 163.9 | 18.3 | 30.3 | 28.8 | 25.8 |
| 34 | 143.2 | 233.6 | 215.2 | 171.8 | 18.4 | 30.5 | 29.1 | 26.1 |
| 35 | 148.1 | 242.9 | 224.5 | 180.6 | 18.4 | 30.7 | 29.4 | 26.4 |
| 36 | 153.1 | 252.3 | 233.6 | 189.4 | 18.4 | 30.9 | 29.8 | 26.7 |
| 37 | 158.1 | 261.8 | 242.7 | 198.3 | 18.5 | 31.1 | 30.1 | 26.9 |
| 38 | 163.0 | 271.1 | 251.8 | 207.1 | 18.5 | 31.2 | 30.4 | 27.2 |
| 39 | 168.0 | 280.4 | 261.2 | 216.0 | 18.6 | 31.4 | 30.6 | 27.4 |
| 40 | 173.0 | 289.9 | 270.3 | 224.9 | 18.6 | 31.6 | 30.9 | 27.6 |
| 42 | 182.9 | 308.7 | 288.8 | 242.7 | 18.7 | 31.9 | 31.4 | 28.0 |
| 44 | 192.9 | 327.6 | 307.3 | 260.4 | 18.7 | 32.1 | 31.8 | 28.4 |
| 46 | 202.9 | 346.4 | 325.6 | 278.3 | 18.8 | 32.4 | 32.2 | 28.7 |
| 48 | 212.8 | 365.1 | 344.0 | 296.1 | 18.8 | 32.6 | 32.6 | 29.0 |
| 50 | 222.8 | 384.0 | 362.7 | 314.0 | 18.9 | 32.8 | 33.0 | 29.3 |
| 52 | 232.8 | 402.8 | 380.9 | 331.8 | 18.9 | 33.0 | 33.3 | 29.5 |
| 54 | 242.7 | 421.7 | 399.7 | 349.7 | 19.0 | 33.2 | 33.6 | 29.8 |
| 56 | 252.7 | 440.8 | 418.7 | 367.6 | 19.0 | 33.3 | 33.8 | 30.0 |
| 58 | 265.1* | 459.4 | 438.9 | 385.4 | 19.0 | 33.5 | 34.1 | 30.2 |
| 60 | 279.0* | 478.3 | 459.3 | 403.3 | 19.1 | 33.6 | 34.3 | 30.4 |

**Publication 238 (2024 Edition), Appendix IP 03-A**
**PA Bridge Posting Vehicles Table of Live Load Effects on Simple Spans**
**(No Impact Included)**

| SPAN | LIVE LOAD MOMENT Ft.-Kips per Wheel Line | | | | LIVE LOAD SHEAR Kips per Wheel Line | | | |
|---|---|---|---|---|---|---|---|---|
| | NON-COMBINATION VEHICLE | | | COMBINATION | NON-COMBINATION VEHICLE | | | COMBINATION |
| Feet c/c | H20 | ML-80 (1) | TK-527 (1) | HS20 | H20 | ML-80 (1) | TK-527 (1) | HS20 |
| 62 | 293.3* | 497.0 | 479.7 | 421.2 | 19.1 | 33.8 | 34.6 | 30.6 |
| 64 | 307.8* | 515.9 | 500.1 | 439.1 | 19.1 | 33.9 | 34.8 | 30.8 |
| 66 | 322.7* | 534.7 | 520.5 | 457.0 | 19.2 | 34.0 | 35.0 | 30.9 |
| 68 | 338.0* | 553.7 | 540.9 | 474.9 | 19.2 | 34.1 | 35.1 | 31.1 |
| 70 | 353.5* | 572.1 | 561.8 | 492.8 | 19.2 | 34.2 | 35.3 | 31.2 |
| 72 | 369.4* | 591.4 | 582.2 | 510.7 | 19.2 | 34.3 | 35.5 | 31.3 |
| 74 | 385.5* | 610.1 | 602.7 | 528.6 | 19.2 | 34.4 | 35.6 | 31.5 |
| 76 | 402.0* | 629.1 | 623.1 | 546.6 | 19.3 | 34.5 | 35.8 | 31.6 |
| 78 | 418.9* | 648.1 | 643.5 | 564.5 | 19.3 | 34.6 | 35.9 | 31.7 |
| 80 | 436.0* | 666.8 | 663.9 | 582.4 | 19.3 | 34.7 | 36.1 | 31.8 |
| 85 | 480.3* | 713.9 | 715.1 | 627.3 | 20.1* | 34.8 | 36.4 | 32.0 |
| 90 | 526.5* | 760.9 | 766.3 | 672.2 | 20.9* | 35.0 | 36.6 | 32.3 |
| 95 | 574.8* | 808.1 | 817.5 | 717.1 | 21.7* | 35.1 | 36.9 | 32.5 |
| 100 | 625.0* | 854.9 | 868.6 | 762.0 | 22.5* | 35.3 | 37.1 | 32.6 |
| 105 | 677.3* | 902.1 | 919.8 | 806.9 | 23.3* | 35.4 | 37.3 | 32.8 |
| 110 | 731.5* | 949.7 | 971.9 | 851.8 | 24.1* | 35.5 | 37.5 | 32.9 |
| 115 | 787.8* | 996.2 | 1023.1 | 896.7 | 24.9* | 35.6 | 37.6 | 33.1 |
| 120 | 846.0* | 1044.3 | 1074.4 | 941.6 | 25.7* | 35.7 | 37.8 | 33.2 |
| 125 | 906.3* | 1091.1 | 1125.7 | 986.6 | 26.5* | 35.8 | 37.9 | 33.3 |
| 130 | 968.5* | 1137.8 | 1177.0 | 1031.5 | 27.3* | 35.8 | 38.0 | 33.4 |
| 135 | 1032.8* | 1185.2 | 1228.2 | 1076.5 | 28.1* | 35.9 | 38.1 | 33.5 |
| 140 | 1099.0* | 1232.4 | 1279.5 | 1121.4 | 28.9* | 36.0 | 38.3 | 33.6 |
| 145 | 1167.3* | 1279.3 | 1330.7 | 1167.3* | 29.7* | 36.0 | 38.4 | 33.7 |
| 150 | 1237.5* | 1326.6 | 1382.3 | 1237.5* | 30.5* | 36.1 | 38.5 | 33.8 |
| 155 | 1309.8* | 1373.9 | 1434.2 | 1309.8* | 31.3* | 36.1 | 38.5 | 33.8 |
| 160 | 1384.0* | 1421.4 | 1485.0 | 1384.0* | 32.1* | 36.2 | 38.6 | 67.8 |
| 165 | 1460.3* | 1469.0 | 1536.2 | 1460.3* | 32.9* | 36.2 | 38.7 | 34.0 |
| 170 | 1538.5* | 1515.4 | 1587.7 | 1538.5* | 33.7* | 36.3 | 38.8 | 34.0 |
| 175 | 1618.8* | 1562.1 | 1639.8 | 1618.8* | 34.5* | 36.3 | 38.8 | 34.5* |
| 180 | 1701.0* | 1609.1 | 1690.8 | 1701.0* | 35.3* | 36.4 | 38.9 | 35.3* |
| 185 | 1785.3* | 1656.7 | 1742.4 | 1785.3* | 36.1* | 36.4 | 39.0 | 36.1* |
| 190 | 1871.5* | 1704.9 | 1794.8 | 1871.5* | 36.9* | 36.4 | 39.0 | 36.9* |
| 195 | 1959.8* | 1752.0 | 1846.1 | 1959.8* | 37.7* | 36.5 | 39.1 | 37.7* |
| 200 | 2050.0* | 1797.9 | 1896.3 | 2050.0* | 38.5* | 36.5 | 39.1 | 38.5* |
| 205 | 2142.3* | 1846.6 | 1949.4 | 2142.3* | 39.3* | 36.5 | 39.2 | 39.3* |
| 210 | 2236.5* | 1892.1 | 1999.4 | 2236.5* | 40.1* | 36.6 | 39.2 | 40.1* |
| 215 | 2332.8* | 1940.8 | 2050.5 | 2332.8* | 40.9* | 36.6 | 39.3 | 40.9* |
| 220 | 2431.0* | 1988.6 | 2102.8 | 2431.0* | 41.7* | 36.6 | 39.3 | 41.7* |
| 225 | 2531.3* | 2035.1 | 2154.1 | 2531.3* | 42.5* | 36.6 | 39.4 | 42.5* |
| 230 | 2633.5* | 2080.5 | 2206.9 | 2633.5* | 43.3* | 36.7 | 39.4 | 43.3* |
| 235 | 2737.8* | 2130.0 | 2255.9 | 2737.8* | 44.1* | 36.7 | 39.4 | 44.1* |
| 240 | 2844.0* | 2175.7 | 2309.3 | 2844.0* | 44.9* | 36.7 | 39.5 | 44.9* |
| 245 | 2952.3* | 2223.0 | 2358.8 | 2952.3* | 45.7* | 36.7 | 39.5 | 45.7* |
| 250 | 3062.5* | 2269.3 | 2410.2 | 3062.5* | 46.5* | 36.8 | 39.6 | 46.5* |

**PA Bridge Posting Vehicles Table of Live Load Effects on Simple Spans**
**(No Impact Included)**

| SPAN | LIVE LOAD MOMENT Ft.-Kips per Wheel Line | | | | LIVE LOAD SHEAR Kips per Wheel Line | | | |
|---|---|---|---|---|---|---|---|---|
| | NON-COMBINATION VEHICLE | | | COMBINATION | NON-COMBINATION VEHICLE | | | COMBINATION |
| Feet c/c | H20 | ML-80 (1) | TK-527 (1) | HS20 | H20 | ML-80 (1) | TK-527 (1) | HS20 |
| 255 | 3174.8* | 2317.6 | 2463.6 | 3174.8* | 47.3* | 36.8 | 39.6 | 47.3* |
| 260 | 3289.0* | 2364.8 | 2516.1 | 3289.0* | 48.1* | 36.8 | 39.6 | 48.1* |
| 265 | 3405.3* | 2411.4 | 2566.1 | 3405.3* | 48.9* | 36.8 | 39.6 | 48.9* |
| 270 | 3523.5* | 2458.6 | 2617.6 | 3523.5* | 49.7* | 36.8 | 39.7 | 49.7* |
| 275 | 3643.8* | 2505.8 | 2669.0 | 3643.8* | 50.5* | 36.8 | 39.7 | 50.5* |
| 280 | 3766.0* | 2552.9 | 2720.5 | 3766.0* | 51.3* | 36.9 | 39.7 | 51.3* |
| 285 | 3890.3* | 2600.1 | 2772.0 | 3890.3* | 52.1* | 36.9 | 39.8 | 52.1* |
| 290 | 4016.5* | 2647.3 | 2823.5 | 4016.5* | 52.9* | 36.9 | 39.8 | 52.9* |
| 295 | 4144.8* | 2694.5 | 2875.0 | 4144.8* | 53.7* | 36.9 | 39.8 | 53.7* |
| 300 | 4275.0* | 2741.6 | 2926.4 | 4275.0* | 54.5* | 36.9 | 39.8 | 54.5* |

*Based on standard lane loading. All other values based on standard truck loading
(1) Includes 3% scale tolerance (See IP 3.2.2.2) for non-notional vehicles

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# APPENDIX IP 03-B

Guidelines for Live Load Rating of Selected Structures Without Plans Using Engineering Judgment

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**Publication 238 (2024 Edition), Appendix IP 03-B**
**Guidelines for Live Load Rating of Selected Structures**
**Without Plans Using Engineering Judgment**

## METHOD 1 – Bridges Meeting the Criteria Specified Below

**Description:**

The following is a guideline for using engineering judgment to determine the live load rating capacity of selected concrete bridges where the structural components of the main load carrying members are not known with sufficient confidence to use an analytical approach for the rating. These bridges are frequently known as "concrete bridges without plans". These guidelines follow the approach outlined in Publication 238, IP 3.6.1.1.

**Disclaimer:**

This guideline does not relieve the rating engineer of their responsibility of determining the applicability of the bridge to this methodology, of properly assessing the condition of the bridge and its behavior under live load, and/or of verifying the accuracy of the resulting ratings.

**Applicability of Guidelines:**

- The structural components of the main load carrying members are not known sufficiently to use an analytical approach to determine the live load ratings
- The condition of the main load carrying members is known and rated using the Condition Rating as set forth in BMS2 Coding Manual, Publication 100A
- The behavior of the bridge under vehicular live load is known by visual observation
- This method is limited to the following types of non-NSTM superstructures
  - Reinforced Concrete Slab
  - Reinforced Concrete T-Beam
  - Prestressed/Pretensioned Concrete Beams (Not permitted for Adjacent Non-Composite Prestressed Concrete Box Beams)
- The method is limited to simple span structures with minimum length of 8' and for all skews
- Based on engineering judgment, SLC values greater than those given in Table A may be used, however the value shall be no greater than the Operating Rating.

**Assumptions:**

1. The critical legal load for the range of applicability is the ML80 vehicle
2. Moment controls the live load rating
3. Safe Load Capacity = 100% of Operating Rating, except for members in poor, critical, or serious condition
   When following the procedure below, a reduction factor is already incorporated into Table A for bridges with a superstructure and/or substructure condition rating of 4 or less. The Safe Load Capacity Reduction Factor should not be applied as shown in Publication 238, IP 3.6.1.1.
4. Inventory Rating = 60% of Operating Rating

**Procedure:**

1. Determine the condition rating for the critical main load-carrying member of the bridge from a bridge safety inspection performed in accordance with Publication 238.

2. Determine the distress level of the bridge superstructure under vehicular live load using Table B of these guidelines.

   NOTE: Member condition ratings of 5 through 9 should not see distress under live load. If distress is observed, the member condition rating should be no higher than a 4.

3. Determine the ML80 truck live load ratings ($IR_{ML80}$-Inventory Rating, $OR_{ML80}$-Operating Rating, $SLC_{ML80}$- Safe Load Capacity) from Table A of these guidelines, using the following:
   A. Condition rating of main load carrying member
   B. Distress level of bridge superstructure
   C. ADTT (Average Daily Truck Traffic) on the bridge

4. Determine the live loadings for the other Bridge Posting Vehicles (H, HS, TK527) based on a comparison

of their live load bending moments to the ML80 bending moment for the bridge's span length.

A. Determine the Rating Factor for ML80 Safe Load Capacity Rating Factor for $SLC_{ML80}$ = ($SLC_{ML80}$ / $W_{ML80}$)
*Where: $SLC_{ML80}$ is from Table A*

B. *$W_{ML80}$ = ML80 Gross Vehicle Weight = 36.64 T* Determine the Rating Factor for each of the Bridge Posting Vehicles at SLC using the ratio of live load moments. Using HS as an example:

Rating Factor for $SLC_{HS}$ = (Rating Factor for $SLC_{ML80}$) * ($LLM_{ML80}$ / $LLM_{HS}$)
*Where: LLM = Live Load Moment for each vehicle*
*LLM values from App IP-03A may be used here*

C. Determine the SLC (in Tons) for each of the Bridge Posting Vehicles. Again, using HS as the example:

$SLC_{HS}$ = (Rating Factor for $SLC_{HS}$) * ($W_{HS}$)
*Where: $W_{HS}$ = HS Gross Vehicle Weight = 36.0 Tons*

D. Determine the Operating Rating and Inventory Rating for each of the Bridge Posting Vehicles. Continuing with the HS example:

$OR_{HS}$ = $SLC_{HS}$ * ($OR_{ML80}$/$SLC_{ML80}$)

$IR_{HS}$ = 60% * $OR_{HS}$

E. Repeat Steps 4B through 4D for H and TK527 vehicles

5. The rating engineer is to review the ratings developed using this method to determine if the results are acceptable based on the rater's knowledge of the bridge and other traffic and site conditions. The rating engineer may decide to adjust the values obtained based on the factors to consider listed in IP 3.6.1.1. The rating engineer is to place their signature and affix their PE stamp to the rating documentation.

6. The Load Rating Summary Form for Engineering Judgment Ratings – Method 1 (Page 5 of 10) is acceptable documentation for a live load rating developed using these guidelines.

7. The rating engineer develops a post evaluation and recommendation based upon live load capacities determined in Steps 1-5. The posting level must be between the limits of the Inventory Rating and Safe Load Capacity. Posting increments of 5 tons shall generally be used.

**TABLE A: ML80 Live Load Ratings for Selected Concrete Bridges without Plans – Method 1**

| Controlling Member | | ML80 Ratings (Tons) | | | |
|---|---|---|---|---|---|
| **Condition Rating** | **Distress Level** | **Safe Load Capacity** | | **Operating Rating** | **Inventory Rating** |
| **9** Excellent | N/A | 65 | | 65 | 39 |
| **8** Very Good | N/A | 65 | | 65 | 39 |
| **7** Good | N/A | 65 | | 65 | 39 |
| **6** Satisfactory | N/A | 60 | | 60 | 36 |
| **5** Fair | N/A | 60 | | 60 | 36 |
| **4** Poor | 1 | *ADTT ≥500* | 36 | 45 | 27 |
| | | *ADTT <500* | 40 | | |
| | 2 | *ADTT ≥500* | 30 | 40 | 24 |
| | | *ADTT <500* | 36 | | |
| **3** Serious | 1 | *ADTT ≥500* | 30 | 40 | 24 |
| | | *ADTT <500* | 32 | | |
| | 2 | *ADTT ≥500* | 18 | 25 | 15 |
| | | *ADTT <500* | 20 | | |
| | 3 | 12 | | 20 | 12 |
| **2** Critical | 2 | 6 | | 10 | 6 |
| | 3 | 3 | | 5 | 3 |
| **1** | N/A | Closed | | 0 | 0 |

**NOTES:** N/A Not Applicable

**TABLE B: Distress Level for selected concrete bridges without plans**

| DISTRESS LEVEL | | GENERAL DESCRIPTION OF DISTRESS | SPECIFIC DESCRIPTION |
|---|---|---|---|
| Prestressed Concrete Superstructure | Reinforced Concrete Superstructure | | |
| 1 | 1 | GOOD | No signs of Live Load induced distress due to vehicular traffic |
| 1 | 2 | FAIR | Hairline flexure cracks visible |
| 2 | 3 | FAIR | Working flexure cracks with movement visible under vehicular loads – Or loss of anchorage for mild reinforcement |
| 3 | 3 | POOR | Working shear cracks or working flexure cracks with visible deflection under vehicular loads – Or loss of anchorage for P/S strand |

**Load Rating Summary Form for Engineering Judgment Ratings – Method 1**

**County:** ______________________ **S.R. ID.** ______________________

**Feature Carried:** ______________________ **Over:** ______________________

**Bridge Type:** ______________________________________________

**Span Length:** ______________________ **ADTT:** ____________

**Controlling Member:** ______________________________________________

**Condition:** ______________________________________________

______________________________________________

______________________________________________

**Distress Level:** ______________________ **Date of Inspection:** ____________

| Bridge Posting Vehicle | Live Load Moment | SLC Rating Factor | W (Tons) | Ratings in Tons | | | Comments |
|---|---|---|---|---|---|---|---|
| | | | | SLC | OR | IR | |
| ML80 | | | | | | | From Table A |
| | | | 36.64 | | | | Calculated |
| H | | | 20.00 | | | | Calculated |
| TK527 | | | 40.00 | | | | Calculated |
| HS | | | 36.00 | | | | Calculated |

| BMS2 DATA | Vehicle Configurations | | | | | | |
|---|---|---|---|---|---|---|---|
| | 1: H | 2: HS | 8: ML80 | 4 | 5 | LF | 0: TK527 |
| IR10 IR | | | | | | NNN | |
| IR11 OR | | | | | | NNN | |
| IR11a SLC | | | | | | NNN | |

**IR06 = 7 - Engineering Judgment**

**Prepared By:** ______________________________
Rating Engineer

**Date:** ____________

**Reviewed By:** ______________________________

**Date:** ____________

**Engineer's Seal:**

## METHOD 2 – Bridges Not Meeting the Criteria for Method 1

**Description:**

The following is a guideline for using engineering judgment to determine the live load rating capacity of selected bridges where the structural components of the main load carrying members are not known with sufficient confidence to use an analytical approach for the rating. Method 2 shall be utilized for bridges, which require an engineering judgment rating, but do not meet the requirements of Method 1. These guidelines follow the approach outlined in Publication 238, IP 3.6.1.1.

**Disclaimer:**

This guideline does not relieve the rating engineer of their responsibility of determining the applicability of the bridge to this methodology, of properly assessing the condition of the bridge and its behavior under live load, and/or of verifying the accuracy of the resulting ratings.

**Applicability of Guidelines:**

- The structural components of the main load carrying members are not known sufficiently to use an analytical approach to determine the live load ratings
- The condition of the main load carrying members is known and rated using the Condition Rating as set forth in BMS2 Coding Manual, Publication 100A
- The behavior of the bridge under vehicular live load is known by visual observation
- This method shall be utilized for bridges that do not meet the requirements of Method 1.
- Based on engineering judgment, SLC values greater than those given in Table A may be used, however the value shall be no greater than the Operating Rating.

**Assumptions:**

1. Inventory Rating = 60% of Operating Rating

2. The SLC reduction factors are in accordance with IP 4.3.2 and are already accounted for in Tables C and D. If the substructure condition rating is ≤ 4 and lower than the superstructure or culvert rating, an SLC factor shall be applied to the Operating Rating Factor to be in accordance with IP 4.3.2. As indicated in IP 4.3.2, the SLC factor for the H20 vehicle is 1.0 for all cases.

**Procedure:**

1. Determine the condition rating for the critical main load-carrying member of the bridge (Superstructure or Culvert) from a bridge safety inspection performed in accordance with Publication 238.

2. For bridges with a superstructure or culvert rating ≤ 4, determine which level best describes the section loss. Level 1 indicates there may be section loss to main load carry members; however, the section loss is in a non-critical area. Level 2 indicates there is section loss to main load carrying members in critical areas.

3. Determine the rating factors from Tables C or D of these guidelines, using the following:
   A. Use Table C for lower risk structures. Use Table D for higher risk structures, which includes additional 0.9 reduction factor (e.g. stone masonry arch and metal arch bridges).
   B. Condition rating of main load carrying member (Superstructure or Culvert)
   C. Level of bridge superstructure (Level 1 or Level 2)
   D. ADTT (Average Daily Truck Traffic) on the bridge

4. Calculate the rating tons using the gross vehicle weights of each vehicle.

5. The rating engineer is to review the ratings developed using this method to determine if the results are acceptable based on the rater's knowledge of the bridge and other traffic and site conditions. The rating engineer may decide to adjust the values obtained based on the factors to consider listed in IP 3.6.1.1. The rating engineer is to place their signature and affix their PE stamp to the rating documentation.

6. The Load Rating Summary Form for Engineering Judgment Ratings – Method 2 (Page 9 of 10) is acceptable documentation for a live load rating developed using these guidelines.

7. The rating engineer develops a post evaluation and recommendation based upon live load capacities determined in Steps 1-5. The posting level must be between the limits of the Inventory Rating and Safe Load Capacity. Posting increments of 5 tons shall generally be used.

**TABLE C: Rating Factors for Select Bridges - Method 2 – Non-High-Risk Bridges**

| **Controlling Member** | | **Rating Factors** | | | |
|---|---|---|---|---|---|
| **Superstructure or Culvert Condition Rating** | **Level [1]** | **Safe Load Capacity [2]** | | **Operating Rating** | **Inventory Rating** |
| **9** Excellent | N/A | 1.67 | | 1.67 | 1.00 |
| **8** Very Good | N/A | 1.67 | | 1.67 | 1.00 |
| **7** Good | N/A | 1.67 | | 1.67 | 1.00 |
| **6** Satisfactory | N/A | 1.50 | | 1.50 | 0.90 |
| **5** Fair | N/A | 1.50 | | 1.50 | 0.90 |
| **4** Poor | 1 | ADTT ≥ 500 | 1.00 | 1.26 | 0.76 |
| | | ADTT < 500 | 1.11 | | |
| | 2 | ADTT ≥ 500 | 0.90 | 1.11 | 0.67 |
| | | ADTT < 500 | 1.00 | | |
| **3** Serious | 1 | ADTT ≥ 500 | 0.80 | 1.11 | 0.67 |
| | | ADTT < 500 | 0.90 | | |
| | 2 | ADTT ≥ 500 | 0.55 | 0.80 | 0.50 |
| | | ADTT < 500 | 0.65 | | |
| **2** Critical | N/A | 0.20 | | 0.25 | 0.15 |
| **1** | N/A | Closed | | 0.00 | 0.00 |

Notes:
[1] Level 1: Section loss to main load carry members is in non-critical areas.
Level 2: Section loss to main load carry members is in critical areas.
[2] Safe Load Capacity is applied to HS20, ML80, and TK527 (Not applied to H20).

**TABLE D: Rating Factors for Select Bridges - Method 2 – High-Risk Bridges**
**(Table C values multiplied by 0.9 for higher risk bridges – e.g. Stone masonry arch and metal arch bridges)**

<table>
<tr><th colspan="2">Controlling Member</th><th colspan="4">Rating Factors</th></tr>
<tr><th>Superstructure or Culvert Condition Rating</th><th>Level [1]</th><th colspan="2">Safe Load Capacity [2]</th><th>Operating Rating</th><th>Inventory Rating</th></tr>
<tr><td>9<br>Excellent</td><td>N/A</td><td colspan="2">1.50</td><td>1.50</td><td>0.90</td></tr>
<tr><td>8<br>Very Good</td><td>N/A</td><td colspan="2">1.50</td><td>1.50</td><td>0.90</td></tr>
<tr><td>7<br>Good</td><td>N/A</td><td colspan="2">1.50</td><td>1.50</td><td>0.90</td></tr>
<tr><td>6<br>Satisfactory</td><td>N/A</td><td colspan="2">1.35</td><td>1.35</td><td>0.81</td></tr>
<tr><td>5<br>Fair</td><td>N/A</td><td colspan="2">1.35</td><td>1.35</td><td>0.81</td></tr>
<tr><td rowspan="4">4<br>Poor</td><td rowspan="2">1</td><td>ADTT ≥ 500</td><td>0.90</td><td rowspan="2">1.13</td><td rowspan="2">0.68</td></tr>
<tr><td>ADTT < 500</td><td>1.00</td></tr>
<tr><td rowspan="2">2</td><td>ADTT ≥ 500</td><td>0.81</td><td rowspan="2">1.00</td><td rowspan="2">0.60</td></tr>
<tr><td>ADTT < 500</td><td>0.90</td></tr>
<tr><td rowspan="4">3<br>Serious</td><td rowspan="2">1</td><td>ADTT ≥ 500</td><td>0.72</td><td rowspan="2">1.00</td><td rowspan="2">0.60</td></tr>
<tr><td>ADTT < 500</td><td>0.81</td></tr>
<tr><td rowspan="2">2</td><td>ADTT ≥ 500</td><td>0.50</td><td rowspan="2">0.72</td><td rowspan="2">0.45</td></tr>
<tr><td>ADTT < 500</td><td>0.59</td></tr>
<tr><td>2<br>Critical</td><td>N/A</td><td colspan="2">0.18</td><td>0.23</td><td>0.14</td></tr>
<tr><td>1</td><td>N/A</td><td colspan="2">Closed</td><td>0.00</td><td>0.00</td></tr>
<tr><td>Notes:</td><td colspan="5">[1] Level 1: Section loss to main load carry members is in non-critical areas.<br>Level 2: Section loss to main load carry members is in critical areas.<br>[2] Safe Load Capacity is applied to HS20, ML80, and TK527 (Not applied to H20).</td></tr>
</table>

## LOAD RATING SUMMARY FORM

**(For Engineering Judgement Ratings - Method 2)**

Done By: ____________ Date: ____________
Checked By: ____________ Date: ____________

Structure ID (5A01): ____________ Inspection Date (7A01): ____________
Facility Carried (5A08): ____________
Feature Intersected (5A07): ____________
Structure Type: ____________
Analysis Method (IR06): Engineering Judgement - Method 2 SLC RF
Controlling Member: [ ] Controlling Cond. Rating : [ ] [ ]
ADTT: [ ] Substructure Rating : [ ] [ ]
Level: [ ] Additional SLC Red. factor for Substructure: 1.00
High Risk Structure: [ ]
Reduction Factor for High Risk: 1.00 (Applied to IR, OR, and SLC)

Justification for Level:

## GOVERNING LOAD RATINGS

| VEHICLE | IR | | OR | | SLC | | Controlling Member | | Load Effect (M/V) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | | | | | | | | | | |
| HS20 | | | | | | | | | | |
| ML80 | | | | | | | | | | |
| TK527 | | | | | | | | | | |

Comments / Assumptions*:

Engineer's Seal:

* Provide justification for using engineering judgement. Provide justification for rating factors not in accordance with Pub 238 Appendix IP 03-C. Indicate the recommending posting level when needed. These comments should also be recorded in field IR19 in addition to the condition of the controlling member and Level.

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# APPENDIX IP 03-C

Load Rating Summary Form

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## LOAD RATING SUMMARY FORM

Done By: ____________________ Date: ____________

Checked By: ____________________ Date: ____________

Structure ID (5A01): ______________________________ Inspection Date (7A01): ____________

Facility Carried (5A08): ______________________________

Feature Intersected (5A07): ______________________________

Structure Type (6A26 - 6A29): ______________________________

Spans / Members Analyzed: ______________________________

Analysis Method: ______________________________

PennDOT Program / Version: ______________________________

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | | | | | | | | | | |
| HS20 | | | | | | | | | | |
| ML80 | | | | | | | | | | |
| TK527 | | | | | | | | | | |
| PHL-93 | | | | | | | | | | |
| EV2 | | | | | | | | | | |
| EV3 | | | | | | | | | | |

Comments/Assumptions*: ______________________________

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

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# APPENDIX IP 03-D

Assigned Load Rating Approval

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APPENDIX IP 03-D

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DEPARTMENT OF TRANSPORTATION

# ASSIGNED LOAD RATING APPROVAL

**Structure ID (5A01):** ____________________ **BRKEY (5A03):** ____________________

**Facility Carried (5A08):** ____________________

**Feature Intersected (5A07):** ____________________

**The following conditions, described in Sec. IP 3.6.1.2, must be met to assign the load rating from the design plans/calculations:**

1. The bridge was designed and checked using either the AASHTO Load and Resistance Factor Design (LRFD) or Load Factor Design (LFD) methods to at least PHL-93 or HS-20 live loads, respectively. *

   _____ True _____ False

2. The bridge was built in accordance with the design plans or shop drawings.

   _____ True _____ False

   Design Drawing Number:

3. No changes to the loading conditions or the structure condition have occurred that could reduce the inventory rating below the design load level.

   _____ True _____ False

4. An evaluation has been completed and documented, determining that the force effects from State legal loads or permit loads do not exceed those from the design load (Note: This will be true for all bridges designed by PennDOT standards for the PHL-93 vehicle).

   _____ True _____ False

5. The checked design calculations, and relevant computer input and output information, are accessible and referenced or included in the individual bridge records. *

   _____ True _____ False

** Note: If complete design files have not been retained for existing bridges, design plans that clearly identify the loading as at least PHL-93 or HS-20 and bear the stamp of a licensed professional engineer may be used by the individual responsible for load rating under 23 CFR 650.309(c) as the basis for an assigned load rating. The approval needs to be documented as the basis for the assigned rating and become part of the official bridge records. This information demonstrates satisfaction of conditions (1) and (5) above. Conditions (2), (3), and (4) still need to be met.*

☐ Design plans have been used to satisfy (1) and/or (5) above in lieu of complete design files

This bridge meets the above mentioned criterion (all marked "True") and qualifies for an assigned load rating.

Signed: ____________________

Name: ____________________

Title: ADBE-I (State-owned) or
Local/Other Owner Engineer (Non-state owned) **

Date: ____________________

*** Assistant District Bridge Engineer-Inspection or an individual meeting 23 CFR 650.309(c) and delegated, in writing, this approval authority. For non-state owned bridges, a registered professional engineer employed by the local/other owner or their consultant shall have approval authority.*

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# APPENDIX IP 03-E

Load Rating Best Practices Manual

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# Load Rating Best Practices Manual

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# LIST OF FIGURES

# INTRODUCTION

## PURPOSE AND SCOPE

The purpose of this Manual is to provide guidance to Department engineering staff, consultants and contractors performing and submitting load rating evaluations of PennDOT bridges. Users of this Manual will develop more confidence and comfortability when evaluating the safe live load carrying capacity of bridges. This Manual suggests practices to avoid common mistakes observed in previous PennDOT load rating submissions. Following the guidance provided in this Manual will lead to more consistent, thorough, and reproducible load rating evaluations.

Procedures and practices described in this Manual conform to PennDOT preferences and policies. This Manual serves as a supplement to the Key References listed below. It aims to provide suggestions and methods where the Key References leave room for interpretation.

PennDOT approved programs and simplified analyses are to be utilized in the development of load ratings for the majority of PennDOT bridges. A refined analysis should only be considered when PennDOT approved software is not applicable or to improve ratings. Specific situations where a refined analysis is warranted are discussed throughout this Manual and must be coordinated with the District, Central Office, or Owner.

## DEFINITIONS

As-Built Plans – Plans that show the structure condition at the end of construction.

As-Built Ratings – Load ratings that are developed based on the as-built plans.

As-Inspected Ratings – Load ratings that are developed based on the condition of the members depicted in the most recent inspection report.

Bridge – A structure, including supports, erected over a depression or an obstruction, such as water, highway, or railway, having an opening of more than 8 ft., and having a track or passageway for carrying traffic or other moving loads.

Bridge Management System 2 (BMS2) – A PennDOT system designed to optimize the use of available resources for the inspection, maintenance, rehabilitation, and replacement of bridges.

Condition Rating – An evaluation of the physical condition of a bridge component in comparison to its original as-built condition.

Extreme Event Limit States – Limit states relating to events such as earthquakes, ice load, and vehicle and vessel collision, with return periods in excess of the design life of the bridge.

Failure – A condition where a limit state is reached or exceeded. This may or may not involve collapse or other catastrophic occurrences.

Non-redundant Steel Tension Members (NSTM).– As stated in the NBIS, a "NSTM 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".

Force Effects/Load Effects – A deformation, stress, or stress resultant (i.e., axial force, shear force, or torsional or flexural moment) caused by applied loads, imposed deformations, or volumetric changes.

Inventory Rating (IR) – Load ratings that allow comparisons with the capacity for new structures, and, therefore, results in a live load, which can safely utilize an existing structure for an indefinite period of time.

Limit State – A condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed.

Live Load Distribution Factor – The fraction of the particular live load vehicle that will be resisted by the structural member under consideration.

Load Factor – A statistically-based multiplier applied to force effects accounting primarily for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads, but also related to the statistics of the resistance through the calibration process.

Load Rating – According to the Code of Federal Regulations (CFR), Title 23 Highways, Section 650.305, load rating is defined as "The analysis to determine the safe vehicular live load carrying capacity of a bridge using bridge plans and supplemented by measurements and other information gathered from an inspection."

National Bridge Inspection Standards (NBIS) – Federal regulations establishing requirements for inspection procedures, frequency of inspections, qualifications of personnel, inspection reports, and preparation and maintenance of bridge inventory records. The NBIS apply to all structures defined as bridges located on or over all public roads.

National Bridge Inventory (NBI) – Structure inventory containing information to fulfill the requirements of NBIS.

Operating Rating (OR) – Load ratings based on the maximum permissible live load to which the structure may be subjected. Allowing unlimited numbers of vehicles to use the bridge at operating level may shorten the life of the bridge.

Quality Assurance (QA) – The independent verification or measurement of the level of quality of a sample product or service.

Quality Control (QC) – The enforcement, by a supervisor, of procedures that are intended to maintain the quality of a product or service at or above the specified level.

Rating Factor – The ratio of the member's capacity to the load effects produced by the particular live load vehicle under consideration.

Resistance Factor – A statistically-based multiplier applied to nominal resistance accounting for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance, but also related to the statistics of the loads through the calibration process.

Service Limit States – Limit states relating to stress, deformation, and cracking under regular operating conditions.

Shop Drawings – Drawings produced by a contractor, supplier, or manufacturer for specific components of a bridge. These drawings typically show more detail than construction documents.

Strength Limit States – Limit states relating to strength and stability during the design life.

**ABBREVIATIONS**

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| ABLRFD | PennDOT's abutment LRFD computer software design and analysis program |
| AISC | American Institute of Steel Construction |
| ARCH | PennDOT's arch culvert LFD computer software design and analysis program |
| ASD | Allowable Stress Design |
| ASR | Allowable Stress Rating |
| BAR7 | PennDOT's LFD and ASD computer analysis and load rating program for bridges |
| BC-(prefix) | PennDOT's Bridge Construction Standards (alpha prefix of numbered standards) |
| BD-(prefix) | PennDOT's Bridge Design Standards (alpha prefix of numbered standards) |
| BMS2 | Bridge Management System 2 |
| BOX5 | PennDOT's box culvert LFD computer software design and analysis program |
| BRKEY | Bridge Key |
| BrR | AASHTOWare Bridge Rating Software |
| BXLRFD | PennDOT's box culvert LRFD computer software design and analysis program |
| C/D | Capacity to Demand Ratio |
| CFR | Code of Federal Regulations |
| CMP | Corrugated Metal Pipe |
| DBE | District Bridge Engineer |

| | |
|---|---|
| DL | Dead Load |
| EV | Emergency Vehicles |
| FAST Act | Fixing America's Surface Transportation Act |
| FBLRFD | PennDOT's steel floorbeam LRFD computer software design and analysis program |
| FEA | Finite Element Analysis |
| FEM | Finite Element Model |
| FHWA | Federal Highway Administration |
| FRP | Fiber Reinforced Polymer |
| FWS | Future Wearing Surface |
| GVW | Gross Vehicle Weight |
| IM | Dynamic Load Allowance or Impact |
| IR | Inventory Rating |
| LFD | Load Factor Design |
| LFR | Load Factor Rating |
| LL | Live Load |
| LLDF | Live Load Distribution Factor |
| LRFD | Load and Resistance Factor Design |
| LRFR | Load and Resistance Factor Rating |
| NBI | National Bridge Inventory |
| NBIS | National Bridge Inspection Standards |
| NCABB | Non-Composite Adjacent Box Beam |
| NCHRP | National Cooperative Highway Research Program |
| NLF | No Load Fit |
| NSBA | National Steel Bridge Alliance |
| NSTM | Nonredundant Steel Tension Members |

| | |
|---|---|
| ODOT | Ohio Department of Transportation |
| OR | Operating Rating |
| PA | Pennsylvania |
| PE | Professional Engineer |
| PEB | Plate with Eccentric Beam |
| PennDOT | Pennsylvania Department of Transportation |
| PL | Pedestrian (Sidewalk) Live Load |
| PSLRFD | PennDOT's prestressed concrete girder LRFD computer software design and analysis program |
| PS3 | PennDOT's prestressed concrete girder LFD computer software design and analysis program |
| PUB | PennDOT Publication |
| QA | Quality Assurance |
| QC | Quality Control |
| SDLF | Steel Dead Load Fit |
| SLC | Safe Load Capacity |
| SNBI | Specifications for the National Bridge Inventory |
| STLRFD | PennDOT's steel girder LRFD computer software design and analysis program |
| TDLF | Total Dead Load Fit |

**KEY REFERENCES**

PennDOT load ratings must adhere to AASHTO, as supplemented by DM-4 and PUB 238. The following key references are useful for users of this manual as they develop load ratings:

AASHTO Publications

MBE – The Manual for Bridge Evaluation, 3rd Edition including Revisions up to 2020 Interims

AASHTO LRFD – AASHTO LRFD Bridge Design Specifications, 8th Edition September 2017

AASHTO Std. Spec. – Standard Specifications for Highway Bridges, 17th Edition 2002

PennDOT Publications

DM-4 (PUB 15M) – Design Manual, Part 4, Structures, December 2019 Edition

PUB 238 – Bridge Safety Inspection Manual, 2024 Edition

References made throughout this Manual are to the specific publication editions and dates listed above.

# SECTION 1 – GENERAL LOAD RATING PROCESS

## 1.1 BASICS

### 1.1.1 What is a Load Rating?

According to the Code of Federal Regulations (CFR), Title 23 Highways, Section 650.305, a load rating is defined as "The analysis to determine the safe vehicular live load carrying capacity of a bridge using bridge plans and supplemented by measurements and other information gathered from an inspection."

The load rating or vehicular live load carrying capacity is calculated by subtracting the dead load force effects from the capacity of the component being evaluated, and then dividing by the predetermined vehicular live load force effects. Different factors in the equation are used depending on which method of analysis (LRFR, LFR, or ASR) is employed. This is discussed in further detail in Section 1.1.3 of this Manual.

The bridge plans provide the design and material properties of the various components of the bridge. Shop drawings contain information on various components of the bridge including information on the bridge girders. The field inspection provides the current conditions of the various components of the bridge. Considering the current conditions allows for a more accurate assessment as what is shown on the bridge plans may not reflect what is in the field. This may be due to unforeseen circumstances such as a construction change or error that was not documented or material deterioration such as corrosion or spalling.

### 1.1.2 Why is a Load Rating Performed?

Load rating is performed for three main reasons: posting and permitting, bridge management, and to meet the requirements of the FHWA's NBIS.

#### 1.1.2.1 Posting and Permitting

When a structure has insufficient load capacity based on its load rating, a posting (i.e., weight restriction) is required. The posting can be either the maximum weight limit, a one truck at a time limit, or a combination of both to restrict vehicular live loads. The purpose of such postings is to ensure the safe load capacity of the structure is not exceeded. This reduces the likelihood of the structure being overstressed which would cause structural damage or failure. 23 CFR 650.313(l) requires the implementation of load postings or restrictions when the maximum unrestricted legal loads or State routine permit loads exceed that allowed under the design load operating rating, legal load rating, or permit load analysis. In cases where the bridge has a poor, serious, or critical condition rating (4 or less) of the superstructure and some cases the substructure, PennDOT requires reducing the legal load ratings by the Safe Load Reduction Factor to determine the SLC to post a bridge. Additional information on bridge posting evaluations and the SLC can be found in PUB 238 IP Article 4 and in Section 1.8.4 of this document.

In addition, load rating is performed to assist in the overload permit review process. When a vehicle exceeds the safe load capacity of a structure at the inventory level, it may be able to cross the bridge legally if it does not exceed the load capacity at operating level and obtains an approved PennDOT Permit Form. Additional information on permitting can be found in PUB 238 IP Article 10.

PennDOT's PUB 764 provides further information regarding the posting and permit review process.

1.1.2.2 Bridge Management

According to FHWA, "bridge management is a core bridge discipline that focuses on making informed and effective decisions on the operation, maintenance, preservation, replacement, and improvement of bridges within a bridge inventory." The information obtained from performing a load rating allows one to determine the best course of action for a bridge. For example, no posting actions are required for a bridge with an operating load rating above 1.0. That bridge is safe to be used by current design loads..

1.1.2.3 FHWA Requirements

Bridge inventory information, including load rating data, is to be collected by each State and reported to the FHWA in accordance with 23 CFR Subpart C - National Bridge Inspection Standards (NBIS). The specific inventory reporting requirement is stipulated in 23 CFR 650.315. The inventory information is maintained in the NBI and used to monitor and manage bridges on the National Highway System to ensure safety and for reporting the performance of the Nation's bridges to Congress for Federal funding programs. Guidance on reporting load rating for the NBI can be found in Section 5 of the Specifications for the National Bridge Inventory (SNBI), released in March 2022. The SNBI replaces the "Recording and Coding Guide of the Structure Inventory and Appraisal of the Nation's Bridges" (Report No. FHWA-PD-96-001). PennDOT uses BMS2 for recording and storing bridge inventory and inspection data for Pennsylvania's bridges (PUB 238 IP Article 5.1). Guidance on inputting load rating data in BMS2 can be found in PUB 100A.

Additional NBIS load rating requirements are stipulated in 23 CFR 650.313(k), each bridge is to be rated to its safe load carrying capacity using the MBE; this is a requirement for new and existing bridges. Re-ratings are also required when a change in the structure is identified such as a condition change, reconstruction, new construction, or changes in dead or live loads. Load rating data is reviewed annually by FHWA for compliance with the NBIS (PUB 238 IP Article 1.3.1.1).

### 1.1.3 Methods

There are three common methods for load rating:

- Load and Resistance Factor Rating (LRFR)
- Load Factor Rating (LFR)
- Allowable Stress Rating (ASR)

According to the FHWA, LRFR is currently the preferred rating method and is the required method when reporting bridge inventory information in the NBI for all bridges designed using LRFD after October 1, 2010. LRFR is also required when doing a re-rating if the previous rating was done in LRFR. The LFR method is used for existing bridges originally designed using LFD or designed using LRFD prior to October 1, 2010, unless the bridge was rated using LRFR previously. The ASR method is only permitted in certain situations where the LRFR or LFR methods are not required. Additional federal guidance on which rating method to use can be found in the FHWA "Bridge Load Ratings for the National Bridge Inventory" memo, dated October 30, 2006.

PennDOT specific rating method guidance can be found in PUB 238 IP Articles 3.1 and 3.6. Table 3.6.2-1 in Article 3.6 clearly defines which rating method to use for bridge load rating evaluation results that are reported to the NBI. These selections are based on the method used for the original design of the bridge, the date the bridge was initially placed in service (i.e., opened to traffic), and the purpose for the current rating evaluation (e.g., initial design rating, reconstruction rating, re-rating due to loading or condition change, superstructure replacement rating, etc.).

No rating method requirements are provided in the MBE. MBE Article 6.1, states that no preference is placed on any rating method; the above mentioned FHWA and PUB 238 requirements therefore govern which rating method to use.

The general load rating equation is specific to the chosen rating method. The load rating equation can be found in MBE Article 6A.4.2 (LRFR) and MBE Article 6B.4 (LFR and ASR).

#### 1.1.3.1 Load and Resistance Factor Rating (LRFR)

The LRFR method is defined in MBE Article 6A.1.

The LRFR method utilizes the design philosophy and approach of the AASHTO LRFD Bridge Design Specifications. The method employs both load and resistance factors for strength and service limits applicable to bridge load rating. These factors and limit states are defined in the MBE, Table 6A.4.2.2-1. As noted in Section 1.1.1, the factored dead load is subtracted from the factored resistance and then divided by the factored live load to obtain a rating factor using LRFR. When determining the resistance of a member with deterioration, MBE Article 6A.4.2.3 provides direction for using an optional condition factor to account for uncertainty in the severity of the deterioration and to account for increased future deterioration. If PennDOT's SLC factor is then applied, the use of the MBE's optional condition factor as well as the SLC factor would be conservative (In this case, typically apply the PennDOT SLC factor only). Per MBE Article 6A.4.2.4, the LRFR method also incorporates a system factor for applying to the flexural and axial resistance of a member to reflect the redundancy, or lack thereof, of specific superstructure types or when rating gusset plates.

There are three procedures within the LRFR method:

- Design Load Rating
- Legal Load Rating
- Permit Load Rating

The Design Load Rating is used to measure the performance of existing bridges in their present condition utilizing the LRFD design standards. In PA, it is based on the PHL-93 design vehicle loading and evaluates the Strength I limit state at inventory (STR I & IP) and operating levels (STR IA), plus the Service II limit state for steel bridges at inventory (SERV II) and operating (SERV IIA), and Service III limit state for prestressed concrete bridges at inventory (SERV III) and operating (SERV IIIA). MBE Article 6A.4.3.1 states that if the calculated Design Load Rating factor is above 1.0 at the inventory level, all AASHTO Legal Loads will be satisfactory and posting or strengthening is not necessary. Note that this does not apply to the PA Legal Loads. Be aware that the flowchart in MBE, Appendix A6A also does not apply. A Design Load Rating often includes a fatigue life check for steel members if there are Category C or worse fatigue-prone details present, refer to MBE Article 6A.6.4.1.

The Legal Load Rating is used to assist in determining whether a bridge should be load posted or strengthened through rehabilitation. Legal loads, by definition, are unrestricted vehicles that can cross bridges without the need of a permit if the bridge is unposted. All highway bridges in PA are to have a Legal Load Rating performed, not only bridges that have Design Load Ratings less than 1.0 as stated in MBE Article 6A.4.3.1. PUB 238 IP Article 3.2.2.2 lists the PA Legal Load vehicles (PUB 238 refers to these as the "PA Bridge Posting Vehicles") are to be used in lieu of the AASHTO Legal Load vehicles when performing a Legal Load Rating to determine if a bridge requires posting. Therefore, AASHTO Legal Load vehicles that can be ignored are the Type 3, Type 3S2, Type 3-3, and SU4 thru SU7. The FHWA FAST Act Emergency Vehicles EV2 and EV3 are to be included in the Legal Load Rating. Refer to PUB 238 IP Article 3.2.2.5 for more information on these FHWA Emergency Vehicles. The limit states for LRFR ratings include:

- Strength I limit state at inventory (STR I & IP)
- Strength II limit state at operating (STR II)
- Service I limit state for prestressed concrete bridges at inventory (SERV I)
- Service II limit state for steel bridges at inventory (SERV II) and operating (SERV IIA)
- Service III limit state for prestressed concrete bridges at inventory (SERV III) and operating (SERV IIIA)

When evaluating PA Legal Loads, a distinction is made between inventory and operating level ratings; unlike the MBE where only a single load rating factor is produced for the AASHTO Legal Load vehicles. Note that the FHWA FAST Act Emergency Vehicles are only to be evaluated at the operating level with a live load factor of 1.3 as outlined in the FHWA memo dated November 3, 2016.

The Permit Load Rating is used to check permit applications for the passage of overweight vehicles above the established weight limit. Regardless of the Legal Load rating factor, a Permit Load Rating is typically performed when evaluating a submitted permit load for PA highway bridges. MBE Article 6A.1.5.3 should be ignored where it states to only perform a Permit Load Rating if the AASHTO legal loads have a rating factor greater than 1.0. PA's permit vehicles include the P-82 and P2016-13, as detailed in DM-4 Article 3.6.1.2.7P. Be aware that the 0.2 klf partial lane load need not be applied to the P-82 permit vehicle, regardless of the bridge span length. Note that the P2016-13 does not apply when rating steel floorbeams. Permit Load Ratings in PA utilize the Strength II limit state for both steel and concrete bridges (STR II), the Service IIB limit state for steel bridges (SERV IIB), and the Service IIIA limit state for prestressed concrete bridges (SERV IIIA). All limit states for PA's permit vehicle ratings are at operating level.

Additional information on the load factors and live load vehicles for the Design Load Rating, Legal Load Rating, and Permit Load Rating, plus information on the applicable limit states can be found in DM-4 Tables 3.4.1.1P-1 through 3.4.1.1P-6.

#### 1.1.3.2 Load Factor Rating (LFR)

The LFR method is defined in the MBE Article 6B.3.2 and as supplemented by PUB 238 IE Article 6B. The LFR method analyzes a structure using actual loads with load factors applied to each load, depending on the uncertainty of that load. These load factors and limit states are defined in the MBE Article 6B.4.3. The factored loads are used to determine the factored forces (moment, shear, and axial) in a member, then compared to the capacity of the member to obtain a rating. In LFR, the dead load factor is 1.3 for both inventory and operating ratings. The live load factor on the other hand, is either 2.17 for inventory or 1.3 for operating. Unlike LRFR, the LFR method only considers the HS20 vehicle and not the PHL-93 for the NBI rating. Similar to LRFR though, the PA Bridge Posting Vehicles are to be rated at inventory and operating levels, while the FHWA FAST Act Emergency Vehicles are to be rated at operating only. The capacity of the member is defined in the AASHTO Std. Spec. with additional guidance in the MBE Article 6B.5.3, including a list of capacities for historical fasteners and a list of yield points for historical concrete reinforcing steel.

#### 1.1.3.3 Allowable Stress Rating (ASR)

The ASR method is defined in the MBE Article 6B.3.1 and as supplemented by PUB 238 IE Article 6B. The ASR method reflects traditional design methods that were predominantly in use prior to the adoption of the LFR and LRFR methods. The actual unfactored loads are combined to determine the maximum stress in a member due to moment, shear, and/or axial forces. The maximum stress is then compared to the allowable stress (maximum permissible) of the material to obtain a rating. There are no factors applied to the loads; in the general rating equation, set the load factors equal to 1.0 as defined in the MBE Articles 6B.4.1 and 6B.4.2. The allowable stress is dependent on the material and type of member being rated, the maximum stress type, and the rating level evaluated – inventory or operating. The allowable stresses for each rating level can be found in the tables and equations in MBE Article 6B.5.

#### 1.1.3.4 Other Rating Methods (Engineering Judgement, Load Testing, Assigned Load Ratings)

Engineering Judgement is a crude non-analytical rating method that relies on experience, an understanding of the load path, and familiarity of the current condition of the bridge. Engineering Judgement alone when performing a load rating is typically only permitted for structures that meet the requirements discussed in Pub 238 3.6.1.1 and Pub 238 Appendix IP 03-B. Further information on engineering judgement is discussed in Section 1.4 and applicable Sections of Pub 238.

Load testing or proof testing could be considered in rare cases when certain conditions exist that make conventional rating methods less reliable. Load testing should be used with caution as the test loads are high and damage to the bridge could occur. Additional information on load rating through load testing can be found in PUB 238 IE Article 8. The use of this method requires prior approval from the Assistant Chief Bridge Engineer.

An Assigned Load Rating uses the design loading to determine the rating, but only if the bridge was designed using LRFD or LFD to at least PHL-93 or HS-20 live loads. In general, this method is also only permitted if current field data shows that the condition of the structure has not changed from the as-designed or as-built condition, and there were no changes to the dead and live load. More detailed information on when this method is permitted by the FHWA is outlined in PUB 238 IP Article 3.6.1.2. In addition, PennDOT has provided responses to frequently asked questions about Assigned Load Ratings in a document located on the BMS2 homescreen under Bridge Inspection QA Clarifications. PennDOT requires the Assigned Load Rating Approval Form (PUB 238 Appendix IP 03-D) to be included in the bridge file when this rating method is used.

## 1.2 PENNDOT PROGRAMS FOR LOAD RATING

PennDOT has a suite of engineering programs developed to perform the analysis and ratings for different structure types. The use of standard engineering programs can optimize efforts during the analysis and checking stages and provide consistency in rating similar bridges. PennDOT Engineering Software are available for use by PennDOT personnel, external consultants, local governments, educational institutions and federal or state agencies. For information on the PennDOT engineering programs, see PUB 238 IP Article 3.8.

PennDOT LRFR (LRFD) programs get updated frequently and the changes reflect the updates in AASHTO LRFD and DM-4 codes. PennDOT LFR and ASD rating programs are only updated to address bugs within the program. The rating engineer should use the current version of these software programs. If the rating engineer determines the structure rating cannot be properly done using these programs, provide a proposal for a different software along with reasonable justification.

The rating engineer may use other software needed to calculate inputs or to post-process output, such as Mathcad or Microsoft Excel.

For more complex structures which are outside the scope of PennDOT engineering programs, and which require refined analysis, non-PennDOT programs can be used. Refer to PUB 238 IP Article 3.8 for a list of software and programs which have been accepted by the Department for use in projects to determine LFR and LRFR ratings. When using non-PennDOT approved programs, prior approval from the District Bridge Engineer shall be obtained. Other Load Rating Programs not included in this list may be utilized with prior consent of the Chief Bridge Engineer.

The rating engineer should understand basic assumptions and method of solution of the program being used. The output files should be provided clearly showing the member forces and member capacities to provide for verification of program output and for the purpose of using the data for future ratings. The rating engineer is responsible for compliance of the calculated ratings with current codes, proper application of the software and interpretation of the output. If the rating engineer encounters software issues which can affect the rating results, alternative calculations should be performed and documented in the Load Rating Report. If the rating engineer finds the PennDOT program being utilized for the rating is producing incorrect results or a technical question needs addressed, a technical question/revision request form should be completed and the form should be sent to the Bureau of Bridge via email. The technical question/revision request form(s) can be found at the end of PennDOT program user's manuals.

For a summary of recommended structural analysis programs for various structure types, see Table 1.2. Programs which are not included in the Approved list of programs or in this table can be used with the approval of the Chief Bridge Engineer.

**Table 1.2**
**Recommended Load Rating Software/Spreadsheets for Different Structure Types**

| Structure Type | PennDOT Software | | Acceptable Non-PennDOT Software |
|---|---|---|---|
| | LRFR | LFR/ASD | LRFR/LFR/ASD |
| Reinforced Concrete Beam | - | BAR7 | AASHTOWare Bridge Rating (BrR) (or other approved software) [a] |
| Prestressed Concrete I-Beam and Box-Beam | PSLRFD | PS3[c] | BrR (or other approved software) [a] |
| Steel Floorbeam System | FBLRFD | BAR7 | BrR (or other approved software) [a] |
| Steel Girder System | STLRFD | BAR7 | BrR (or other approved software) [a] |
| Girder-Floorbeam-Stringer Structures | - | BAR7 | BrR (or other approved software) [a] |
| Steel Box Girder | - | - | Approved Software [a] |
| Truss | - | BAR7 | BrR (or other approved software) [a] |
| Gusset plates | - | BAR7 | BrR (or other approved software) [a] |
| Curved and Highly Skewed Girders (Steel and Concrete) | - | - | BrR (or other approved software) [a] |
| Concrete Box Culverts & Frame Culvert | BXLRFD | BOX5 [c] | BrR (or other approved software) [a] |
| Concrete Slab Bridge | - | BAR7 | BrR (or other approved software) [a] |
| Metal Culvert | - | - | No current acceptable software |
| Reinforced Concrete Arch | | ARCH [c] | Approved Software [a] |
| Masonry Structure | - | - | No current acceptable software |
| Timber Beams | - | - | BrR (or other approved software) [a] |
| Decks | - | - | BrR (or other approved software) [a] |

[a] Refer to the online PennDOT document “Accepted Commercially Available or Consultant Developed Software”

[c] Program only rates in accordance with LFD Methodology

## 1.3 RESPONSIBILITY

Load rating of bridges in PA is ultimately the responsibility of the District Bridge Unit which is overseen by the Bridge Inspection Section (BIS) within the Bridge Office. Each District, or consultant, is required to have qualified engineer(s), under the direct supervision of a registered Professional Engineer registered in the Commonwealth of Pennsylvania, to perform or review bridge analyses and ratings. The qualified engineer(s) performing the rating is responsible for gathering the required documents, making the calculations, documenting all assumptions and methodologies, and completing the load rating package. At the completion of a load rating analysis, a Load Rating Summary Form (PUB 238 Appendix IP 03-C) is to be completed and signed and sealed by a registered Professional Engineer in PA. If performed by Department personnel, the "signing and sealing" engineer will be the District Bridge Engineer or the Assistant District Bridge Engineer for Inspection.

### 1.3.1 Computer Program Verification

Computer programs for load ratings should be utilized by a qualified engineer, but even then, errors can occur, especially with how the bridge is modeled. In certain situations, independent verification checks, such as rough hand calculations, may need to be performed by the rating engineer to verify the results of the computer program, including checks of the dead load reactions, shears, and moments are as expected. Most computer programs have user's manuals that outline the program's methods and assumptions that may aid in verifying the results. Standard shear and moment diagrams for single or multiple equal span length beams can be found in most structural engineering textbooks, including the AISC Steel Construction Manual, to aid in making verification check calculations by hand. The self-weight of secondary members can usually be spread out full length of the span with a per foot weight to simplify the calculations. Verification checks for live load effects are more involved, although for simple span bridges a standard table is available in PUB 238 Appendix IP 03-A. Capacities or resistances of the bridge members modeled in the computer program are also more involved to calculate by hand. Verification checks are better performed using previously confirmed spreadsheets developed specifically for the member type.

It is recommended to begin with the as-designed ratings because the original structure plans often provide information that can be used as a side check of inputs and verification of hand calculations/computer programs. The General Notes of the original structure plans often provide backup information for assumed dead loads and live load considered for design. Depending on when the structure was built and the structure type, total dead load, live load, and/or section properties may be provided on the original structure plans. It is good practice to compare calculated dead and live loads and section properties of as-designed ratings to those provided in the original structure plans. A list of PennDOT approved computer programs can be found in Section 1.2.

### 1.3.2 Checking

As part of a QA/QC process described in Section 1.11, a qualified checking engineer(s) is responsible for checking the calculations, methods, assumptions, applicability of computer programs, input data, and results, including verifying that all the necessary procedures were followed. Any errors found are to be corrected by the rating engineer and rechecked by the checking engineer until the rating package is error free. The checking engineer's name and date the checking was completed is to be documented on the Load Rating Summary Form provided in PUB 238 Appendix IP 03-C. More discussion on proper documentation of load rating evaluations is provided in Section 1.12.

## 1.4 ENGINEERING JUDGEMENT

Engineering judgement alone to rate a bridge is sometimes needed when original drawings are unavailable and member sizes and critical items (e.g., reinforcement details) cannot be determined by field measurements, such as the case with older concrete and masonry structures.

Engineering judgement alone should not be used to load rate steel structures, except concrete encased steel beams. Exposed steel member sizes and dimensions can be easily determined from field measurements which rules out using engineering judgement. A qualified engineer who is knowledgeable with the structure type and is familiar with its current condition, may use engineering judgement to rate a redundant concrete or masonry bridge with unknown structural components preventing the need for a more accurate analytical method to determine the live load ratings.

Engineering judgement load ratings do not require formal sets of calculations or computer software analyses. The load rating factors should be conservative lower bound rough estimates with ratings in tons to the nearest integer. For posting evaluations, weight limits determined by engineering judgement should be in 5-ton increments as recommended in PUB 238 IP Article 4.4.2. If a posting weight limit is less than 15 tons, it is recommended to use 1-ton increments to avoid the exclusion of school buses, fire trucks, or ambulances. When using engineering judgement, it is useful to know the year of original construction to provide a rough estimate of the intended as-built design capacity, without deterioration, using the design vehicle of that era. Older bridges may only have been designed for H10, H15, or H20 vehicles.

Information on when and how to use engineering judgement for concrete and masonry structures is provided in PUB 238 IP Article 3.6.1.1. Additional direction on when to use engineering judgement is outlined in PUB 238 IE Article 6.1.4 and the MBE Article 6.1.4. There are two methods that can be utilized to complete an engineering judgement. These methods are discussed in Pub 238 Appendix IP 03-B. Method 1 can be used to rate simple span concrete structures with lengths from 8 ft. to 50 ft., which meet the criteria specified in Appendix IP 03-B. Method 2 can be utilized for bridges, which require an engineering judgement, but do not meet the requirements of Method 1.

## 1.5 DATA USED FOR LOAD RATING ANALYSIS

A collection of accurate information is required to perform a dependable and true load rating. This includes field inspection reports, existing plans, as-built drawings, design drawings, shop drawings, rehabilitation and repair drawings, in some instances field measurements, and other records such as material testing reports. Conflicting data between the different sources may occur. It is the responsibility of the rating engineer performing the rating to determine the reliability, accuracy, and applicability of all the data used to perform the load rating.

### 1.5.1 Field Inspection Report

One of the most useful pieces of information for a load rating is field inspection report data. Field data shall supersede plan data if there is a discrepancy. The field inspection documents describe the current condition of the structure and its components with measurements, photos, and notes as outlined in the MBE Article 4.3.8. The most recent inspection data should be used to perform a load rating, especially for older bridges with deterioration. The engineer performing the load rating should consider measurements, where applicable, for the deterioration of the existing material such as structural steel section loss, reinforcing steel section loss, broken prestressing strands, and deteriorated (spalled, delaminated, etc.) concrete. These conditions may result in a lower capacity and consequentially lower rating of the member. Other useful information that is collected during a field inspection includes verification of existing bridge records and, for example, measurements of the actual wearing surface thickness. Changes to the bridge are also usually noted in the inspection reports, such as rehabilitation and repair work performed, presence of new utilities, etc. Field inspection data, including inventory data can be found in Pennsylvania’s BMS2 database.

### 1.5.2 Existing Plans

Existing plans and standards, including as-built drawings and design drawings, provide the engineer with basic data of the structure such as span lengths, width, lane configuration, size and general dimensions of members and connections, plus material specifications in the general notes. A load rating should not be performed solely on the existing plans if rehabilitations or changes to dead load have occurred since the original construction of the bridge. If shop drawings are available (see Section 1.5.2.3), information on those will likely supersede some of the information on the existing structure plans.

#### 1.5.2.1 As-Built Drawings

As-built plans show the structure as it was constructed. Deviations from the design drawings are typically noted in the as-built drawings and should be used in-lieu of the design drawings when available.

#### 1.5.2.2 Design Drawings

Design drawings, also referred to as the Contract Plans, can be used if the as-built drawings are not available, although field verification of member sizes, geometry, etc. is recommended. The design drawings should also include as-designed load ratings which can be used as an independent check, but not as a substitute to performing new rating calculations unless the design utilized the LRFD method. Also note, design drawings for historic bridges, may not have had a load rating performed or if it was, it may have used outdated rating vehicles and the rating method was likely ASR or LFR.

PennDOT BC standard details are not included on the design drawings but are instead noted as references. Be aware that the BC standards are frequently updated, when using them for a load rating evaluation, archived BC standards are available that match the year the bridge was initially constructed.

#### 1.5.2.3 Shop Drawings

When available, shop drawings tend to be reliable documents to determine what was constructed in the field. Shop drawings for prestressed concrete beams and steel structures provide more detail and often relay information not shown on original contract plans. Particularly, shop drawings are important for evaluating connections and gusset plates. Typically, the shop drawings supersede the contract plans where they differ as they better represent the as-built structure.

#### 1.5.2.4 Rehabilitation and Repair Drawings

Rehabilitation and repair design (contract plan) drawings may provide information on past or recent alterations to the bridge, including dead load and live load changes due to new deck overlays or wearing surfaces, deck replacement, structure widening, barrier changes or additions, sidewalk additions, beam changes from non-composite to composite, etc. Changes to the structural capacity of the bridge would also be evident in rehabilitation and repair drawings if existing beams were strengthened or new beams were added. As-built and/or shop drawings for rehabilitation and repair projects may also be available that would provide more accurate and detailed information than the rehab/repair design drawings. Also check the inspection reports to see if changes to the bridge were noted, and for confirmation that repairs were performed.

### 1.5.3 Other Records

Other structure records may exist that will provide additional information pertinent to the load rating. These records may override specifications or measurements that are reported in the construction plans or repair plans. Examples include:

- Prior Ratings, and posting history

- Field Testing Reports - Non-destructive and destructive testing of steel, concrete, and timber materials, strain gage or load cell testing, etc. (see MBE Article 5).

- Maintenance History

- Steel Mill Reports – For steel beams, plates, angles, reinforcing bars, etc., certification reports from steel mills note the chemical and physical properties of the supplied steel through testing, which are performed to certify that the supplied steel meets specific ASTM and AASHTO requirements as required by the Contract Plans and Pub 408. The mill reports will indicate the measured yield and ultimate strengths of several samples which are used to certify the steel at a standard grade/strength. It is not recommended to use these larger measured strengths than what the steel was ultimately certified at, and if the certified strength is greater than the strength initially utilized in the design, it is recommended to use the design strength in the load rating.

## 1.6 LOADS

In most cases, dead loads and live loads are the only loads that need to be considered to develop a load rating. Environmental loads are not typically considered in ratings. Refer to MBE Articles 6A.2.3 or 6B.6.7 if unusual conditions are encountered that justify their inclusion.

### 1.6.1 Dead Loads

Calculate the dead load effects based on current conditions of the bridge at the time of analysis. Verify current conditions with inspection reports and field measurements. Include locked-in force effects from construction.

Future dead loads such as FWS are not required for load rating an existing bridge as specified in PUB 238 IP Article 3.2.1. FWS is typically only for the design.

Latex modified concrete overlays should be considered structurally effective if requirements in DM-4 Article 5.5.5.1 are met.

Unless more precise information is available, use the dead load unit weights given in the current AASHTO LRFD Table 3.5.1-1 and as modified by DM-4 Article 3.5.1 and AASHTO Std. Spec. Article 3.3.6 as applicable. For unit weights of materials not provided in the AASHTO Std. Spec., refer to DM-4 and AISC.

For guidance on distribution of DC2 dead loads to girder and box beam structures for LRFR ratings, see DM-4 Article 3.5.1.1P. Distribute utility loads in a manner that best represents the force effects on supporting bridge components.

Include secondary effects from post-tensioning applied to statically indeterminate structures.

For typical steel multi-girder structures, account for additional DC1 dead loads such as stiffeners, splices, bolts, connections, etc. through calculations based on plan dimensions unless otherwise measured. When the information is not available, determine the additional dead load in a rational way. Typically, this is done by increasing the self-weight by a determined percentage. The percentage used should be specific to the project and may require input from PennDOT and in some cases may be deemed an unacceptable method of determining these dead loads. Typically, 10% is a reasonable estimate of the extra dead load.

Fully document dead loads used in the rating analysis as specified in PUB 238 IP Article 3.2.1.

Refer to MBE Articles 6A.2.2.1 and 6B.6.1 for additional information.

### 1.6.2 Live Loads

The NBIS requires all states to load rate their bridges "to provide a uniform measure of live load capacity of the nation's bridges in the NBI for planning and programming purposes and to ensure bridges are properly posted for legal load configurations used in individual states" according to PUB 238 IP Article 3.2.2.1.

For live load NBI planning purposes, the HS20 has traditionally been used for ASR/LFR ratings at inventory and operating levels. When the LRFD method was implemented, Pennsylvania developed the PHL-93 for LRFR at inventory and operating levels. Older bridges were designed using older AASHTO Specifications and frequently do not have the capacity of a modern designed bridge. If an existing bridge has an LRFR inventory rating below 1.0, it has no regulatory impact on the bridge operations and is of no specific concern. As mentioned above, it is used to report the bridge load rating to the NBI.

Pennsylvania has state specific legal loads that are used to determine the need for posting a bridge. Along with the legal loads, the H20 and HS20 loadings are also used for posting purposes. All posting loads should be evaluated at inventory and operating levels according to PUB 238 IP Article 3.2.2.1. The emergency vehicles are only required to be evaluated at operating levels according to FHWA Memo HIBS-1 and are not evaluated for posting.

Permit loads should be evaluated at the operating level only.

Refer to Section 1.1.3 for more information.

#### 1.6.2.1 Design Load: LL

The Design Load rating generally produces the largest load effects and is commonly considered the first level of load rating. Bridges with inventory rating factors greater than 1.0 typically have adequate capacity for legal loads.

The Design Load for an LRFR rating is the PHL-93 and consists of a combination of the AASHTO design truck or design tandem and the AASHTO design lane load. Refer to AASHTO LRFD Article 3.6.1.2. The PHL-93 tandem modifies the AASHTO design tandem axle weights from 25 kips to 31.25 kips. The PHL-93 is considered a notional load and only axles that contribute to the desired force effect are included. The PHL-93 truck, tandem, and lane load can be combined and modeled in several different ways to produce the largest force effects. Refer to DM-4 Article 3.6.1.3.

The Design Load for an ASR/LFR rating is the HS20 and it consists of either the AASHTO Std. Spec. design truck or design tandem or the design lane load, whichever controls for the desired force effect. Refer to AASHTO Std. Spec. Article 3.7.

#### 1.6.2.2 Legal Load: LL

The Legal Loads regardless of rating method used are the H20, HS20, ML-80 and TK527. The legal load ratings are used to determine bridge postings and are based on the bridge's SLC. If the SLC is less than the gross vehicle weight of the legal load, the bridge should be posted or restricted as discussed in Section 1.1.2.1.

The H20 and HS20 with axle weights and spacings for the design truck, design tandem, and design lane load can be found in AASHTO Std. Spec. Article 3.7. For additional information and posting considerations, refer to PUB 238 IP Article 3.2.2.2.

The ML-80 and TK527 with axle weights and spacings for the Legal Load can be found in DM-4 Articles 3.6.1.2.8P and 3.6.1.2.9P, respectively. It should be noted that the axle weights shown in DM-4 include a 3% increase for tolerance and that the GVW used to determine the rating tonnage is 36.64 tons for ML-80 and 40 tons for TK527. Both the ML-80 and TK527 are not considered notional loads and all axles are considered when determining force effects. For additional information and posting considerations, refer to PUB 238 IP Article 3.2.2.2.

PUB 238 IP Articles 3.2.2.3 and 3.2.2.4 states that AASHTO Typical Legal Loads (e.g., Type 3, Type 3S2 and Type 3-3 vehicles) and AASHTO Special Hauling Vehicles (e.g. SU4, SU5, SU6, and SU7) are not required to be included for rating or posting because studies found that PA's posting vehicles have greater live load effects than those produced by these vehicles.

FHWA FAST Act Emergency Vehicles (e.g., EV2 and EV3) are required to be evaluated in accordance with the implementation of the FAST Act. They are to be evaluated for the load rating as discussed in PUB 238 IP Article 3.2.2.5, but are not to be used for posting evaluations. The axle weights and spacings for the EV2 and EV3 can be found in FHWA Memo HIBS-1.

#### 1.6.2.3 Permit Load: LL

Permit load ratings are generally performed to determine the ability of a bridge to safely carry overweight permit vehicles and are not concerned with serviceability. They are also used to help identify future bridge repair and bridge improvement projects. For permit load ratings below 1.0, see Section 1.1.2.1 for information on posting and restriction requirements.

The Design Permit Loads are the P-82 and P2016-13 and are applied using the operating rating factors similar to the Strength II load combination. For guidance on which bridge components the Strength II load combination applies to, see DM-4 Article 3.4.1. Axle weights and spacings for the Design Permit Loads can be found in DM-4 Article 3.6.1.2.7P.

It should be noted that although both the P-82 and P2016-13 are considered notional loads (i.e., axles that do not contribute to maximizing force effects are to be neglected), consider all axles for the P2016-13 regardless of the impact on the force effect under consideration.

#### 1.6.2.4 Dynamic Load Allowance (Impact): IM

LRFR Ratings:

Dynamic Load Allowance (Impact), as defined in AASHTO LRFD Article 3.6.2.1, is a factor applied to the static live load as: (1+Impact/100). Impact is not applied to centrifugal and braking forces nor is it applied to pedestrian load. Impact is not applied to the PHL-93 lane loads, but it is applied to the H20 and HS20 lane loads.

For components for which Impact is applicable, see DM-4 Article 3.6.2.1.1P and for components for which Impact is not applicable, see DM-4 Article 3.6.2.1.2P and AASHTO LRFD Article 3.6.2.

The Impact allowance for an LRFR rating is computed in accordance with LRFD and DM-4 Article 3.6.2. The impact applicable to structural components is not to exceed 33% for the Design and Legal Loads and to not exceed 20% for Permit Loads. Deck ratings should use 50% for impact. Impact for buried components (e.g., culverts) can be calculated per DM-4 Article 3.6.2.2. Impact may be reduced for legal loads if the conditions specified in MBE Article 6A.4.4.3 are satisfied. For culverts, impact may be reduced according to MBE Article 6A.5.12.10.3b. For discussion of reducing the impact factor for permit loads, reference PUB 238 IP Article 10.3.4. For the design loads, no reduction in impact should be permitted unless justification is provided through AASHTO LRFD Article 4.7.2.1.

ASR/LFR Ratings:

Impact allowance for an ASR/LFR rating applicable to structural components is computed by AASHTO Std. Spec. Article 3.8.2.1 Equation 3-1 (Impact = 50 / (L +125) but cannot be greater than 30%) for the Design and Legal Loads. For Permit loads, impact should be calculated using the same equation, but cannot exceed 20%. Apply the loaded length (L = portion of span loaded) in accordance with AASHTO Std. Spec. Article 3.8.2.2. Deck ratings should use 50% for impact. The calculated impact may be reduced if conditions specified in MBE Article 6B.6.4 are met. For discussion of reducing the impact factor for permit loads, reference PUB 238 IP Article 10.3.4.

#### 1.6.2.5 Pedestrian (Sidewalk) Live Load: PL

The Pedestrian (Sidewalk) Live Load is distributed through the superstructure members in the same manner as the sidewalk dead load. See DM-4 Article 3.5.1.1P.

For LRFR ratings, follow guidance provided in MBE Article 6A.2.3.4 and refer to DM-4 Article 3.6.1.6 for a discussion of what pedestrian load to apply, to apply this load only to sidewalks wider 2.0 ft., and for discussion of using the Strength IP load combination.

For ASR/LFR ratings, follow guidance provided in MBE Article 6B.6.2.4 and refer to AASHTO Std. Spec. Article 3.14.1 for sidewalk loading applied only to sidewalks wider than 2.0 ft. Since it is unlikely that the maximum vehicle and sidewalk live loads will act simultaneously on the bridge, it is appropriate to evaluate this case at the operating level.

For guidance on how and when to apply the Pedestrian (Sidewalk) Live Load, see Section 2.3.1.

### 1.6.3 Environmental Loads

In accordance with MBE Article 6B.6.7, environmental loads mentioned in this Section of the MBE need only to be applied at the operating level.

#### 1.6.3.1 Wind Loads: WL and WS

Wind loads are not typically considered in load ratings. Exceptions would be load ratings for unique or complex structures such as movable bridges, long-span bridges, suspension bridges, and other high-level bridges. Refer to MBE Articles 6A.2.3.5 and 6B.6.7.1 as applicable and DM-4 Article 3.8.

#### 1.6.3.2 Temperature Effects: TG and TU

Temperature effects are generally not considered in calculating load ratings when reinforcement is well-distributed to control thermal cracking. These effects may need to be considered for segmental bridge components and complex structures such as long-span, framed, and arch bridges. Refer to MBE Articles 6A.2.3.6 and 6B.6.7.3 as applicable and DM-4 Article 3.12.

#### 1.6.3.3 Creep and Shrinkage: CR and SH

Creep and shrinkage effects are generally not considered in calculating load ratings when reinforcement is well-distributed to control cracking. These effects may need considered for segmental or prestressed bridge components and complex structures such as long-span, framed, and arch bridges. Refer to MBE Articles 6A.2.3.8 and 6B.6.7.3 as applicable and DM-4 Article 3.12.

#### 1.6.3.4 Earthquake Effects: EQ

Earthquake effects should not be considered in calculated load ratings for structures design in Pennsylvania. Refer to MBE Articles 6A.2.3.7 and 6B.6.7.2 as applicable and DM-4 Article 3.10.

#### 1.6.3.5 Earth Pressure: EV, EH, and ES

Earth pressures are typically considered for buried structures such as reinforced concrete box and arch culverts. Refer to MBE Article 6A.5.12.10.2 and DM-4 Article 3.11.

#### 1.6.3.6 Live Load Surcharge: LS

Earth pressures are typically considered for buried structures such as reinforced concrete box and arch culverts. Refer to MBE Article 6A.5.12.10.3c and DM-4 Article 3.11.6.

#### 1.6.3.7 Braking and Friction Forces: BR and FR

Braking and friction forces are typically considered for load rating a substructure. Refer to DM-4 Articles 3.6.4 and 3.13, respectively. For how to apply these loads to substructures, see MBE Article 6.1.5.2. However, do not apply these longitudinal loads as specified in PUB 238 IE Article 6.1.5.2.1I to reinforced concrete pier caps.

## 1.7 ANALYSIS METHODS

Several analysis method options are available for a load rating engineer. Basic guidance on choosing the appropriate method is provided in this Section. A single method of analysis will likely be sufficient for the entire bridge load rating evaluation, although several analysis methodologies and levels of refinement may be investigated during the analysis to determine load ratings that are most representative of the as-inspected/as-built structure as discussed below in Section 1.7.1. The chosen analysis methodology and assumptions should be consistent throughout the analysis and clearly stated. The analysis methodology used for the design rating vehicle evaluation should also be used for legal and permit load evaluations, except as discussed in Section 1.7.1.4 below where a poor rating could be re-analyzed with a more refined method.

Members to consider for load rating analysis should be identified (see Section 2.4) and analyzed for all applicable limit states and vehicles to determine which members control the load rating.

Simplified analyses and PennDOT-approved programs should be used to develop load ratings because, for the majority of PennDOT bridges, they will produce consistent and reliable load ratings. Refer to Section 1.2 for a list of PennDOT-approved programs to use in load ratings and Section 1.7.1 for a discussion on levels of analysis. For those unique situations where simplified analyses and PennDOT-approved programs are not applicable or an improved load rating is warranted, a refined analysis should be considered with District, Central Office, or Owner coordination. Section 1.7.1.4 provides more discussion on situations that warrant refinement of analysis.

### 1.7.1 Levels of Analysis

The typical default method of analysis for any load rating is a simplified line girder analysis. For complex structures or if the bridge load ratings are low, a refined method of analysis may be appropriate. A refined analysis will generally provide a more accurate distribution of loads which may increase the controlling load rating factor.

For steel I-girder bridges, DM-4 Appendix Table E6.1.3.1P-1, provides guidance on the appropriate level (or type) of analysis to use based on the skew, skew index (see DM-4 Appendix E6.1.2.1P), and whether the bridge is curved or not. For other bridge types, see guidance provided in the Sections below. The Connectivity Index (see DM-4 Appendix E6.1.2.2P and AASHTO LRFD Article 4.6.3.3.2) and Amplification factor (DM-4 Appendix E6.1.2.3P) should also be evaluated to determine the appropriate level of analysis.

#### 1.7.1.1 Types of Analyses – 1D, 2D & 3D Refined

As a way of describing the level of refinement in an analysis, the analysis is generally expressed as a 1D, 2D, or 3D analysis. The D refers to the system dimensions and does not always correspond to the element dimensions. For example, it is common for a 2D analysis to use 3D elements.

*Simplified Analysis.* A 1D simplified analysis uses the empirical method to analyze each member as a single non-interacting element (i.e., does not take into account the overall system behavior of the structure) and requires one spatial coordinate to define results. For a curved bridge, this coordinate (i.e., dimension) is measured along the curved axis.

Dead load force effects for 1D simplified analysis are typically calculated using the tributary area.

A 1D simplified analysis is typically performed using the lever rule or the DM-4 and AASHTO distribution factors to determine live load distribution. The lever rule can be used for both longitudinal and transverse members as discussed in PUB 238 IP Articles 3.3.1 and 3.4.1, respectively.

Note that for longitudinal exterior girders, when utilizing the lever rule, moments should be summed about the first interior beam to compute the distribution factor for the exterior girder to increase the distribution factor effect from the outside wheel above 1.0. The reason for this conservative approach is the force from the wheel outside the centerline of the exterior girder will introduce some torsional stresses into the beam, which are not accounted for in the standard PennDOT programs.

Note that for transverse members that directly support the deck without help from longitudinal member, the load rater should follow guidance provided in PUB 238 IP Article 3.4.3.1 for applicability of the DM-4 and AASHTO distribution factors. For all other transverse members, DM-4 and AASHTO distribution factors do not apply and lever rule or a refined analysis should be considered to distribute live load. The application of the DM-4 and AASHTO distribution factors for transverse members that directly support the deck should use DM-4 and AASHTO distribution factors as discussed in PUB 238 IP Articles 3.4.3.2 for ASR and LFR. For LRFR method, follow PUB 238 IP Article 3.4.3.3.

The DM-4 and AASHTO simplified line girder analysis distribution factors for rating evaluations of longitudinal members can be used when the conditions specified in PUB 238 IP Article 3.3.2.1 are met. For additional line girder analysis guidance, see DM-4 Appendix E6P. Curved steel I-girder and closed box and tub girder bridges meeting the requirements of AASHTO LRFD Articles 4.6.1.2.4b and 4.6.1.2.4c respectively, are also permitted to be analyzed using the DM-4 and AASHTO simplified line girder analysis distribution factors if their original designs were of that method. The application of the DM-4 and AASHTO distribution factors for longitudinal members is discussed in PUB 238 IP Articles 3.3.2.2 for ASR and LFR. For LRFR method, follow PUB 238 Article 3.3.2.3.

For additional LRFR approximate methods of analysis, see MBE Article 6A.3.2. Also refer to Section 2.2.1 for guidance on load rating non-standard bridge configurations.

*Refined Analysis.* A refined analysis may be required for bridges with significant skew, horizontal curvature, splayed (i.e., variable spaced) girders and other non-standard bridge configurations, if empirical distribution of live load effects cannot accurately approximate the true live load effects, if severe defects or section loss are found (i.e., severe deterioration, severe damage from impact, failed or fractured members), or if it is determined that a 1D simplified analysis is not appropriate for the structure for any of the reasons discussed above.

To determine when a refined method of analysis is required for skewed, curved, or splayed beam bridges, see PUB 238 IP Articles 3.3.3.1, 3.3.3.2, or 3.3.3.3, respectively and DM-4 Appendix E6P. Also refer to Section 2.2.1 for guidance on load rating non-standard bridge configurations. To determine when a refined analysis is required due to severe defects or section loss, see Section 2.5.1.2 (for concrete components), Section 2.6.1.2 (for steel components), Section 2.7.1 (for wood components), and Section 2.8.1 (for masonry components).

A refined method of analysis can be an enhanced 2D analysis or 3D refined finite element analysis, a geometric nonlinear analysis, or an approved analysis method in accordance with DM-4/AASHTO LRFD

Articles 4.4 and 4.6.3. For all refined methods of analysis, the level of mesh refinement should be determined by the load rating engineer. Some general guidance for I-girder bridges is provided in DM-4 Appendix E6.1.1P. Additional general guidance is provided in Adams et al. (2019). Ultimately, the load rating engineer is responsible for properly implementing the analysis method and correctly interpreting the results, no matter which method is selected.

Enhanced 2D refined analyses are a finite element method using two spatial coordinates to define results. Examples include an enhanced grid (or grillage) analysis and an enhanced PEB analysis. Both enhanced methods include improvements to their traditional counterparts as is discussed in DM-4 Appendix E6.1.1.2P which also includes details of additional modelling characteristics of the enhanced grid and PEB analysis methods. For most typical concrete slab bridges, a PEB analysis is the recommended approach. Guidance for improved modeling of cross-frame stiffness and improved modeling of the torsional stiffness of I-girders can be found in DM-4 Appendix E6.2P.

A 3D refined finite element analyses also use the finite element method, but it explicitly models the individual member components (i.e., girder web and flanges) and requires three spatial coordinates to define results. Modeling in 3D is well suited for analyses that require more precise behavior of the structure and for sub-models, some examples include:

- Phased construction or post-tensioning that causes locked-in stresses
- Complex global or local stability analyses
- Complex dynamic analyses
- Local analyses such as details where stress concentrations are anticipated
- Analyses with significant flange lateral bending
- Analyses that consider warping torsional stiffness

A 3D linear elastic finite element analysis is a first-order analysis that assumes all components to behave elastically, hence the name. The model does not consider incremental load. For guidance on modeling a 3D linear elastic finite element analysis and further discussion on how force effects are determined, see DM-4 Appendix E6.1.1.3P for I-girder bridges. For additional general guidance, see Adams et al. (2019).

A 3D geometric nonlinear finite element analysis is a second-order analysis that assumes all components to behave elastically, but it uses incremental loading to more accurately account for large deflection responses. This method of analysis is typically only used when large second-order deflection amplifications are anticipated as is discussed in DM-4 Appendix E6.1.1.4P for I-girder bridges and approximate methods such as AASHTO moment magnification will not provide an accurate representation of the second-order effects. For additional general guidance, see Adams et al. (2019).

For a list of approved software for live load ratings, see PUB 238 IP Article 3.8 and Section 1.2 of this Manual.

#### 1.7.1.2 Common Mistakes

Analysis of a structure requires a thorough understanding of the behavior of the structure and how the plans, field measurement, and other data obtained can be used to produce an accurate representation of the actual condition of the structure. Either a simplified analysis or refined analysis typically requiring the creation of a model is required to determine the structure's ability to carry live loads. Below is a list of common mistakes that the load rater should be aware of so that these mistakes can be avoided during the development of the load rating analysis.

- Verifying that data is input in the analysis model correctly may be the most common mistake made in developing a load rating analysis. Ensure the following types of inputs are correct:

    - Units for loads, distances, etc. (e.g., psi vs. ksi, lbs vs. kips, inches vs. feet, English vs. Metric, axles vs. wheels)
    - Direction of applied loads (e.g., vertical vs. transverse, up vs. down)
    - Orientations of component attributes in local coordinate system such as section properties (e.g., Sx vs. Sy)
    - Orientation of components in global coordinate system
    - Using software default values without verifying if they are appropriate
    - Review program assumptions and limitations, since each program varies

  Not being careful with inputting units or orientation can result in severely inaccurate structural models and load rating values.

- Determining physical properties such as material strength can have a significant impact on an analysis. It is a common mistake to use incorrect material strength in the analysis which could result in either conservative or unconservative results if the strength selected is inaccurate. For guidance on how to best determine physical properties, see Section 1.10.

- Using field measured data for components such as wearing surface thickness, deck thickness, member thickness, etc., can significantly increase load effect accuracy. It is a common mistake to assume all information provided on the plans is correct. Field measuring components will increase analysis accuracy.

- Interpreting member or structure condition is critical to obtaining accurate analysis results. It is common for section loss or defects to be under-estimated or over-estimated in the development of the member as-inspected section properties which results in inaccurate load ratings. In some cases, defects and section loss can be accounted for by adjusting the analysis model or capacity/resistance calculation instead of simply applying section loss. Examples include:

    - Severely deteriorated beams or truss eyebars may become ineffective and should be removed from the analysis model. Note, the deck may need rated if a beam is considered ineffective and removed from the analysis.

  - Built-up truss members with loss of one of the built-up components or deterioration of lacing bars or batten plates may change the member element unbraced length and/or stiffness. Note that excluding an outstanding leg of a built-up truss compression member component that has reduced thickness due to section loss, may improve the capacity/resistance if local buckling of that outstanding leg was controlling.

  - Member distortion may induce second order effects.

  - Impact damage could permanently deform a member which may change the member shape and/or load carrying capacity that should be used in the analysis.

  For guidance on how to best account for severe defects or section loss in an analysis, see Section 2.5.1.2 (for concrete components), Section 2.6.1.2 (for steel components), Section 2.7.1 (for wood components), and Section 2.8.1 (for masonry components).

- Applying the correct PennDOT standards, software, and rating methodology to the analysis is essential to obtaining the desired results. Ensure the PennDOT standards used fit the time-period that the structure was designed. Past archived PennDOT standards may have significant differences to the current PennDOT standards. The software used for the analysis should match the level of refinement appropriate for the structure type as discussed in Section 1.7.1.1. The software selected should also use the appropriate load rating methodology as is discussed in Section 1.1.3. If the software is insufficient to simulate the behavior of the structure, the results will not be reliable. Conversely, selecting a more robust software than required may add unnecessary time to the load rating analysis development.

- Modifying poor analysis results based on good condition ratings to avoid posting is a common engineering judgement pitfall. A bridge component may not show signs of distress due to a number of reasons such as that it has not yet experienced the full design load. Poor load ratings should not be modified, overlooked, or ignored simply because the member condition looks fine.

- Applying skew correction factors only to 1D simplified analyses such as a line girder analysis helps account for increased force effects due to skew that a simplified analysis overwise would not be able to consider. Be aware that skew correction factors discussed in DM-4/AASHTO LRFD Article 4.6.2.2.3c are not applicable to enhanced 2D analyses or any other refined 3D finite element analyses because the system behavior of the model should distribute the load appropriately to account for the skew. It is a common mistake that the skew correction factor is applied to the 2D and 3D models and is essentially double counted resulting in overly conservative load effects. Skew correction factors are typically applied to the LLDF's. Similar to skew correction factors, LLDF's should not be applied to 2D and 3D models as the software used to develop refined models will distribute the loads based on the stiffness of the structure. For more information on how and when to apply LLDF's, see Section 2.13.1.

- Applying boundary conditions that represent the structure member connections locally and represent the superstructure's bearing supports and substructure's foundational supports globally are essential to accurately represent the structure behavior in the analysis model. Below are some considerations for the load rater to consider when defining and applying boundary conditions:

- Ensure the orientation of fixity in direction is correct. This can have a large effect on load rating results if the orientation is misaligned, particularly for supports of bridges with complex geometry such as curved and skewed structures.

- Modeling a continuous beam as simply supported or defining fixed floorbeam connections when they are pinned, will drastically change force effects and load rating results. Actual supports are never fully fixed or fully pinned due to friction and limited stiffness of connected elements, but approximating them as such is typically sufficient. Exceptions would be significantly skewed or curved structures, where defining supports with spring stiffness rather than unyielding supports may provide more realistic force effects. For these structures, modeling the bearings as rigid can cause horizontal force effects to be artificially large. For further guidance, see Adams et al. (2019).

- At supports with guided expansion bearings, ensure proper restraints are defined.

- The movements in the analysis that are permitted by the defined boundary conditions should be confirmed with field measurements. This can help shed light on incorrect boundary condition assumptions in the model.

- For refined analysis programs, ensure the model is stable. This is achieved when boundary conditions prevent rigid body motion of the model in any of the 6 degrees of freedom. Even though the model is unstable, some programs will still run and produce noticeably inaccurate force effects and deflections.

#### 1.7.1.3 Load Sensitivity

In a load rating analysis, it is beneficial to know how sensitive the rating factor results are to changes in loading. A sensitivity analysis may be required to determine an upper and lower bound of possible results based on changes to assumptions such as load distribution, load placement, etc. To conduct a sensitivity analysis, change one variable (e.g., unit weight of a material where information is not available, thickness of an overlay, load distribution, design speed, live load placement, impact, etc.) and keep all other variables fixed. See how changing that variable affects the ratings. Then, move onto the next variable and follow the same procedure to better understand what variables have the most effect. If changing a variable significantly affects the controlling ratings, the ratings are said to be “sensitive” to that variable.

#### 1.7.1.4 Refinement of Analysis

If the load rating results differ significantly from the design load ratings or if load ratings indicate that the bridge will require restrictions or posting, refinement of the analysis should be considered. First, verify that the analysis has been thoroughly checked for overly conservative assumptions and errors in the calculations and in the software. Then, scrutinize any variables discussed in the Section above that are determined to be “sensitive” to the rating results to ensure that the loads are not artificially high and confirm that they properly represent the behavior of the structure. A load rating engineer licensed in PA should verify where conservative assumptions have been refined in the analysis to achieve a rating above 1.0 and these assumptions should be included in the load rating summary report.

Refining the analysis will generally provide a more accurate distribution of loads which may increase the controlling load rating factor. For guidance on refining the method of analysis for bridges with low ratings using simplified methods and for non-standard bridges that have no applicable live load distribution formulas that produce reliable results, see MBE Article 6A.3.3 and Section 1.7.1.1. When refined analysis is performed, include a table of live load distribution factors for the controlling limit state in accordance with MBE Article C6A.3.3.

MBE Article 6A.3.4. provides general guidance for performing a rating analysis using field testing methods. PUB 238 IE Article 5 provides specific guidance on material testing, and PUB 238 IE Article 8 provides specific guidance on load testing. The following scenarios can justify use of the empirical evaluation methods such as material testing and load testing:

- If the load rating engineer has reason to believe the analysis cannot accurately represent the actual behavior and load distribution of the bridge, load testing may be justified.

- If there is doubt about the material properties assumed in the analysis, material testing may show that the capacity is greater than assumed in the initial analysis.

Refining the analysis method, load testing, or material testing should not be performed without prior approval of the Assistant Chief Bridge Engineer - Inspection.

## 1.8 LOAD RATING METHODS

The acceptable methodologies used to load rate a structure for PennDOT are LRFR, LFR and ASR. For descriptions of these methodologies and general guidance on how they are used in a load rating, see Section 1.1.3. The Sections below discuss how a load rating is performed for a specific purpose such as the load rating of a new, existing, or repaired or rehabilitated structure. In addition, guidance is provided on how to properly include the safe load capacity in the posting evaluation.

### 1.8.1 New Structures

New structure load ratings should be included with the design computations and placed on the structure plans before construction. The load ratings should be determined using the design and analysis methods used to design the structure, unless modified per direction of the Chief Bridge Engineer, and conform to the requirements of DM-4, PUB 238 and this Manual as applicable. Two sets of rating calculations should be provided: (1) with consideration of an FWS in the ratings and (2) without FWS in the ratings. See DM-4 PP Article 1.8.3 for further guidance and sample load rating chart. All rating factors calculated should be 1.0 or greater for new structure designs.

Load ratings should be provided for the design loads, legal loads (including the emergency vehicles) and permit loads. According to FHWA Memo HIBS-1 dated November 2016, the emergency vehicles can be considered in a single lane with the other unrestricted legal loads in adjacent lanes if this produces favorable ratings. See Section 1.8.2 below for more information.

In conjunction with the structure initial inspection at completion of construction, the structure load ratings should be reviewed and updated for any changes that would affect the load ratings between design and construction in accordance with PUB 238 IP Article 2.3.1 and PUB 238 Appendix IP 01-F.

### 1.8.2 Existing Structures

After each inspection, existing structures are required to be evaluated to determine if the structure has had any changes that could affect the load ratings. For further discussion about how to determine when the existing ratings can still be considered valid or if a new load rating analysis is required, see Section 1.8.4.

Existing structures are rated and posted for PennDOT legal loads. Ratings are also developed for EV2 and EV3 vehicles; however, structures are not posted for these vehicles. In addition, rating factors are provided for PHL-93 live loading when required (when the structure is designed with LRFD methodology on or after 2011). Permit loads are rated on an as needed basis.

For rating the permit loads, DM-4 equation 3.4.1-3 may be used for existing structures. This equation assumes the permit load is in the controlling lane with the other lanes occupied by the design vehicular live load. It should be noted that this equation should not be used for floorbeam ratings or when the lever rule is used for both the single lane and multi-lane distribution factors. FHWA Memo HIBS-1 dated November 2016 allows the emergency vehicle to be placed in the controlling lane with other unrestricted legal loads for rating purposes. Therefore, it is appropriate to use the equation above that was developed for a permit load in a single lane combined with design load. The FHWA memo does not make a distinction between allowing this for existing or new structure ratings.

For existing structures, load rating evaluations should be created using as-inspected member properties. Unless specified, as-built ratings are typically not required, and only as-inspected ratings will need to be developed (assuming the conditions for as-built and as-inspected differ).

### 1.8.3 Repaired and Rehabilitated Structures

Major structure improvements such as repair or rehabilitation should have updated load ratings included with the structural plans that reflect the new conditions. Examples of repair or rehabilitation include member repairs or strengthening, bridge widening, superstructure or deck replacement, and preservation projects. For guidance on what methodology to use to analyze a repaired or rehabilitated bridge, see Section 2.11.2 and PUB 238 Table IP 3.6.2-1.

In conjunction with the initial structure inspection at the completion of construction (see PUB 238 IP Article 2.3.1.1 for when this is required), the structure load ratings should be reviewed and updated for any changes that would affect the load ratings between design and construction in accordance with PUB 238 IP Article 2.3.1 and PUB 238 Appendix IP 01-F.

For subsequent rating evaluations that would occur following the as-built load rating evaluation at the time of completion of work, the structure should be analyzed as an existing structure.

### 1.8.4 Safe Load Capacity and Posting

A component of every bridge safety inspection is the posting evaluation which is performed after the bridge conditions have been determined for the current safety inspection cycle and the load rating factors for the posting vehicles are known. It may be sufficient to use the previously calculated load rating factors as outlined in PUB 238 IP Article 4.3.1, unless changes have been made to the bridge since the previous rating was performed such as:

- A change in member capacity of the bridge in critical areas (e.g., additional structural section loss, spalled concrete, broken rebar or strands, distress, impact, construction damage, etc.).

- A change in the condition rating or distress level on a bridge load rated with the engineering judgment load rating method.

- Change in the condition of the superstructure, substructure, or culvert, which would result in the need to apply or reduce the SLC factor.

- Rehabilitation/retrofit/replacement of members or major portions of the bridge such as a new deck or floor system or changes to the functionality of the bridge.

- Substructure has deteriorated to a condition that may limit or further limit the load carrying capacity of the bridge (e.g., a substructure unit exhibits advanced section loss).

- Change in superstructure configuration (e.g., the bridge has been widened). Change in deadloads (e.g., additional wearing surface has been placed).

- Change in live load configuration (e.g., barriers have been installed to restrict vehicles from certain areas).

- Deck has deteriorated to a condition that may limit the load carrying capacity of the bridge.

- The controlling super/culvert condition rating is = 5 for 15 years or more and the current load rating is >= 15 years old.

- The controlling super/culvert condition is <= 4 and the current load rating is >= 10 years old.

- An update to the load rating software that would affect outcome of the load rating analysis.

This criterion is also used to determine when to update the load rating of an existing structure discussed in Section 1.8.2. In this case, only update the SLC if the condition rating has changed. For more information, see PUB 238 IP Article 4.3.

The SLC is calculated for controlling members or components of the bridge for each force effect using the posting vehicles in the evaluation. For more information on how to calculate the rating factors used in the SLC evaluation, see Section 1.1.2.1 above and see Section 1.1.3.1 for information on the SLC using the LRFR method.

The SLC is equal to the safe load capacity reduction factor ‘f’ times the OR. For values of ‘f’, see PUB 238 Table IP 4.3.2-1. The SLC shall be applied to the rating values based on the condition of the Superstructure/Culvert and Substructure. The rating engineer may use some judgment when choosing the SLC factor to apply to Superstructure ratings, when the Substructure rating is controlling the SLC, if the Substructure condition has been determined not to affect the load carrying capacity of the bridge. An explanation for the SLC factor used shall be provided in the load rating summary for these situations.

The safe load capacity reduction factor ‘f’ is generally based on the condition of the bridge. However, an ‘f’ of less than 1.0 may be warranted for a variety of other reasons such as: (1) unknown or limited data on the extent of section loss, spalling, or any other structurally significant deterioration, (2) unknown or limited information on materials used, (3) unknown or limited information on the size and/or spacing of reinforcement in concrete components, and (4) as is discussed in PUB 238 IP Article 4.3.2. For the H20 vehicle, 'f' should equal 1.0, thus the SLC will equal the OR. Therefore, 'f' can only reduce the SLC for the HS20, ML80 and the TK527. The SLC cannot be less than the Inventory Rating (IR) because the IR represents that load that can be carried by the structure or component being evaluated for an indefinite period time.

To summarize, the SLC should be equal to or less than the OR and the SLC should be equal to or greater than the IR. This leads to: IR ≤ SLC ≤ OR.

Before posting a bridge, verify that the analysis is accurate and that overly conservative assumptions have been removed to avoid unnecessarily placing weight restrictions on a structure. If the bridge rates low using the methodology suggested in PUB 238 Table IP 3.6.2-1, it may show favorable ratings using another methodology. Use of an alternative methodology other than that shown in the table referenced above should be approved by the Chief Bridge Engineer. For further guidance on analysis refinement, see Section 1.7.1.3. If the SLC is still found to be less than the gross vehicle weight of the legal load, the bridge should be posted. Bridges cannot be posted above the SLC and they cannot be posted below the IR.

For bridge posting evaluation requirements, weight restriction requirements and procedures for posting restrictions, see PUB 238 IP Article 4.

## 1.9 CAPACITY

Capacity, in the context of a bridge load rating, is a bridge component's ability to safely resist the applied loads at the desired rating level while considering the controlling limit states and load effects. The nominal capacity is based on the rating method, rating level, and material properties. The capacity should consider effects of deterioration such as concrete or steel section loss, loss of composite action, corrosion, etc.

For guidance on how to calculate structural capacity, refer to the following Sections of this Manual and the discussion herein:

- Section 2.5 for concrete components
- Section 2.6 for steel components
- Section 2.7 for timber components
- Section 2.8 for masonry components

*LRFR:* When determining the capacity (typically referred to as resistance in LRFD methodology) of a member using the LRFR method, the MBE provides equations for the strength and service limit state resistances (i.e., MBE equation 6A.4.2.1-2 and 6A.4.2.1-4 respectively). The strength limit state equation multiplies the factored resistance ($\varphi R_n$) by a system factor ($\varphi_s$) and a condition factor ($\varphi_c$).

The MBE Article C6A.4.2.1 states that the AASHTO LRFD load modifiers (η) are replaced with system factors applied to the resistance for a load rating. PennDOT does not permit load modifiers other than 1.0 for new structures in accordance with DM-4 Article 1.3.2.1. Therefore, the system factors should also be taken as 1.0 for a PennDOT load rating of a new bridge.

For existing and remediated structures, system factors corresponding to the load factor modifiers specified in AASHTO LRFD can be used. The system factors specified in MBE Table 6A.4.2.4-1 are more conservative and can be used at the discretion of the load rater as applicable, and they are based on the structures level of redundancy for checking flexural and axial effects at the strength limit state. A system factor of 1.0 should be used for checking shear at the strength limit state. All other limit states should use a system factor of 1.0 for checking all load effects. A system factor of 1.0 should be used to checking flexure and shear for timber bridges. Please note that the MBE directs the load rater to use a system factor for riveted and bolt gusset plates and connections for all force effects of 0.90. If the load rater can verify that the superstructure has an acceptable level of redundancy, see Ghosn et al. (1998), then the system factor can be set to 1.0.

The condition factor specified in MBE Table 6A.4.2.3-1 are optional and may be included to account for member deterioration, uncertainty of remaining resistance in deteriorated members, and potential for accelerated future deterioration. If element level condition data is not available, use NBI condition ratings for the superstructure and MBE Table C6A4.2.3-1 to convert the NBI rating into the condition factor. As noted in MBE Article C6A.4.2.3, if the section properties are accurately measured by field measurements of section loss rather than by estimated section loss percentages, the condition factors specified in in MBE Table 6A.4.2.3-1 can be increased by 0.05 to a maximum of 1.0. Note that it is considered conservative to apply the condition factor for a posting evaluation where the Safe Load Capacity (see Section 1.8.4) is applied.

The resistance factor (φ) should have the same value in a load rating as it does in design and φ = 1.0 for all non-strength limit states. For additional guidance on calculating member capacities, see Section 2 of this Manual as discussed above in this Section and MBE Article 6A.5 for concrete structures, MBE Article 6A.6 for steel structures, and MBE Article 6A.7 for wood structures.

The service limit state equation is equal to the allowable stress as specified in the AASHTO LRFD Bridge Design Specifications and as discussed in this Manual.

See Section 1.1.3.1 for further guidance.

*LFR:* The capacity of a member using the LFR method is the same for any rating level evaluated. The capacity of a member should be evaluated in accordance with the AASHTO Std. Spec. with additional guidance provided in this Manual as discussed above in this Section and MBE Article 6B.5.3. The capacity should consider observable effects of deterioration as is discussed MBE Article C6B.5.3. Use of the condition factors discussed above for LRFR are applicable for LFR load ratings when accurate field measurements of section are not available. For more information on the LFR method, see Section 1.1.3.2.

*ASR:* The capacity of a member using ASR method depends on the rating level evaluated. Unlike LRFR and LFR, ASR does not apply reduction factors such as the condition factor. The allowable stresses for each rating level are provided in this Manual in this Section and MBE Article 6B.5.2. For more information on the ASR method, see Section 1.1.3.3.

## 1.10 MATERIAL PROPERTIES

Material properties are used in all load ratings and are essential in determining member capacities as was discussed in Section 1.9. Material properties are typically determined from the material grade or design stresses specified in the plans.

PUB 238 IP Article 3.7.2 lists the hierarchy of which document should be used to determine the material properties and is especially helpful if information is not consistent between documents. The list does not include any type of remediation plans or shop drawings which should also be consulted. Shop drawings typically have more reliable material property information than the design plans.

If plans are not available, another means should be used to determine the material properties such as the year the bridge was built (use the tables provided in MBE), the specification that was used to design the structure, or through material testing. Material samples removed should be documented and removed in a manner that does not affect the structural integrity of the bridge and does not compromise the bridge components. See PUB 238 IP Article 3.7.2 for additional information.

Specific guidance for determining the material properties of concrete, steel, timber, and masonry can be found in the following Sections:

- Section 2.5.1 for concrete
- Section 2.6.1 for steel
- Section 2.7.1 for timber
- Section 2.8.1 for masonry

## 1.11 QA/QC

PennDOT has a QA Bridge Inspection Program that focuses on evaluating the load rating performance statewide to determine where targeted training is needed. The goal of the program is to establish statewide uniformity, consistency, and accuracy.

The QA program is broken into 6 parts. For the purposes of the Manual, only the portions of these sections with a focus on the load rating analysis will be discussed in detail.

1. District File Reviews – The District File Reviews is the review of the bridge office files to determine completeness, accuracy, consistency, and to confirm that the current documentation agrees with the information recorded in the BMS records. Staff assigned to this task, among other things, determine if the files contain the latest load rating analysis and if applicable, posting recommendations.

2. Independent Field Inspection and Report – As part of the Independent Field Inspection and Report, the QA team collects the data required to perform the load rating analysis and does not examine any of the information collected from the bridge files prior to completing their work.

3. Load Rating Evaluation – The Load Rating Evaluation involves performing an independent load rating analysis for a selected number of bridges. The QA team's load ratings are then compared to

the records found in the bridge files and any discrepancies are documented for presentation at the Close Out Meeting.

4. Draft Summary Report – The Draft Summary Report includes the results from the independent load rating analysis and recommendations for system compliance. Summary spreadsheets show comparisons between the QA results and results obtained from the District or Turnpike files, with in-tolerance percentages noted. In-tolerance results are defined as less than 15% for bridges that are not posted and 2 tons for bridges with a load posting.

5. District Close Out Meetings – In the Close Out Meeting, the results of the QA team's load rating analysis are discussed and recommendations for improvement are made.

6. Cycle End Summary Report – At the end of the inspection cycle, the QA team prepares reports summarizing the results of the QA evaluation for each PennDOT District and the Turnpike receives a separate report. The report includes detailed recommendations that will improve the overall inspection program and addresses any recommendations that may suggest a modification or improvement to the bridge inspection training program and the BMS2 Coding Manual.

For a load rating analysis, QC refers to ensuring that the load rating analysis is performed accurately, uses reasonable assumptions, selects the most appropriate rating methodology and analysis software, and represents the current condition of the structure based on the inspection findings. These measures are typically verified through checking of the load rating analysis. For guidance on how to perform a check of the load rating analysis, see Section 1.3.2.

To facilitate a high level of QC for a load rating analysis, it is important to ensure the individual performing the load rating analysis, called the load rating engineer, is qualified to perform the work. The PE code of ethics requires that any individual functioning as a qualified rating engineer be knowledgeable of all required processes and procedures as deemed necessary to complete the work at a quality level consistent with the standard of practice prescribed for the work. Reference PUB 238 IP Articles 2.1.1 and 4.3.1 for further information.

### 1.12 DOCUMENTATION AND SUBMITTALS

Upon completion of the load rating analysis, the rating engineer should compile a load rating package. The load rating package should be a comprehensive document including information about the bridge background, rating analysis procedure, assumptions, and the rating output. The contents of the rating package are discussed in detail below:

1. Report Cover: BMS ID, BRKEY No., County, Title of Rating Report, State route, Superstructure type, Number of spans, Material (Steel, Concrete, etc.), Date of load rating

2. Table of Contents. All documents should be bookmarked for ease of access.

3. Summary of Rating Tables

4. Description of Bridge: Feature carried, Feature crossed, Superstructure type, Number of spans, Skew, Total length, Out-to-out width, Bridge width curb-to-curb, Number of striped lanes, Number of lanes used in rating, Type of deck, Barrier or railing type, Year built, Rehabilitation year(s),

Design live load, Existing posted load, Any applicable plans or Standard drawings, Bridge sketches, Date of most recent inspection.

5. Rating Analysis Assumptions and Criteria: Includes Rating Specifications, Analysis method (ASD, LFD, LRFD), Scope of work, Software (PennDOT engineering programs or approved programs), Limit states, Condition factors, System factors, Resistance factors, Considerations for damage or deterioration, Material properties, Live loads and impact factors, Structure modeling notes, Live load distribution factor method, DC1/ DC2 calculation method, Sketches showing cross-sections of members.

6. Evaluation & Recommendations

7. Appendix A (Rating Files and Output): Include the electronic copy of the input data files for rated members, excel calculation spreadsheets, bridge analysis input and output files.

The input and output data files should show the initials of the preparer and checker along with the date. Include references to inspection reports, specifications, manuals, books, reports, and documents at appropriate locations in the report. It should be noted that input files (spreadsheets or other calculations) should clearly demonstrate how inputs were determined. Equations hidden in excel calculation spreadsheet cells that do not show how inputs are determined should not be used.

*Load Rating Summary Form:*

Summarize the load rating results on the bridge load rating form. Ratings are to be documented for the controlling as-built (if applicable) and as-inspected conditions. All assumptions should be clearly documented to facilitate updating calculations in the future. Copies of the load rating form are available in PUB 238 Appendix IP 03-C.

# SECTION 2 – BEST PRACTICES

## 2.1 GENERAL

When performing load rating evaluations, there are several best practices to be aware of and follow to adhere to PennDOT, AASHTO, and FHWA requirements, and to produce accurate, reliable, and consistent load rating results. Many of the best practices in the following sub-sections discuss several key topics; including common mistakes encountered in the load rating process, geometry for atypical bridge configurations, live load placement, elements to be load rated and when, load rating guidance for several superstructure types, material properties, and how to include deterioration and section loss in a load rating. These sub-sections also provide common best practice guidance on load rating substructures, foundations, non-typical bridges, and secondary members.

## 2.2 GEOMETRY

The load rating engineer should pay special attention to bridges with atypical geometry (e.g., skewed, curved). Subsequent sections provide guidance for rating these bridge types, including horizontally curved, skewed, and other non-standard bridge geometries.

### 2.2.1 Non-Standard Bridge Configurations

Several types of non-standard bridge configurations and associated load rating considerations are outlined below. Horizontal curvature and skew are discussed in later sections.

- *Splayed girders*. Bridges with beams or girders that are variably spaced, i.e., splayed or flared, (see Figure 2.2.1-1) require special considerations when determining load distribution. Follow the guidance in PUB 238 IP Article 3.3.3.3 and AASHTO LRFD Article 4.6.2.2.1 and commentary for LRFR, and the guidance in AASHTO Std. Spec. Article 3.23 for ASR and LFR when determining live load distribution factors for use in a simplified line girder analysis. Typically, the average beam spacing, the beam spacing at the 2/3 point of the span, the beam spacing at the centroid of the contributing deck area, or a weighted average beam spacing are acceptable for determining live load distribution factors. This choice is often inconsequential when the spacing variation is small. In other cases, multiple analyses with different distribution factors may be appropriate (e.g., multi-span bridges, large variations in beam spacing). Whatever beam spacing value is used, it must be between the actual minimum and maximum values present on the bridge. If the rating engineer has reason to believe that a simplified line girder analysis utilizing any of the previous assumptions is not producing accurate results, a refined analysis may be warranted.

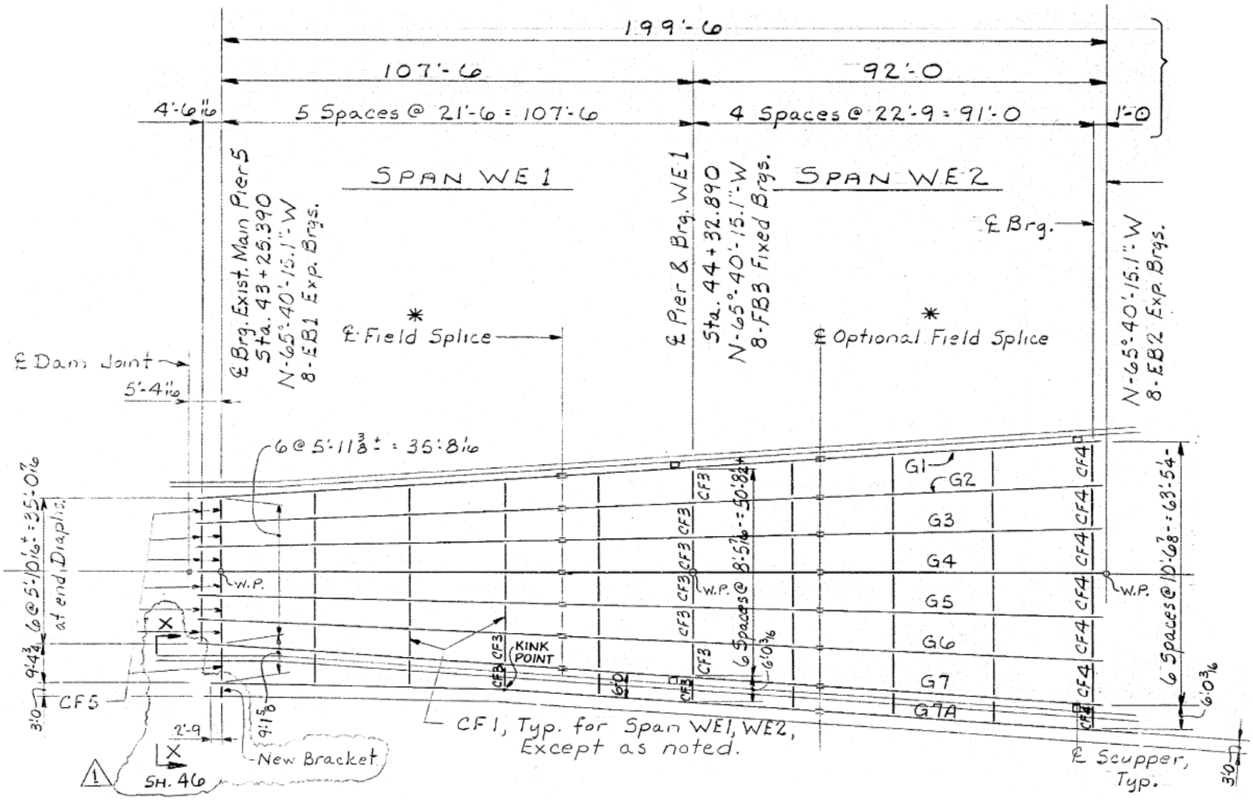



**Figure 2.2.1-1 – Example of a Splayed Girder Bridge**

- *Kinked girders*. Straight girders with "kink" points are often used on bridges with a curved alignment (see Figure 2.2.1-2). Follow the guidance in DM-4 Article 4.6.1.2.1 to determine if the girders should be analyzed as curved or straight. See Section 2.2.2 below for guidance on curved bridges.

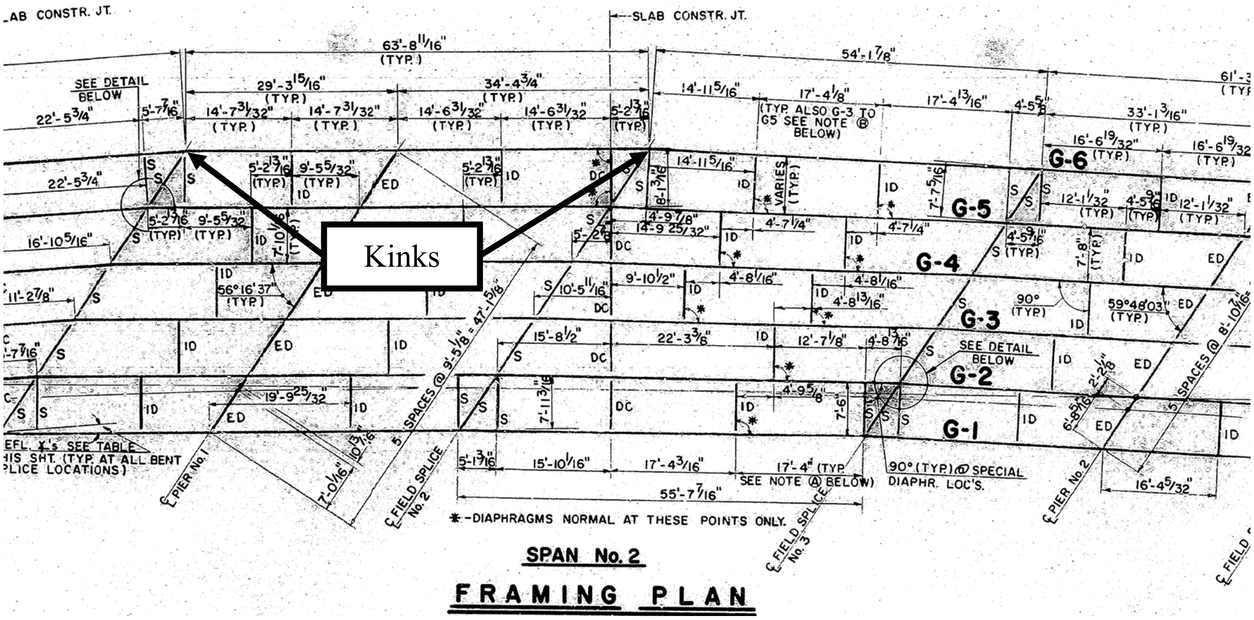



**Figure 2.2.1-2 – Example of a Kinked Girder Bridge (Partial-Framing Plan Shown)**

- *Variable deck width*. Bridges with splayed girders and varying deck widths where the width is proportional to the girder spacing (i.e., deck width and girder spacing vary at the same rate resulting in a constant overhang dimension) may be analyzed in a similar manner as outlined above for splayed girder bridges. As indicated in AASHTO LRFD Article 4.6.2.2.1, live load distribution factors in combination with a simplified line girder analysis may be used for small to moderate variations in deck width when using the LRFR method. For ASR and LFR analyses, distribution factors may be determined in accordance with AASHTO Std. Spec. Article 3.23. For large variations in deck width, a refined analysis should be used.

- *Plan aspect ratio*. The plan aspect ratio refers to the superstructure span length to width. Follow the guidance in AASHTO LRFD Article 4.6.1.1 when determining appropriate analysis methods based on aspect ratio. Bridges with small aspect ratios may warrant a refined analysis.

- *Others*. There are a number of other non-standard geometry types such as certain types of ramp structures (Figure 2.2.1-3), beams with significantly different stiffness (Figure 2.2.1-4), and bridges with discontinuous beams (Figure 2.2.1-5). These bridges should be evaluated on a case-by-case basis to determine appropriate analysis methods. In most cases, a refined analysis is warranted.

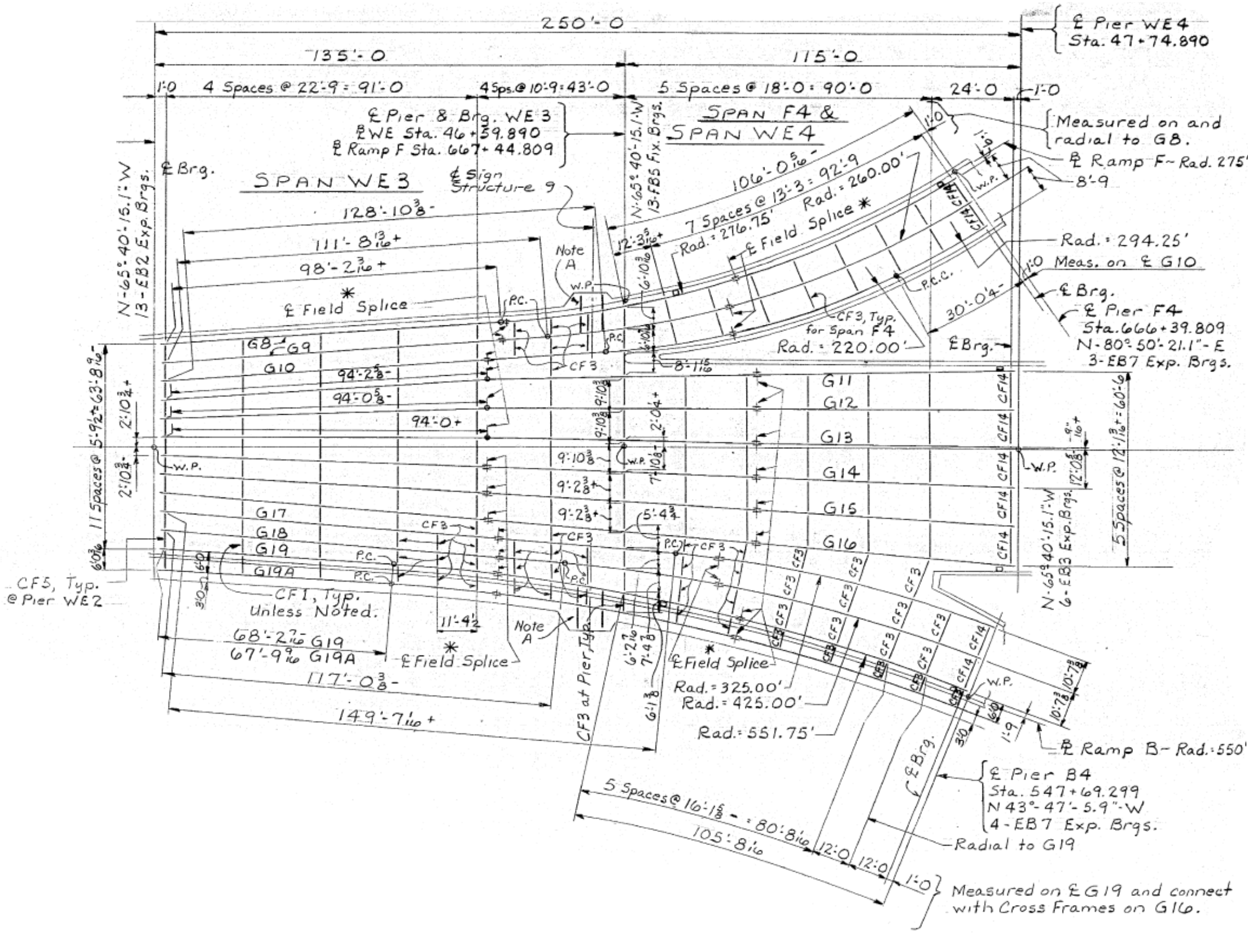



**Figure 2.2.1-3 – Example of a Ramp Structure with Atypical Geometry such as Splayed Girders, Variable Deck Width, and Horizontal Curvature**

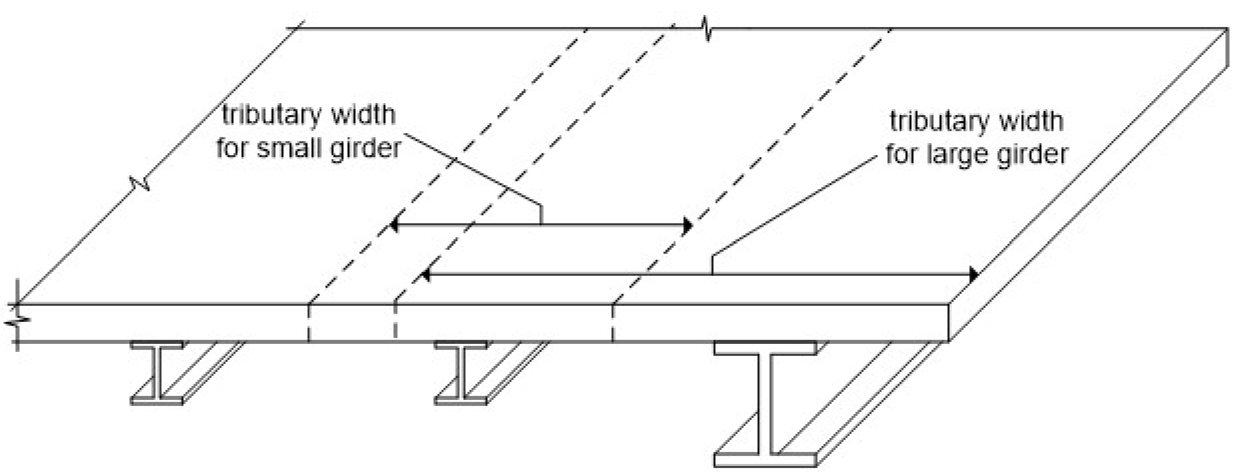



**Figure 2.2.1-4 – Example of Beams with Large Stiffness Differences (Taken from FHWA, *Manual for Refined Analysis in Bridge Design and Evaluation*, 2019)**

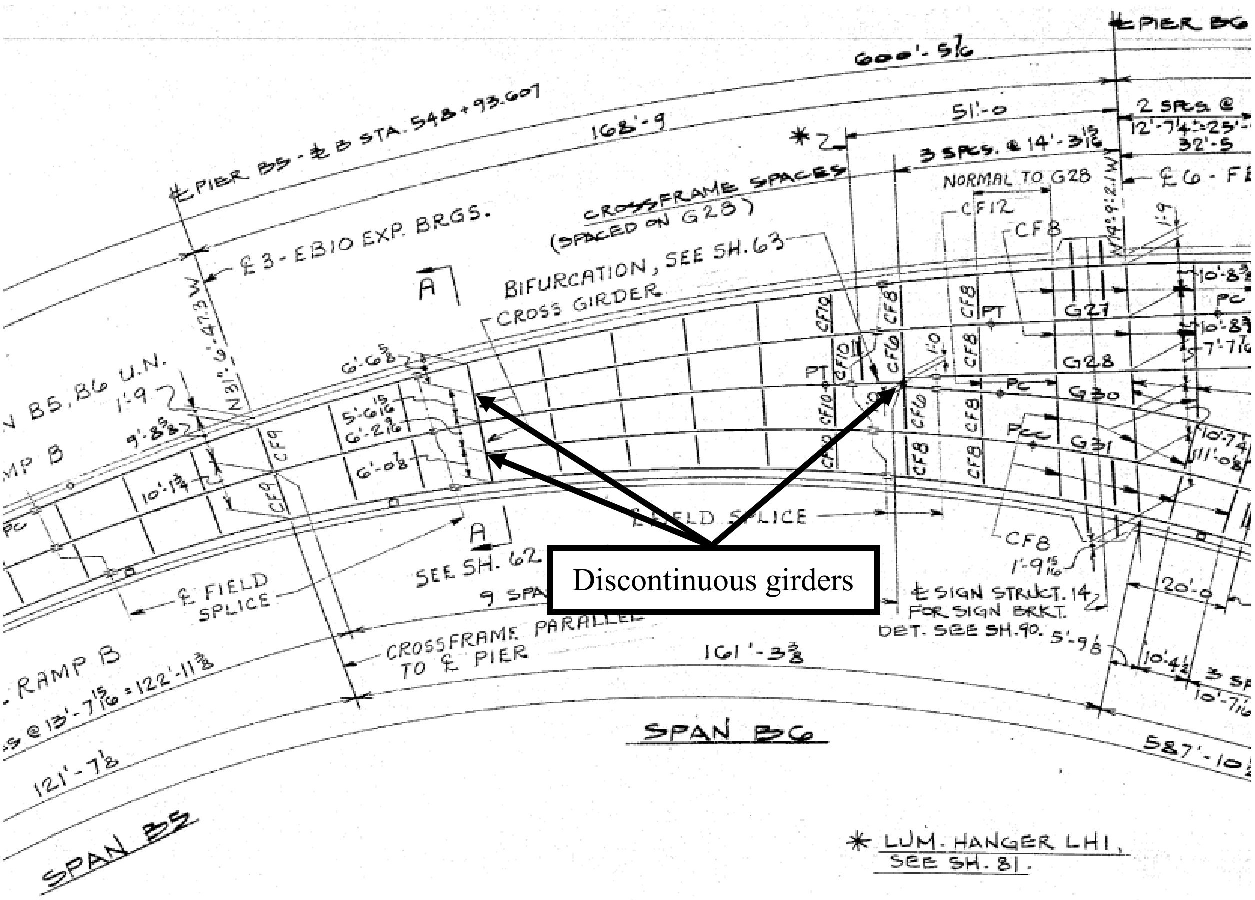



**Figure 2.2.1-5 – Example of a Bridge where Select Girders are Discontinuous (i.e., Partial Length) within a Span**

### 2.2.2 Horizontally Curved

A horizontally curved bridge is any bridge that is curved in a horizontal plane. Follow the guidance provided in PUB 238 IP Article 3.3.3.2, AASHTO LRFD Article 4.6.1.2, and DM-4 Article 4.6.1.2 to determine what level of analysis is required for load rating horizontally curved bridges. General considerations when load rating these bridges include the following:

- Bridges or spans within bridges that contain both straight and curved segments should be treated as curved (MBE Article C6A.6.1).

- For bridges with horizontally curved decks but straight girders (e.g., kinked or chorded), follow DM-4 Article 4.6.1.2.1 to determine if the girders should be treated as curved or straight for analysis purposes. If the girders can be analyzed as straight, for the purposes of determining live load distribution factors, the variation in the deck overhang may be treated using a method similar to those described for splayed girders in Section 2.2.1. Depending on the deck overhang geometry, it may be appropriate to use the average value, a weighted average, or to envelope the distribution factors using the maximum and minimum values.

- Secondary members like diaphragms and cross frames may need to be analyzed and rated as primary members depending on the degree of curvature and the type of bridge. See Sections 2.4.2.2 and 2.12 for guidance.

- Centrifugal forces should be included in the live load analysis. Centrifugal forces should be calculated in accordance with AASHTO LRFD Article 3.6.3 for LRFR. AASHTO Std. Spec. Article 3.10 provides guidance on determining centrifugal forces for ASR or LFR. The design speed used in calculating the centrifugal force should be taken from AASHTO's *A Policy on Geometric Design of Highways and Streets* or existing roadway plans, if available. With Chief Bridge Engineer approval, it may be acceptable to use a reduced design speed in an effort to reduce the centrifugal force and consequently increase the rating factor. The reduced design speed could be taken as the posted speed limit or the actual anticipated speed in the case of slow moving permit vehicles.

- On curved multi-girder bridges, typically girders on the outside of the curve carry more load than girders on the inside of the curve due to the global overturning moment effect caused by the curved span. Therefore, all else being equal, the girder on the outside of the curve will most likely control the load rating.

- Boundary conditions, mainly bearing fixity and orientation, should be considered when analyzing curved bridges. Correctly modeling boundary conditions is important to obtaining accurate analysis results and subsequent load ratings. There are often multiple types of bearings used in horizontally curved bridges with different orientations (Figure 2.2.2-1). Guided bearings are often aligned with the direction of thermal movement rather than the centerline of a girder (Figure 2.2.2-1 and Figure 2.2.2-2). Consult existing plans or inspection notes for bearing information.

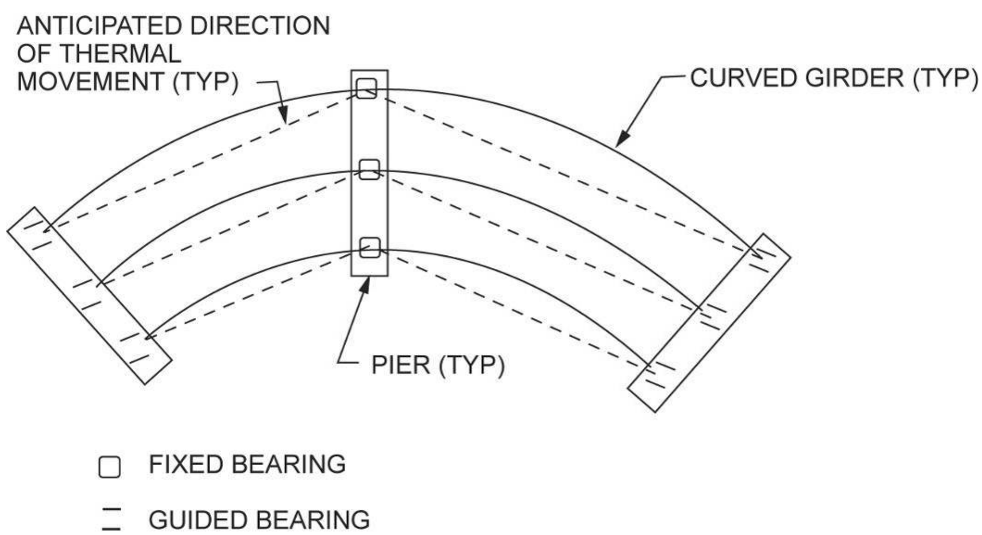



**Figure 2.2.2-1 – Example Curved Girder Bridge Bearing Boundary Conditions (Used with permission from *Guidelines for Steel Girder Bridge Analysis, G13.1-2019*, developed by the AASHTO/NSBA Steel Bridge Collaboration)**

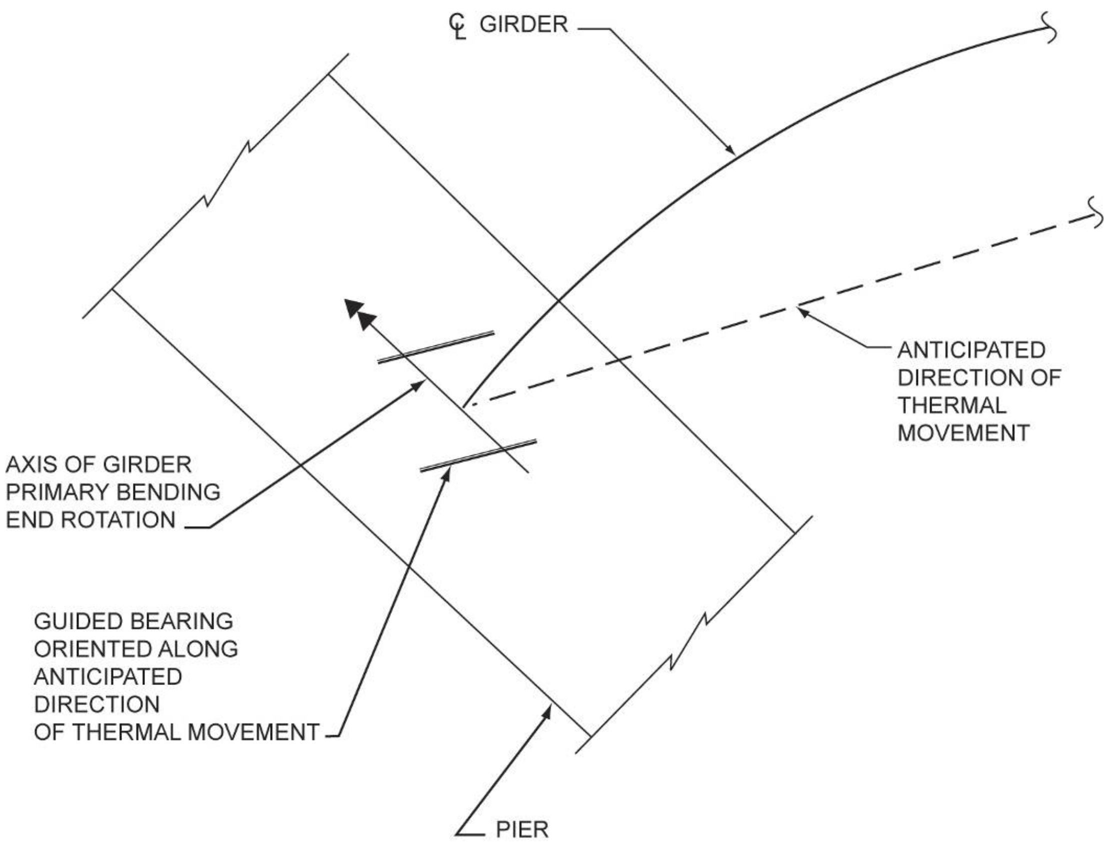



**Figure 2.2.2-2 – Example of a Guided Bearing Orientation on a Curved Girder Bridge (Used with permission from *Guidelines for Steel Girder Bridge Analysis, G13.1-2019*, developed by the AASHTO/NSBA Steel Bridge Collaboration)**

Additional considerations specific to different bridge types are provided below.

- *Single Girder Torsionally Stiff Superstructures*. Non-concrete box girder bridges of this type meeting the requirements of AASHTO LRFD Article 4.6.1.1 can be analyzed as curved spine beams (see AASHTO LRFD Article 4.6.1.2.2).

- *Concrete Box Girders*. Follow AASHTO LRFD Article 4.6.1.2.3 to determine if the bridge should be analyzed as straight or curved. Key provisions from this article include the following.

    - Bridges with central angles up to 12 degrees within one span may be analyzed as straight.

    - Nonsegmental bridges with central angles up to 34 degrees within one span may be analyzed as spine beams with straight segments.

    - Segmental bridges with central angles between 12 and 34 degrees within one span may be analyzed as spine beams with straight segments, provided the central angle of any individual element does not exceed 3.5 degrees.

    - Segmental and nonsegmental bridges with either central angles exceeding 34 degrees in one span or central angles exceeding 12 degrees in combination with other unusual or non-standard geometry should be analyzed using a refined 3D analysis method.

- *Steel I-Girders*. Follow AASHTO LRFD Article 4.6.1.2.4b and DM-4 Article 4.6.1.2.4b to determine analysis requirements. If certain criteria in these articles are met, then the girders can be analyzed as straight girders using a simple line girder analysis (i.e., 1D analysis). Otherwise, use a refined analysis and follow DM-4 Article 4.6.3.3.3, and AASHTO LRFD Articles 4.6.3.3.2 and 4.6.3.3.3. Refer to DM-4 Appendix E6P and Table E6.1.3.1P-1 to determine the recommended analysis method to use for load rating. White et al. (2012) and AASHTO/NSBA G13.1 Guidelines for Steel Girder Bridge Analysis (2019) provide additional guidance on choice of analysis for various geometries including curved girder bridges.

- *Steel Box and Tub Girders*. For structures meeting criteria from AASHTO LRFD Article 4.6.1.2.4c, the girders can be analyzed as straight girders using a simple line girder analysis (i.e., 1D analysis). Otherwise, use a refined analysis and follow DM-4 Article 4.6.3.3.3 and AASHTO LRFD Articles 4.6.3.3.3 and 4.6.3.4. White et al. (2012) and AASHTO/NSBA (2019) provide additional guidance on analysis. See Section 2.6.2.3 for guidance on rating steel box and tub girders.

### 2.2.3 Skewed

Skewed bridges are those where the intersection between the highway/roadway centerline and the major axis of a substructure unit is not orthogonal (i.e., at an angle other than 90 degrees). Skew tends to increase the vertical shear and reactions at obtuse corners of a bridge. Note that PennDOT has a different definition of skew angle, $\theta_P$, than AASHTO (see DM-4 PP Article 3.2.2).

Follow PUB 238 IP Article 3.3.3.1 when rating skewed bridges. Special considerations include the following.

- Bridges with 30° ≤ $\theta_P$ ≤ 90°: Use a simplified line girder analysis (i.e., 1D analysis) with skew correction factors for shear from DM-4 Table 4.6.2.2.3c-1 applied. Skew correction factors should be applied regardless of rating method used (i.e., LRFR, ASR, or LFR). For LFR and ASR ratings, refer to Table 3.23.2(A) of the 1993 DM-4 (These factors are similar to the current DM4 factors).

- Skew correction factors for moment are not applied (Pub 238 3.3.3.1 and DM-4 Article 4.6.2.2.2e).

- The skew correction factor shall be applied to the exterior beams only in accordance with the 1993 DM4 3.23.2. Note, for prestressed concrete adjacent beam beams, the skew correction factor shall be applied to all the beams which are on a skew.

- Straight Steel bridges with 30° ≤ $\theta_P$ ≤ 70°: Use the simplified line girder analysis with skew correction factors as indicated above. Cross-frames do not have to be rated as long as they are not distressed. If the cross-frames are distressed, treat them as main load carrying members and rate them.

- Girders with $\theta_P$ ≤ 30° require a refined analysis. DM4 4.6.2.2.1 is intended for design purposes; however, it can be used for guidance when a refined analysis is needed.

## 2.3 LIVE LOADS

The rating engineer should consider the guidance provided in the subsequent sections for assessment of live load with a focus on the practical aspects of bridge loading. These recommended best practices can help the rating engineer evaluate all relevant loading scenarios and improve structural ratings. For discussion on dynamic load allowance (impact), see Section 1.6.2.4.

### 2.3.1 Sidewalks and Non-Vehicular Lanes

For LRFR ratings, pedestrian load on sidewalks typically need not be considered for ratings concurrently with the vehicular load unless it has been evaluated by the rating engineer that this load case can produce the controlling loading on structural members. The probability of simultaneous application of full live load and full pedestrian load is low, therefore MBE Article 6A.2.3.4 does not require performing rating for combination of the two types of live loads.

For ASR/LFR ratings, apply the pedestrian live load to bridges with a sidewalk wider than 2’ in accordance with MBE Article 6B6.2.4. The pedestrian load shall be distributed similar to distribution of DL2 and include the same load factors. If the load ratings, which include the pedestrian load, are producing undesirable results, removal of application of the pedestrian live can be considered if the probability that full pedestrian live load and vehicle live load is low for the bridge site.

For permit load evaluations assume the permit trucks remain within the lanes, and that there are no pedestrians on the sidewalk.

PennDOT does not define a curb height for mountable curbs. Generally, curb heights greater than or equal to 6 inches can be considered as non-mountable and curb heights less than 6 inches as mountable. For mountable curbs, live load should be applied on the roadway including the sidewalk. This is applicable to design vehicles, permit trucks and emergency vehicles.

### 2.3.2 Loaded Lanes and Striped Lanes

Typically, all bridge members are load rated by positioning the live load (both design and permit) in the design lanes defined in MBE Article 6A.2.3.2. Typically, a bridge should be rated by placing the wheels in the design lanes, with the outside wheel no closer than 2.0 ft. the face of the curb.

In certain situations, with approval from the Chief Bridge Engineer, the striped lanes can be utilized for placement of the vehicle to represent the actual loading conditions. When the wheel is positioned at the inside face of barrier, it results in a very conservative estimation of live load effect on superstructure members which are sensitive to loading on the overhang such as exterior beams, through girders, trusses, arches, etc. Some existing bridges have wide shoulders, using the design loaded lane can give lower ratings which is not consistent with the normal service level bridge loading. With approval of the Chief Bridge Engineer, an alternate load rating method may be used per MBE Article 6A.2.3.2 and MBE Article 6B.6.2.2 to accurately model the structure and increase the load ratings. If alternate loading is approved, the wheel of the truck may be placed on the outside edge of the actual lane, but no closer than 2.0 ft. to the face of curb. If field conditions show that painted lanes are not visible on the bridge, design lanes should be used. Placing the live load in striped lanes instead of the design lanes can also be utilized for evaluation of permit trucks and design truck operating rating with the approval of the Chief Bridge Engineer.

In some instances, a vehicle could be driving outside the designated striped lane. Such incidents have a low probability of occurrence and need not be considered with the Strength-I load factors. The rating engineer should consider the number of loaded design lanes versus the number of striped traffic lanes for analysis of members with poor ratings with the approval of the Chief Bridge Engineer. MBE Article 6A.5.11.4 permits the use of striped lane for the service limit state evaluation of segmental concrete bridges for operating ratings.

Prior to applying loading within striped lanes, verify the correct location of actual or striped lanes from the inspection report. If live loads were placed in striped lanes, the rating engineer should state this in the load rating report and load rating summary form. Refer to MBE Article 6A.2.3.2 which describes placing live load in striped lanes. Emergency vehicle live load analysis should be done for the full roadway which includes the active traffic lanes and shoulders. In this case, MBE Article 6A.2.3.2 should not be considered.

### 2.3.3 Multiple Presence Factor

Multiple presence factors noted in AASHTO LRFD Article 3.6.1.1.2 should not be applied in combination with the distribution factors calculated using AASHTO LRFD Article 4.6.2.2 (PUB 238 IP Articles 3.3.2.3 and 3.4.3.3) for longitudinal and transverse members.

Follow MBE Article 6A.4.5.4.2.c for rating analysis of permit trucks with refined methods and MBE Article 6A.4.1 to evaluate single lane loaded permit trucks. Per MBE Article 6A.3.2 and C6A.6.4.2.2, a multiple presence factor should not be applied to the rating evaluation of one lane loaded permit trucks and emergency vehicles. Hence, the multiple presence factor should be divided out from a single lane LRFD Distribution Factor of permit trucks and emergency vehicles.

Multiple presence factors are not applicable to LFR and ASR ratings. Follow AASHTO Std. Spec. Article 3.12 to determine the reduction in live loads for different numbers of loaded lanes.

The multiple presence factors for box culvert rating of live load in the direction of the spans are described in MBE Article 6A.5.12.10.3.

## 2.4 ELEMENTS TO RATE AND WHEN

The load rating for a particular bridge is affected by the structural elements that are part of the direct load path for the live load. Typically, only the primary load carrying superstructure elements are load rated (e.g., beams, girders, stringers, floorbeams, truss members). Other bridge elements like secondary members, decks, substructures, and foundations are rated when their conditions warrant capacity evaluation or when overloading could affect the safe load capacity of the structure.

To expedite the load rating process, it is acceptable to take advantage of simplifications related to the elements that require rating. Regardless of the analysis method used for load rating (e.g., hand calculations, computer analyses), these simplifications can include the following:

- For identical members with equal properties (e.g., material properties, section properties, geometry), only rating one of the members or a smaller subset of members. Any differences in loading on individual members require separate rating calculations. An example would be rating one exterior and one interior beam out of a series of identical beams in a superstructure. Another example would be if the controlling interior and exterior beams are identical, only the beam with the higher distribution factor needs analyzed.

- Taking advantage of symmetry to reduce analysis effort. For example, only analyzing and rating one half of a symmetrical span or bridge. A second example is a truss bridge with a symmetrical cross section that results in only one truss line having to be analyzed and rated.

- Identifying loading conditions that produce maximum forces or stresses, and identifying locations of maximum force or stress along members. An example of the former is proving a vehicular live load controls over a pedestrian live load for a shared use lane or shared use bridge, thereby eliminating the need to perform an analysis for the pedestrian load. An example of the latter is a simple span girder of constant properties along its length where it can be recognized that maximum bending moment will occur at mid-span.

- Any other simplification methods that reduce analysis and rating efforts, yet still can justify that the controlling rating is captured, are acceptable.

However, several of the above simplifications may not be valid for bridges with significant deterioration and/or section loss. In these cases, the members, spans, etc. experiencing deterioration and/or section loss may need to be rated individually without consideration of simplifications to reduce rating efforts.

Element specific load rating requirements and considerations are provided in subsequent sections.

### 2.4.1 Decks

Requirements for when to rate decks are provided in MBE Article 6.1.5.1. Considerations for the load rating engineer when determining if a deck should be rated include the following:

- Deck slab superstructures are considered main load carrying members and should be load rated.
- Concrete decks supported by stringers and metal decks typically do not need to be load rated provided they are performing satisfactorily. Concrete decks may need to be rated for punching shear when subject to heavy wheel loads from permit vehicles.
- Wood decks can control the rating, particularly when exhibiting large live load deflections under normal traffic. In these situations, wood decks should be rated.
- Post-tensioned decks (longitudinal or transverse) may carry significant live load depending on the bridge type and support conditions, which could warrant a load rating.
- Decks with large support beam spacing (e.g., approximately 10.0 ft. or greater) or a large overhang width (larger than 1/2 of the beam spacing) may need to be rated.
- Deck slabs that are part of the main load carrying members (e.g., top flanges of concrete box girders, slab bridges) should be rated. Deck slabs that are composite with main load carrying members (e.g., composite with stringers or girders) may be considered in the composite section properties of the main member as appropriate, but need not be rated on their own.
- Decks on older bridges that were designed for lower weight design vehicles (e.g., H15, H10) in comparison to modern rating vehicles, and decks subject to overweight or heavy permit loads may need to be rated.
- Decks in poor condition may need to be rated if it is determined that the capacity is reduced. In addition, a deck in poor condition that does not distribute live load properly may warrant a modified live load distribution factor for the supporting member(s) (e.g., beams, girders, etc.).

### 2.4.2 Superstructures

Bridge superstructures are always load rated. All primary members are required to be load rated and, in some cases, select secondary members must be rated as well. There are not clear, specific definitions for primary and secondary members with exhaustive lists of examples for each; however, AASHTO LRFD provides the following definitions.

*Primary member* – A member designed to carry the loads applied to the structure as determined from an analysis.

*Secondary member* – A member in which stress is not normally evaluated in the analysis.

Guidance on distinguishing between primary and secondary members is discussed in subsequent sections.

#### 2.4.2.1 Primary Members

The following primary members should be load rated:

- On girder bridges (e.g., multi-girder, beam-slab): girders, stringers, and floorbeams, where applicable.
- On truss bridges: truss members (chords, diagonals, verticals, posts, eyebars on older bridges), stringers, floorbeams, and gusset plates.
- On complex structures (e.g., long-span bridges, movable bridges): members that are part of the primary load carrying path.
- Slabs on concrete slab bridges.
- Buried structures including culverts and arches.
- Any nonredundant members and their splices or connections, such as Nonredundant Steel Tension Members (NSTM) (formerly "Fracture Critical").
- Any pins or hinges within a primary member (e.g., pin and hangers on girder bridges, eyebar pins).

#### 2.4.2.2 Secondary Members

Secondary members that may need evaluated in a load rating include splices and connections, joints, bearings, and bracing members (e.g., diaphragms, cross frames, lateral bracing). There are cases where members that would typically be classified as secondary members act as main load carrying members and, therefore, are actually primary members that should be load rated.

Secondary members to load rate include:

- Diaphragms and cross frames in horizontally curved and/or skewed superstructures. See Sections 2.2.2, 2.2.3, and 2.12 for guidance.
- Splices and connections on nonredundant members as noted above in Section 2.4.2.1.
- Connecting elements in built-up compression members (e.g., lacing bars, batten plates) need not be rated individually, but their condition and contribution to the total member capacity should be considered. See PUB 238 IE Article 6.1.2.
- Permanent bottom flange lateral bracing members as required by MBE Article 6A.6.9.7 and AASHTO LRFD Article 6.7.5.1. See Section 2.12.3 for rating guidance.
- Bearings on certain skewed (see Section 2.2.3) or continuous bridges, while typically do not need to be rated, should be checked for uplift. Uplift at the ends of continuous spans subject to permit vehicle loads should be checked in accordance with PUB 238 IP Article 10.3.5. In other cases, project-specific criteria may require the bearings to be rated.

### 2.4.3 Substructures

Requirements for when to rate substructures are provided in MBE Article 6.1.5.2, PUB 238 IP Article 3.4.5, and PUB 238 IE Article 6.1.5.2. Similar to decks, substructures (e.g., abutments, piers) typically do not need to be rated. However, there are instances when substructures should be rated, including:

- Cross girders are required to be rated per PUB 238 IP Article 3.4.5.
- When a substructure is in poor condition. Concrete piers are specifically required to be load rated when they show signs of distress (see PUB 238 IE Article 6.1.5.2). Conditions that could warrant a load rating include corrosion and section loss, and impact damage, among others.
- When there is evidence of instability, or the potential for instability. This can be caused by scour, settlement, and changes in member end conditions, among others.
- When there is reason to believe the substructure rating will control the rating of the entire bridge. Examples include pier caps, columns, and wood or steel bents.
- When designed for lower weight design vehicles (e.g., H15, H10) in comparison to modern rating vehicles, when subject to overweight or heavy permit loads, or when significant additional load is added to the bridge (e.g., as part of a rehabilitation project).

### 2.4.4 Foundations

Unless otherwise required, foundations are not typically load rated. The Owner may require, or the encountered conditions may necessitate a load rating. An example of such conditions is loss of soil around piles by scour or other means that could cause buckling or reduce pile friction capacity.

## 2.5 CONCRETE STRUCTURES

This Section covers load rating best practices associated with reinforced concrete and prestressed concrete superstructures. This includes topics such as material properties, section loss, and specific concrete superstructure types. See Section 1.2 for load rating programs applicable to different concrete superstructure types. See Section 1.6 and 1.9 for design loads to consider and a discussion on capacity, respectively. See Section 2.4 for when to rate members of a concrete structure.

Do not consider the wearing surface when determining section properties and capacities of concrete members, but its dead load should still be considered in the evaluation. If the top of deck does not have a distinct wearing surface present, it is recommended to ignore the top 1/2 inch of deck thickness when calculating section properties.

### 2.5.1 Material Considerations

#### 2.5.1.1 Material Properties

See Section 1.10 for best practices to determine material properties of structural members. If plans are not available or concrete, steel reinforcement, or prestressing steel material properties are uncertain, assume values listed in PUB 238 IP Article 3.7.2 and MBE Article 6A.5.2 for LRFR method. For ASR method, follow MBE Article 6B.5.2. For LFR method, follow MBE Article 6B.5.3.

If steel reinforcement details (i.e., bar sizes, bar spacings, number of bars, etc.) in concrete structures are unknown, refer to Section 1.4. When appropriate, consider the use of non-destructive testing (see PUB 238 IP Article 3.7.1) or instrumentation (see PUB 238 IP Article 3.7.3).

If prestressing steel strand type is not specified in the available resources, assume stress-relieved strands. Determine the modulus of elasticity following AASHTO LRFD and DM-4 Article 5.4.2.4.

Calculate the moment of inertia of the concrete section based on the member type, the goal of the calculation (i.e., analysis versus capacity), and anticipated behavior of the section. The FHWA Manual for Refined Analysis in Bridge Design and Evaluation (Adams et al., 2019) Article 4.1.1.2 provides helpful dialogue on this topic. Depending on the type of evaluation and strain in steel reinforcement, it may be appropriate to consider the gross, cracked, or effective moment of inertia. For situations where the applied moment is less than the cracking moment, the gross moment of inertia may be considered. However, situations where the applied service moment is greater than the cracking moment capacity of the section, either the cracked moment of inertia or the effective moment of inertia should be considered. When the reinforcement reaches first yield, it is more appropriate to consider the cracked moment of inertia. The cracked moment of inertia is calculated by ignoring all concrete on the tension side of the neutral axis and transforming the area of tensile steel reinforcement. An effective moment of inertia, as calculated per AASHTO LRFD Article 5.6.3.5.2, is considered when the cracking moment is exceeded, but the steel has not yielded. This is typically considered for live load deflections and camber.

#### 2.5.1.2 Deterioration and Section Loss

Typical PennDOT load rating submissions require only as-inspected ratings be performed based on whether or not deterioration, a new wearing surface, and/or rehabilitation needs to be considered in the analysis (See 1.8.4 for a list of reasons the load rating may need to be updated). Review the latest inspection report to determine the condition of each structural component and the amount of section loss to consider, if warranted. Clearly document all assumptions corresponding to how section loss decisions were made. If multiple locations of section loss are reported for one member, consider the critical stress locations along the member length and what will lead to the lowest ratings. Depending on where the section loss is located, it can be overly conservative to consider localized section loss over the entirety of a concrete component. Be aware that PennDOT's BAR7 (ASR, LFR) program will allow varying section properties along the length of a member for most bridge models, but not all, and it does not include inputs for localized section loss on a portion of a component in its cross section. Also be aware that PennDOT's PS3 (ASR, LFR) and PSLRFD (LRFR), which are used to evaluate prestressed beams, do not allow the user to input different beam sizes and strand configurations within the same span. Multiple evaluation runs may be required to evaluate different portions of the same beam when utilizing these programs. Because these programs have no input for localized section loss, the appropriate cross-section dimensions must be reduced to account for section loss. The as-inspected reinforcement area and center of gravity must be determined outside the program. Alternatively, if necessary, AASHTOWare (BrR) may be used to input localized section loss for LRFR, LFR and ASR ratings. BrR is a 3D FEM program that can better capture the effects of localized imperfections. Figure 2.5.1.2-1 shows an example of severe concrete spalling and section loss of exposed longitudinal and shear reinforcement in a reinforced concrete beam.


**Figure 2.5.1.2-1 – Heavy Spalling and Exposed Reinforcement in a Reinforced Concrete T-Beam**

In the event that a vehicle collides with a bridge, any damaged concrete members must be evaluated for loss in their capacity based on the observed and measured damage. This is particularly important when beams are struck by a vehicle traveling below the bridge, where attention must also be given to lateral deformation. Minor damage can sometimes be ignored in a load rating evaluation. When damages are severe or deformations are observed, a refined analysis may be required to capture the deformed shape and any eccentric loading. If vehicle impact causes a primary member failure or if deformations are likely to result in failure of the member, these scenarios are beyond the scope of this Manual and the District Bridge Engineer should be notified immediately.

*Reinforced Concrete Members*

Consider section loss to webs when determining controlling shear ratings of beams, specifically near supports where shear is greatest. Web section loss at low shear locations (mid-span) should not be spread across the entire beam length (i.e., if a design program is used, as described above).

Consider section loss of web depth and flanges when determining controlling flexure ratings of flexural members. Section loss in longitudinal steel reinforcement, particularly tensile reinforcement, must also be considered when rating these members. The area and center of gravity of remaining reinforcement will need to be determined. See Section 1.9 for discussion on how and when to further reduce the capacity of the section based on its condition.

Consider concrete delamination documented in the Inspection Report by reducing the concrete cover. This is especially critical for shear ratings when concrete delamination is observed in the webs. Reduce concrete cover in all appropriate capacity equations. Concrete delamination on the tension face of the concrete member is not overly concerning except as noted in the following paragraphs when the tensile reinforcement is exposed. Concrete delamination on the compression face must be considered by reducing the member's section properties (overall depth, depth to tension reinforcement, etc.) For extreme cases where steel reinforcement is fully exposed, check to ensure the section can still be considered composite between the concrete and reinforcing or if some or all of the reinforcing should be ignored.

A sensitivity analysis may be useful to determine how sensitive the member capacity is to different section loss assumptions and if any assumptions need further refinement. To conduct a sensitivity analysis, change one variable (e.g., web thickness) and keep all other variables fixed. See how changing that variable affects the ratings. Then, move onto the next variable and follow the same procedure to better understand what variables have the most effect. If changing a variable significantly affects the controlling ratings, the ratings are said to be very "sensitive" to that variable. If efflorescence is noted on concrete members in the Inspection Report, no section loss should be considered unless measurable loss is documented. See Figure 2.5.1.2-2 for an example of a reinforced concrete beam with efflorescence and no measurable section loss.

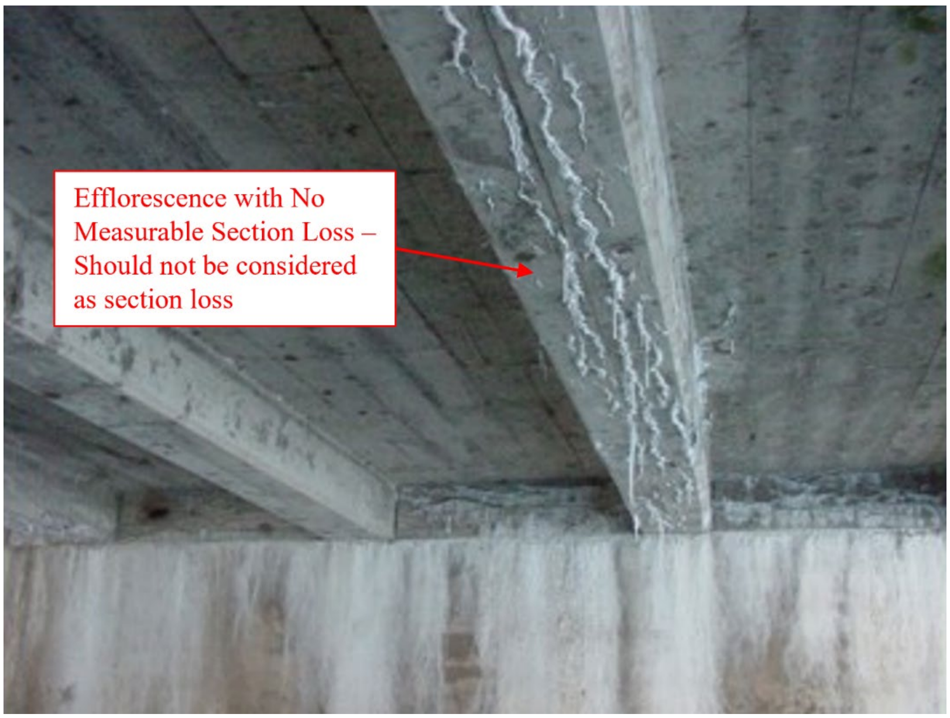



**Figure 2.5.1.2-2 – Reinforced Concrete T-Beam with Efflorescence and No Measurable Section Loss**

Shear stirrups are considered effective if they are hooked/anchored/bonded at the top and bottom of the vertical legs. Per MBE Article 6A.5.8 (LRFR), shear capacity of reinforced and prestressed concrete should be evaluated for all vehicular loads, including permit loads. However, even if there is exposed shear reinforcement and section loss, as shown in Figure 2.5.1.2-1 above, shear capacity does not need to be evaluated for concrete bridges that show no evidence of shear distress or diagonal cracking when rating for design or legal loads. Deteriorated or ineffective shear stirrups may require using a revised, average spacing and an associated reduced shear capacity. However, if this is a localized issue at a high shear location, remove the ineffective stirrups and consider the increased stirrup spacing at that location. If using the ASR method, shear capacity should be calculated per MBE Article 6B.5.2.4.3.

For reinforced concrete beams, consult MBE Article 4.3.5.6.2 for insight into what certain deterioration could indicate.

Reinforcement with insufficient cover due to concrete deterioration and spalling is especially concerning in high flexural stress locations. Exposed reinforcement typically indicates some level of bond loss between the concrete and steel. Reinforcement is considered fully debonded when it is no longer composite with concrete; see Section 2.13.3 for more information on the importance of concrete and steel bonding. Originally straight tensile bars (e.g., bottom longitudinal bars in a simple beam) that are exposed and kinked or bent according to a Bridge Inspection Report indicates ineffective reinforcement since no tensile force is in those kinked/bent bars.

How to handle exposed tensile reinforcement near high flexural stress locations (e.g., midspan of a simple span beam) depends on the length of deterioration. Figure 2.5.1.2-3 shows an example of a simple span beam with exposed tensile reinforcement near midspan. If the debonded length is limited and sufficient development length in sound concrete is present adjacent to the deterioration towards the support, those bars can be considered fully effective at midspan. Sufficient development length of the reinforcement should be determined using AASHTO LRFD Article 5.10 and PennDOT standard drawing BC-736M; however, use engineering judgement to decide if a longer length is needed due to deteriorated concrete since AASHTO LRFD Article 5.10 and BC-736M are for concrete in good condition. Alternatively, if proper development of the tensile reinforcement near high flexural stress locations is not confirmed or when rebar splices are within that area, the entire layer of reinforcement should be considered ineffective. If a rebar splice is visible, the splice lap length is likely inadequate, which causes those bars to be ineffective.

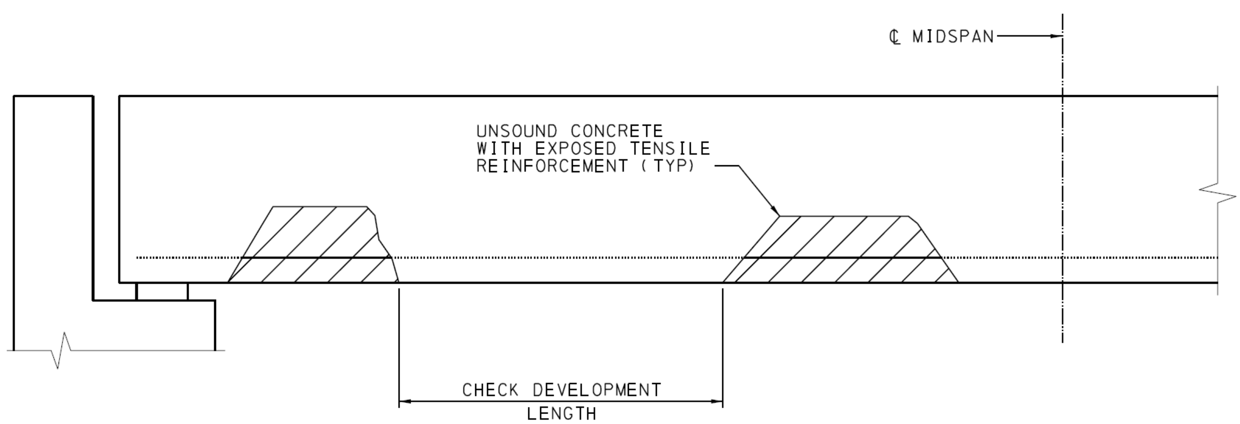



**Figure 2.5.1.2-3 – Debonding Length Schematic for Simple Beam**

Severe spalling with exposed and significantly corroded reinforcement may require a refined analysis to assess if the beam has any live load capacity remaining. Suggestions and details of this evaluation are outside the scope of this Manual. If there is severe spalling, debonded reinforcement, and a complete loss of section over a significant length, the District Bridge Engineer should be contacted immediately if the Inspection team had not done so already. Without a refined analysis, the entire beam should be considered ineffective and the load rating re-evaluated assuming the deteriorated beam is providing no load capacity. As mentioned above, beams with inadequate development length of the tension steel are also concerning and should be handled similarly. Figure 2.5.1.2-4 shows an example situation when the entire beam should be considered ineffective and removed for the evaluation. Steel reinforcement bars with concrete completely deteriorated around them are considered "Air Bars" because it appears as if they are floating. Typical load rating programs (such as BAR7) are ineffective in this situation since they assume steel reinforcement entered into the program is fully bonded in sound concrete. Attempting to use those programs with no tension reinforcement in a beam will result in zero flexural resistance. When a beam is considered ineffective and removed from the analysis, adjacent beams must then be evaluated to ensure they can adequately support the additional load demand. Also, the deck will need to be evaluated to ensure it has adequate capacity to span over the “removed” beam.


**Figure 2.5.1.2-4 – Example of Ineffective Reinforced Concrete T-Beam – "Air Bars"**

*Prestressed Concrete Members*

MBE Article 4.3.5.6.3 and PUB 238 IE Article 4.3.5.6.3 provide insightful dialogue on deterioration in prestressed concrete beams and what that could possibly indicate.

Non-composite adjacent box beam (NCABB) bridges are more prone to section loss, deterioration, and strand corrosion because of the thinner deck or absence of a deck entirely, directly exposing the box beams to water and chlorides. Figure 2.5.1.2-5 shows a NCABB with large concrete spalling and exposed prestressing strands. When load rating NCABB bridges, consult the Inspection Report to determine if there is evidence of shear key failure between the beams. Possible shear key failure is evident by reflective cracking in the wearing surface, camber variations, differential deflections, leakage between beams, poor quality grout, and loose transverse post-tensioning ties. See PUB 238 IE Article 4.3.5.6.3.1I for additional information on deteriorated NCABB’s. If there is shear key failure, the beams will act more independently, and thus, decrease load distribution to the adjacent beams. To account for shear key failure, consider one wheel line will be distributed per beam as directed in PUB 238 IE Article 6B.6.3.

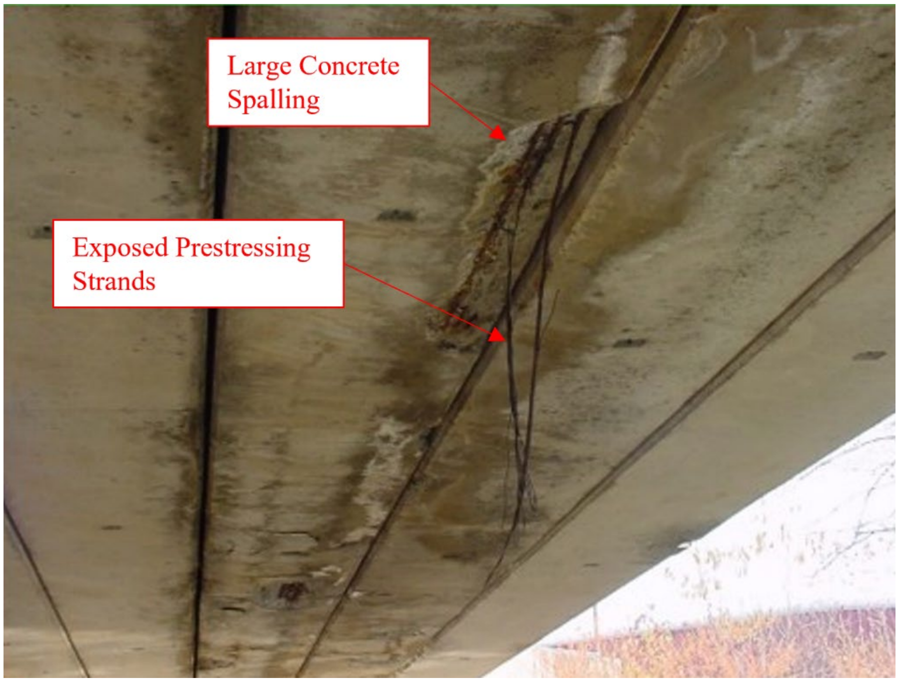



**Figure 2.5.1.2-5 – Spalling and Exposed, Severed Prestressing Strands in a Prestressed NCABB**

If the deck or superstructure is in poor condition, it is possible that composite action is lost between the deck and beams. If there are signs that indicate this, it may be more appropriate to consider the non-composite section will resist superimposed dead load and live load. See Section 2.5.4 for additional commentary on how deck condition affects the evaluation of prestressed concrete girder bridges.

If transverse cracks are observed in large bending moment areas, particularly on the bottom flange or the bottom of web, further investigation may be required. Transverse cracks may imply flexural cracking of the member, which could indicate previous overloading and possibly a loss in capacity.

Evaluate deterioration and section loss, such as longitudinal cracks and exposed strands, in all prestressed concrete beams per PUB 238 IE Article 6.1.5.3I. Hairline longitudinal cracks are typically not considered as section loss unless there is evidence of other deterioration in the surrounding area. Although this article focuses on three methods for load rating NCABB's, such as what is shown in Figure 2.5.1.2-5 above, it may be applied to all prestressed concrete members. Method A is a simple and more conservative evaluation procedure. Method B, sometimes called the "Lehigh Method", is a more refined evaluation based on research performed by Lehigh University in 2010 (Naito et al. 2010). Method C is strictly used for fascia beams when the Capacity/Dead Load < 1.5 or an Operating Rating < 1.5. Common practice is to evaluate the prestressed member following Method A first. If this yields poor ratings, evaluate the member again per Method B. PUB 238 Appendix IE 04-D provides an example of how to evaluate a NCABB with various deterioration and section loss following Method A and Method B. Figure 2.5.1.2-6 is an altered figure taken from that example.

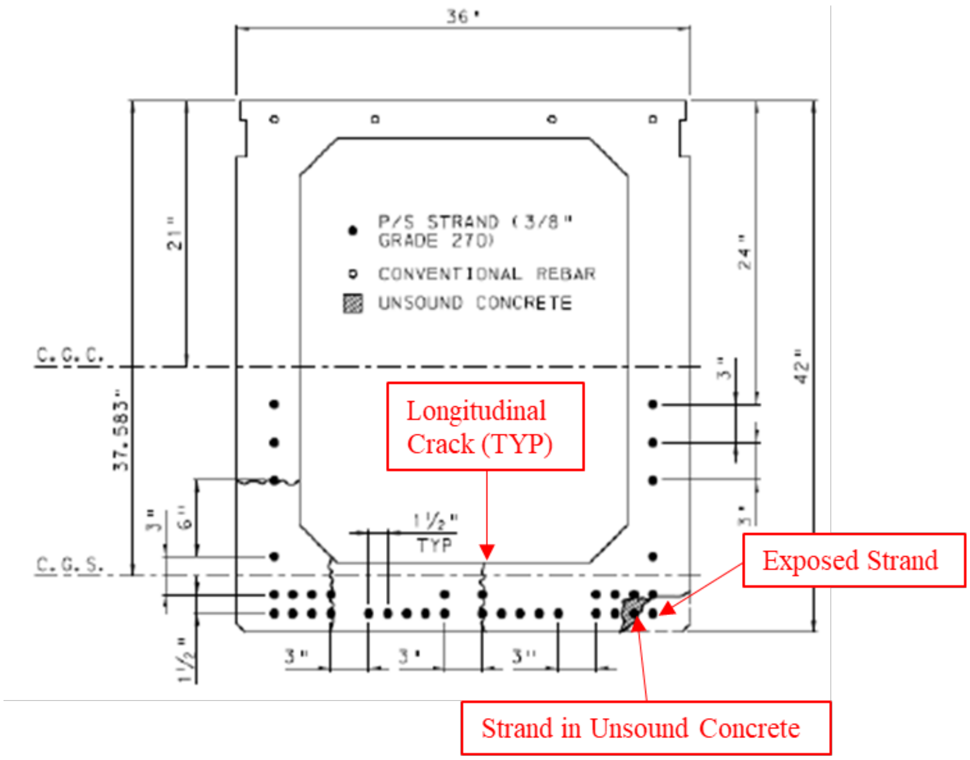



**Figure 2.5.1.2-6 – Example of a NCABB with Various Deterioration and Section Loss (Adapted from PUB 238)**

Following Method A, all exposed strands within the middle 1/3 of the span are considered ineffective over the entire beam length. All exposed strands within the end 1/3 of the span are considered ineffective between the beam end and the location of the deterioration. Therefore, even if various strands are exposed at two different locations longitudinally along the beam, depending on where the section loss is located within the span, the beam capacity must consider all exposed strands as ineffective within the same cross-section. Also, an additional 25% of strands should be conservatively deducted from the lowest possible row of strands. This will reduce the moment arm between the compression arm and the center of gravity of the prestressing strands and, thus, result in a beam with the smallest capacity. Figure 2 and Table 2 in PUB 238 Appendix IE 04-D show how to remove strands per Method A given various deterioration and section loss.

Following Method B, all damage within two development lengths is considered to occur in the same cross-section. If development length cannot be determined based on design information available, refer to PUB 238 Table IE 6.1.5.3I-1. To be conservative, assume the analysis window extends in the direction that will lead to the lowest load rating. Figure 3 and Table 3 in PUB 238 Appendix IE 04-D show how to remove strands per Method B given various deterioration.

PUB 238 Appendix IE 04-D provides more details on the example pertaining to Figure 2.5.1.2-6 and Methods A and B as discussed above. See Section 2.5.4 of this Manual for more details on prestressed concrete members.

Once prestressing strands have been removed due to section loss, the area and center of gravity of remaining strands must be determined. Following this process for steel reinforcement and prestressing strands is especially important for programs such as PS3, BAR7 and PSLRFD where the total area and center of gravity are input instead of individual reinforcement bars or strands.

The strand layout will likely be asymmetric after removing exposed prestressing strands from the evaluation. Asymmetric strand patterns can induce transverse prestress forces, resulting in out-of-plane bending moments that are typically not considered in designs. This should be evaluated to determine its effect on the as-inspected member. Conventional methods of calculating capacities, such as those utilized by PSLRFD and PS3, may no longer be applicable and a refined analysis may be required. Such analysis is beyond the scope of this Manual.

*Box Culverts*

See MBE Article 4.3.5.9 for inspection procedures, section loss to document, and defects to note when load rating reinforced concrete box culverts. AASHTO Culvert and Storm Drain System Inspection Guide (2020) provides additional information on inspection of concrete culverts and what various concrete degradation could indicate. Ream et al. (2019) provides some guidance for how to handle section loss and deterioration in concrete tunnels, which can be applied to culverts.

### 2.5.2 Reinforced Concrete Beam Superstructures

Evaluate shear and flexural load ratings for reinforced concrete beam superstructures. PennDOT's BAR7 can be used to evaluate reinforced concrete beam structures following LFR and ASR load rating methods. AASHTOWare (BrR) is a 3D FEM program that may be used for LRFR and LFR load ratings of reinforced concrete beam structures. When determining load demand on beams, determine whether the beams were constructed continuous for live load and evaluate accordingly. There is more discussion on this and how the deck and diaphragm condition affects the evaluation in Section 2.5.4.

Evaluate flexural ratings for LRFR taking into account the provisions of MBE Articles 6A.5.5 and 6A.5.6, methodologies presented in AASHTO LRFD Articles 5.5.1 and 5.6.3 and DM-4 Articles 5.5.1 and 5.6.3, and assumptions listed in AASHTO LRFD Article 5.6 and DM-4 Article 5.6, as applicable.

Evaluate shear ratings per MBE Article 6A.5.8 and AASHTO LRFD Article 5.7 for LRFR method. For LFR method, follow AASHTO Std. Spec. Article 8.16.6. For ASR method, follow MBE Article 6B.5.2.4.3.

There are specific situations, such as a beam in a severely skewed bridge, where the shear reinforcement is splayed at the end of a beam. Because it is likely this will be at a critical shear location, conservatively determine the shear capacity using the largest spacing and if this yields poor ratings, the average spacing may be appropriate.

MBE Article L6B.3 includes helpful equations and references to AASHTO Std. Spec. articles for capacity determination of typical reinforced concrete members associated with the type of evaluation performed. MBE Appendix A2 provides an example load rating of a reinforced concrete interior T-beam following LRFR, LFR and ASR rating methodologies.

The strut and tie method per AASHTO LRFD Article 5.8.2 is an increasingly popular and often preferred refined analysis method for designing reinforced concrete members. Because this is a refined analysis, Chief Bridge Engineer approval should be acquired before utilizing this method. The method is used to determine the capacity of reinforced concrete components in disturbed areas, or "D" regions, that do not adhere to traditional design assumptions, defined in AASHTO LRFD Article 5.5.1.2.3. Although this method was not common years ago and likely not considered in the design process, it is still a helpful tool to consider when rating unique concrete members in which the sectional method is not appropriate.

Precast channel beams are often used for short-span bridges (typically less than 50 ft.). Many of the best practices mentioned for reinforced concrete members can be applied to this beam type. PennDOT BD-668M has some information on the design of Precast Channel Beam Bridges. For precast channel beam bridges built between 1989 and 1999, consult PennDOT BC-793M Precast Channel Beam Bridges for applicable details, as necessary.

For precast concrete beam bridges, if applicable, compute the live load distribution factors for LFR and ASR ratings per AASHTO Std. Spec. Article 3.23.4.3.

When evaluating permit loads at the Service I limit state, optional reinforcing bar stress limit criteria is provided in MBE Article 6A.5.4.2.2b.

For LFR rating of reinforced concrete NCABB's, determine live load distribution factors per PUB 238 IP Article 3.3.2.2.

### 2.5.3 Reinforced Concrete Slab Bridges

When load rating reinforced concrete slab bridges using LRFR methodology, follow AASHTO LRFD Article 4.6.2.3. Evaluate flexural ratings for LRFR taking into account the provisions of MBE Articles 6A.5.5 and 6A.5.6, evaluation methodologies provided in AASHTO LRFD Articles 5.5.1 and 5.6.3 and DM-4 Articles 5.5.1 and 5.6.3, and assumptions listed in AASHTO LRFD Article 5.6 and DM-4 Article 5.6, as applicable.

Evaluate shear ratings per MBE Article 6A.5.8 and AASHTO LRFD Article 5.7 for LRFR method. For LFR method, follow AASHTO Std. Spec. Article 8.16.6. For ASR method, follow MBE Article 6B.5.2.4.3.

For reference, MBE Appendix A7 provides an example LRFR load rating of a reinforced concrete slab bridge. AASHTOWare can perform LRFR analysis of reinforced concrete slab bridges. When performing LRFR ratings of prestressed concrete slab bridges built prior to 1970, see the live load distribution guidance in MBE Article C6A.3.2.

BAR7 may be utilized to load rate reinforced concrete slab bridges using LFR or ASR methods. For cast-in-place concrete slab bridges, compute the live load distribution factors per AASHTO Std. Spec. Article 3.24.3 for LFR and ASR ratings.

### 2.5.4 Prestressed Concrete Beam Superstructures

Section 2.5.1.2 has an in-depth discussion on various section losses and deteriorations and how they affect the evaluation of prestressed concrete beams.

MBE Article L6B.4 includes helpful references to AASHTO Std. Spec. articles for flexural and shear capacity determination of typical prestressed concrete members following LFR methodology. DM-4 PP

Article 5.7.2 provides a rating procedure for existing prestressed concrete bridges following LRFR methodology.

Evaluate flexural ratings for LRFR considering the provisions of MBE Articles 6A.5.5 and 6A.5.6, design methodology provided in AASHTO LRFD Articles 5.5.1 and 5.6.3 and DM-4 Articles 5.5.1 and 5.6.3, and assumptions listed in AASHTO LRFD Article 5.6 and DM-4 Article 5.6, as applicable.

Evaluate shear ratings per MBE Article 6A.5.8 and AASHTO LRFD Article 5.7 for LRFR method. For LFR method, follow AASHTO Std. Spec. Article 8.16.6. For ASR method, follow MBE Article 6B.5.2.4.3. It is recommended to use an average stirrup spacing when the spacing varies on either side of the beam or at the beam end due to skew.

For LFR rating of prestressed concrete beams and rating equations, follow the criteria defined in MBE Article 6B.5.3.3.

MBE Article 6A.5.4.2.2a does not require checking Service III limit state for LRFR ratings of bridges that satisfy limiting tensile stresses under service loads. This check is appropriate for prestressed concrete bridges that experience cracking under normal traffic. If Service III limit state is evaluated, MBE Article 6A.5.4.2.2a suggests using a live load factor of 1.0 for legal loads.

When evaluating permit loads at the LRFR Service I limit state, optional prestressing strand stress limit criteria is provided in MBE Article 6A.5.4.2.2b. When calculating prestress losses, use the Refined Method defined in AASHTO LRFD Article 5.9.3.4 and DM-4 Article 5.9.3.4 to determine time-dependent losses. The Approximate Method defined in AASHTO LRFD Article 5.9.3.3 and DM-4 Article 5.9.3.3 should not be used unless otherwise approved by the Department. When calculating time-dependent losses using the Refined Method, the following values can be used if actual values are not available or cannot be obtained:

- Prestress transfer time, $t_i$ = 1 day
- Relative humidity, $H$ = 70%
- Deck placement age, $t_d$ = 120 days

Allowable concrete tensile stress limits for prestressed concrete beam bridge ratings using LFR methodology are provided in PUB 238 IP Article 3.6.3.

PennDOT's PS3 may be used to evaluate prestressed concrete beams following LFR methodology.

Load rating of typical prestressed multi-girder bridges using LRFR methodology are often performed using PennDOT's PSLRFD program. PSLRFD can evaluate beams as either simple span or continuous for live load. Refer to the PSLRFD's User Manual for questions on its capabilities, evaluation methods, input questions, etc.

Unless temporary shoring was used during construction, evaluate the beam as simple spans resisting non-composite dead load (DC1). If the deck is in good condition, evaluate the beam as continuous resisting live load and superimposed dead load. If the deck is in poor condition and if evaluating the beam as continuous resisting live load results in poor ratings, consider the beam as simple span when resisting live load. For more discussion on analyzing multi-span prestressed beam bridges, see PUB 238 IP Article 3.5.

### 2.5.4.1 Box Girders

For LFR rating of prestressed NCABB's, determine live load distribution factors per PUB 238 IP Article 3.3.2.2. Per MBE Article C6A.3.2, prestressed NCABB's built prior to 1970 likely do not have sufficient post-tensioning to act as a unit; therefore, the live load distribution should be determined using PUB 238 IP Article 3.3.2.2. When performing ratings according to ASR and LFR of prestressed NCABB's, see PUB 238 IE Articles 6B.6.1 and 6B.6.3 for the distribution of barrier dead load and the distribution of live loads, respectively.

When analyzing single-cell, cast-in-place box beams using an approximate method of analysis, use an effective flange width in accordance with AASHTO LRFD Article 4.6.2.6.2.

See PUB 238 IE Article 6.1.5.3I and Section 2.5.1.2 of this Manual for guidance on how to evaluate concrete box girder bridges with section loss and deterioration.

MBE Appendix A9 provides an example load rating of an interior prestressed NCABB using LRFR methodology.

Torsional moments applied to box girders are resisted by shear flow of the section. Figure 2.5.4.1-1 and Figure 2.5.4.1-2 provide a visualization of shear flow in single-cell and multi-cell box girders, respectively.

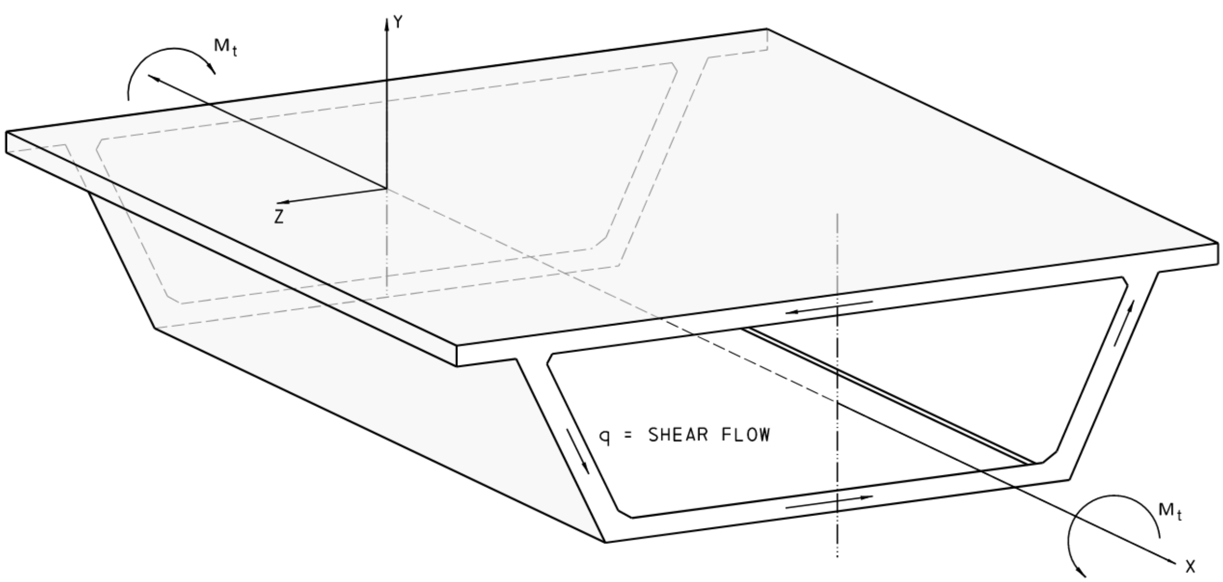



**Figure 2.5.4.1-1 – Direction of Shear Flow Resulting from Torsional Forces in Single-Cell Box Girder (Adapted from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

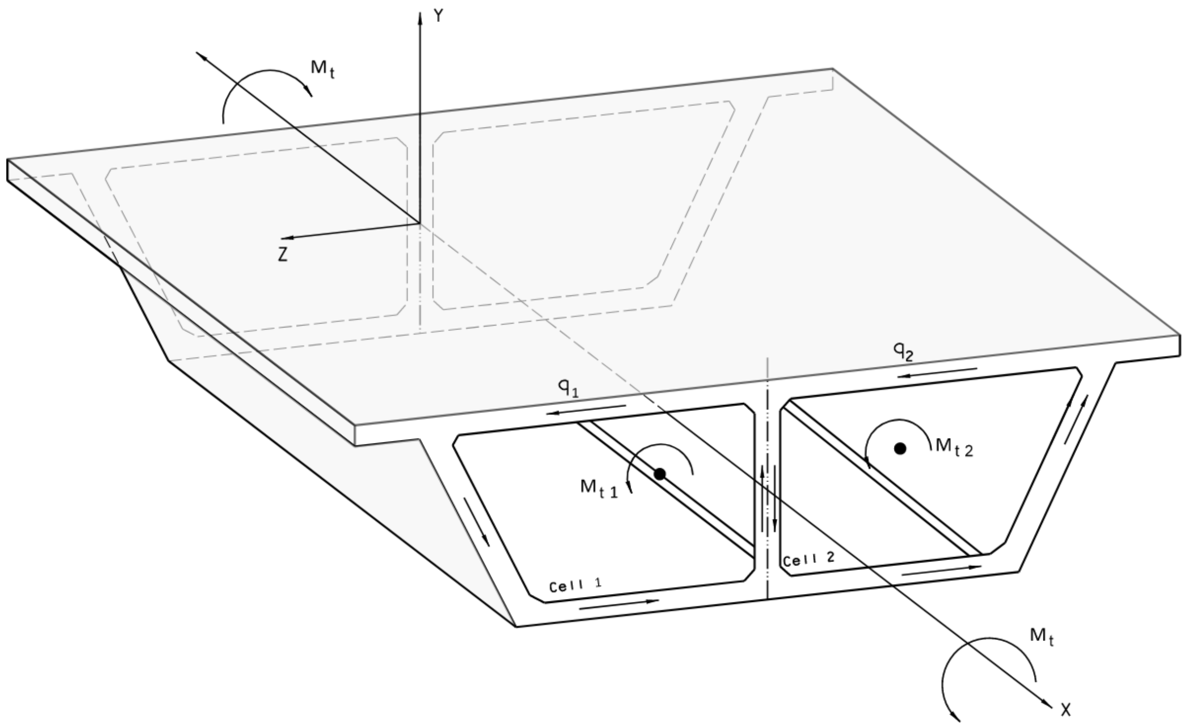



**Figure 2.5.4.1-2 – Direction of Shear Flow Resulting from Torsional Forces in Multi-Cell Box Girder (Adapted from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

Figure 2.5.4.1-3 shows how concentric shear load is distributed throughout the cross-section.

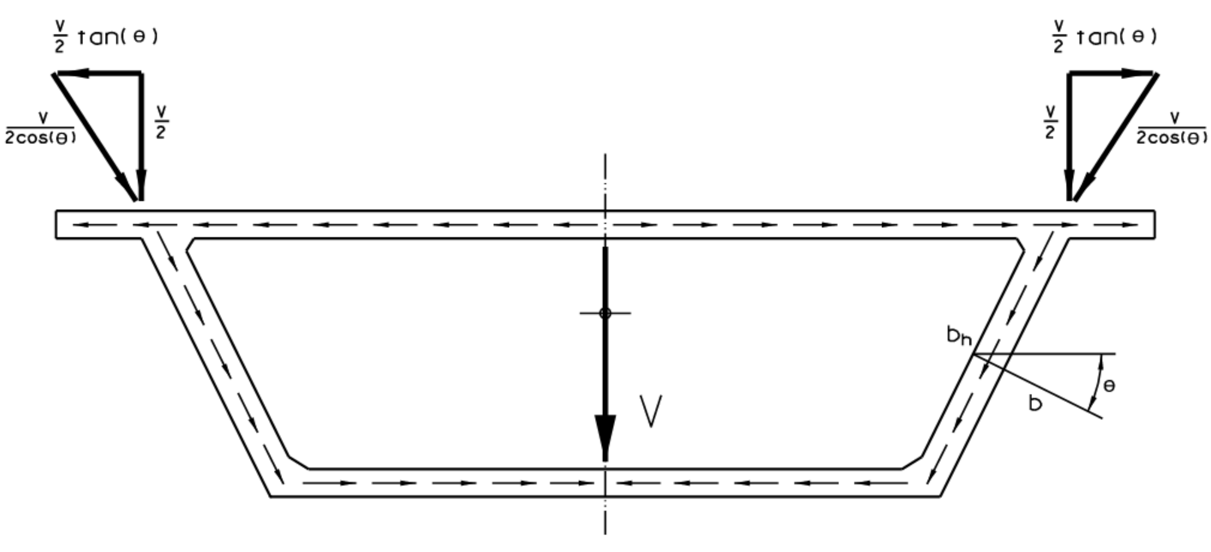



**Figure 2.5.4.1-3 – Distribution of Shear Force Through Box Girder (Adapted from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

#### 2.5.4.2 I-Girders

In lieu of other guidance, it is acceptable to apply the methodology from PUB 238 IE Article 6.1.5.3I when rating I-girders with measured deterioration, as mentioned in Section 2.5.1.1.

For reference, MBE Appendix A3 provides an example load rating of a simple span interior prestressed concrete girder using LRFR methodology.

### 2.5.5 Post-Tensioned Concrete Superstructures

PennDOT program's PS3 and PSLRFD do not have the built-in capability to evaluate post-tensioned members. Therefore, evaluations will need to be performed in spreadsheets or by refined analysis methods, such as AASHTOWare.

Load rating of post-tensioned segmental concrete bridges following LRFR should be determined in accordance with MBE Article 6A.5.11. AASHTO LRFD Article 4.6.2.9 provides some important considerations for evaluating segmental concrete bridges. Compressive and tensile stress limits are specified in AASHTO LRFD Article 5.9.2.3.

In addition to dead and live loads, the rating should include other loads such as permanent effects of phased construction, secondary effects from post-tensioning continuous structures, creep, shrinkage, and temperature, when appropriate. Refer to MBE Article 6A.5.9 for directions on how to handle secondary effects from post-tensioning. Gather information for time-dependent analyses that are part of the load rating, including relevant material properties as a function of time, construction method and loads, dates of key construction activities, post-tensioning strand stresses, and segment casting and erection for use in phased construction, if applicable. Use the actual concrete strength whenever possible because it can have a beneficial effect on the ratings depending on the checks that govern the capacity at the controlling limit state.

The analysis of a post-tensioned concrete bridge should include a longitudinal analysis, a transverse analysis, and, in some cases, an analysis of local details. The longitudinal analysis should be used to determine force effects along the length of the bridge and should account for time-dependent material properties and phased construction.

A transverse analysis is used to determine force effects at locations on the cross section, most often the top slab (deck), due to eccentric dead and live loads applied inside or outside of the superstructure. Note that MBE Article 6A.5.11.3 modifies the multiple presence factor for the single lane loaded case when evaluating the transverse operating rating of the top slab. For non-segmental concrete bridges, a transverse analysis is typically not required for load rating purposes unless the beam spacing is large or the beams are curved. Another jurisdiction recommends the rating engineer only conduct a transverse analysis when the distance between the extreme flange tip of a beam and the web face of an adjacent beam is greater than 13.5 ft. See Corven (2016) for more detailed explanations on how to perform longitudinal and transverse analyses of post-tensioned concrete bridges.

For post-tensioned segmental box girders, a transverse analysis of a unit length of the cross section should be used to determine force effects at key locations on the top slab and should account for time-dependent material properties. For the rating of a top slab in a typical post-tensioned segmental box girder, key locations often include:

- At the root of the cantilever wing
- At each interior face of the web
- At mid-span of interior slab(s)
- Any other taper points in the cantilever

Local details do not often need to be analyzed and load rated, except if there are signs of distress or other reasons to believe that they may control the ratings. Common local details encountered in post-tensioned bridges include dapped hinges, splices, interaction of transverse web flexure and longitudinal shear in box girders, diaphragms, anchorages, and expansion joint support members.

The following are key ratings to consider when evaluating concrete segmental bridges:

1. Service Limit State:
    - Longitudinal Girder Flexure
    - Principle Web Tension
    - Transverse Top Slab Flexure (Segmental)
2. Strength Limit State:
    - Longitudinal Girder Flexure
    - Web Shear
    - Transverse Top Slab Flexure (Segmental)

### 2.5.6 Arches

Arches may include cross walls, spandrel walls, and the arch itself. All three components will need to be rated if they are structural. Arches should be analyzed in accordance with AASHTO LRFD Articles 4.6.2.4 and 4.6.3.6, as applicable. Dead and live loads should be included in the analysis as described in Section 1.6. For closed spandrel arches with fill material, vertical and horizontal earth pressure loads (EV and EH) should also be included in the analysis. Other loads may need to be considered for arches as described in MBE Article 6A.5.10 and Sections 1.6.3.2 and 1.6.3.3 of this Manual.

Arches are susceptible to in-plane bending amplifications due to excessive deflections. For LRFR ratings, amplification factors can be evaluated following simplified methods included in AASHTO LRFD Articles 4.5.3.2.2b and 4.5.3.2.2c or with a refined analysis per AASHTO LRFD Article 4.5.3.2.3. For ASR or LFR ratings, amplification factors can be evaluated for simplified methods included in AASHTO Std. Spec. Articles 8.14.3 and 8.16.5. Arches are also susceptible to in-plane and out-of-plane stresses. Because of this, a refined analysis model will likely be required to determine loads for each concrete component of the arch.

Depending on the geometry, consider evaluating the rating of wide concrete members (e.g., solid arches) on a per foot of width basis. PennDOT's ARCH program is available to develop forces, though capacity and rating values would need to be computed outside the program. One option to evaluate the capacities and ratings would be to use Structure Point's spColumn program. Alternatively, spreadsheets could be developed to compute the ratings. If structural plans are not available, engineering judgement may be used.

Members of a concrete arch are typically subjected to combined axial forces and moments and, when true, should be evaluated in accordance with MBE Article 6A.5.7. Develop axial force (P)-flexure (M) interaction diagrams to determine capacities. For LRFR method, follow AASHTO LRFD Articles 5.6.3 and 5.6.4. For ASR method, follow AASHTO Std. Spec. Articles 8.14.3 and 8.15. For LFR method, follow AASHTO Std. Spec. Articles 8.14.3 and 8.16.

Follow the procedure described in MBE Appendix G6A to determine combined compression and flexural load ratings of concrete arch members. Also, consider ratings due to shear forces on the concrete members per MBE Section 6A.5.8.

#### 2.5.7 Box Culverts

This Section provides guidance on load rating concrete culverts. For a typical box culvert, perform the ratings for the top slab, bottom slab, interior wall and exterior wall. Evaluate the flexural, shear and axial load ratings for members as appropriate. In addition to dead and live loads, include other loads such as weight of fill, vertical and horizontal earth pressure, earth surcharge, water pressure and live load surcharge.

Refer to the provisions of MBE Article 6A.5.12 which applies to load ratings of reinforced concrete box culverts for LRFR ratings and MBE Appendix A10 which provides an example load rating of a reinforced concrete box culvert using LRFR methodology. Follow MBE Articles 6A.5.12.10.1 and 6A.5.12.10.2 for guidance on earth loads. The current earth loading conditions should be accounted for in the rating. The live load distribution should consider AASHTO LRFD Articles 4.6.2.10 and 3.6.1.2.6. Determine dynamic load allowance in accordance with AASHTO LRFD Article 3.6.2.2. MBE Article 6A.5.12.10.3 discusses specific live load related information for rating of top slabs and for analysis of loads in the direction of the box culvert span.

AASHTO Std. Spec. Articles 6 and 16 include specifications applicable to LFR load rating analysis of concrete culverts.

See Section 2.5.1.2 of this Manual for more information on box culverts with section loss and deterioration.

Perform load rating at several critical sections to determine the lowest load rating. Check culvert ratings at the following locations along the culvert length:

- Cross-section dimensions change
- Culvert span length change
- Location of maximum soil cover corresponding to maximum earth pressure load

- Location of minimum soil cover for maximum live load effects

PennDOT's program BXLRFD is useful for performing analysis and specification checks in accordance with the AASHTO LRFD and DM-4 for single-cell or multi-cell cast-in-place and precast box culverts and U-channels. BOX5 is another PennDOT program which can analyze and rate single-cell or multi-cell concrete box culverts as per AASHTO Std. Spec. (LFR and ASR method). AASHTOWare (BrR) is a 3D FEM program that may be used for LRFR and LFR load ratings of reinforced concrete culverts.

For calculation of earth load demand on the culvert per AASHTO LRFD Article 12.11.2.2.1, the choice of installation method (embankment or trench) does not have any impact on the rating consideration of culverts which have been in service for a long time. This is because the residual stresses developed from the installation will have dissipated and can be neglected at the time of rating. Follow guidelines from FHWA Reference Guide for Load Rating of Tunnel Structures (2019) for more information on technical aspects of rating existing concrete culverts such as analysis methods, rating requirements, section loss and defects. Figure 2.5.7-1 through Figure 2.5.7-3 shows live load distribution for different depths of fill.

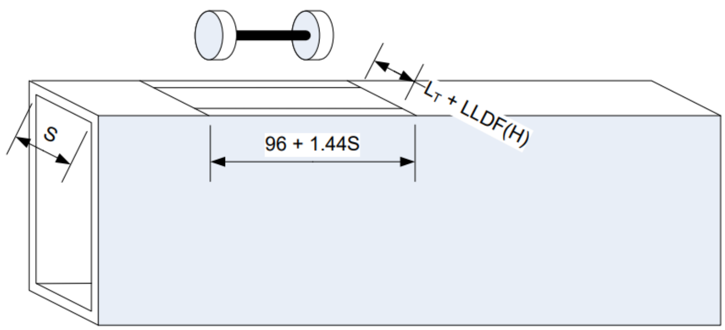



**Figure 2.5.7-1 – Live Load Distribution with Less than 2 ft. of Fill for Traffic Traveling Parallel to Span (Used with permission from *WisDOT Bridge Manual, Chapter 36 - Box Culverts*, developed by the Wisconsin Department of Transportation)**

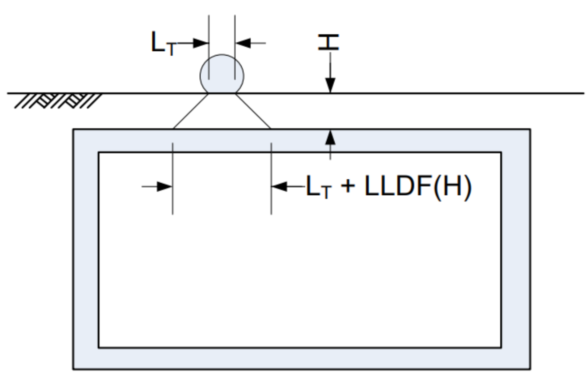



**Figure 2.5.7-2 – Live Load Distribution with Greater than 2 ft. of Fill for Traffic Parallel to Span (Used with permission from *WisDOT Bridge Manual, Chapter 36 - Box Culverts*, developed by the Wisconsin Department of Transportation)**

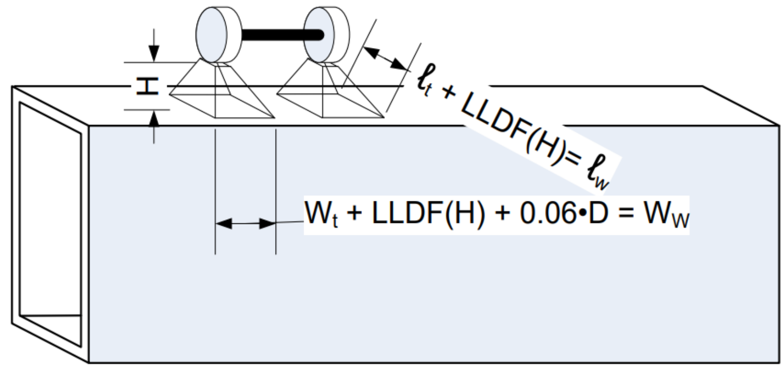



**Figure 2.5.7-3 – Live Load Distribution with Greater than 2 ft. of Fill for Traffic Traveling Parallel to Span with No Projection Overlap (Used with permission from *WisDOT Bridge Manual, Chapter 36 - Box Culverts*, developed by the Wisconsin Department of Transportation)**

## 2.6 STEEL STRUCTURES

This Section covers load rating best practices associated with steel superstructures. This includes topics such as material properties, section loss, fatigue, connections, and specific steel superstructure types. See Section 1.2 for load rating programs applicable to different steel superstructure types. See Sections 1.6 and 1.9 for design loads to consider and a discussion on capacity, respectively. See Section 2.4 for when to rate members of a steel structure. See Sections 2.11 and 2.12 for specific guidance on how to rate non-typical steel bridges and how to rate secondary steel members, respectively. Do not consider the wearing surface when determining capacities of steel composite members.

Do not consider the wearing surface when determining section properties and capacities of concrete members, but its dead load should still be considered in the evaluation. If the driving surface does not have a wearing surface, it is typical to ignore the top 1/2 inch of deck thickness when calculating section properties.

### 2.6.1 Material Considerations

#### 2.6.1.1 Material Properties

The most common steel material properties used in load rating include yield strength ($F_y$), tensile strength ($F_u$), modulus of elasticity ($E$), and, in cases where thermal loads are considered, coefficient of thermal expansion.

See Section 1.10 for best practices to determine material properties of structural members. If plans are not available or steel material properties are unknown, refer to MBE Articles 6A.6.2, 6B.5.2, and 6B.5.3. Specifically, material properties can be determined from the following:

- Typical Structural Steel
    - MBE Table 6A.6.2.1-1 (LRFR)
    - MBE Tables 6B.5.2.1-1 through 6B.5.2.1-2 (ASR, LFR using “date built” columns)

- Pins
  - MBE Table 6A.6.2.2-1 (LRFR)
  - MBE Tables 6B.5.2.1-1 through 6B.5.2.1-2 (ASR, LFR using "date built" columns)
- Wrought Iron
  - MBE Article 6A.6.2.3 (LRFR)
  - MBE Article 6A.6.2.3 (LFR) and Article 6B.5.2.2 (ASR)
- Rivets
  - MBE Table 6A.6.12.5.1-1 (LRFR)
  - MBE Table 6B.5.2.1-3 (ASR) and MBE Article 6B.5.3.1 and Table 6B.5.3.1-1 (LFR)
- Bolts
  - MBE Table 6A.6.12.6.2-1 (LRFR)
  - MBE Tables 6B.5.2.1-3 through 6B.5.2.1-4 (ASR) and MBE Article 6B.5.3.1 and Table 6B.5.3.1-1 (LFR)

When referencing ASTM or AASHTO designations, conservatively consider the minimum strengths, as applicable.

If all other means of determining the material properties have been exhausted to no avail or if precise material properties with a high-level confidence are required, perform material testing in accordance with MBE Articles 5.3 through 5.6.

#### 2.6.1.2 Defects and Section Loss

Typical PennDOT load rating submissions require only As-Inspected ratings. Review the latest inspection report to determine the components to evaluate based on the section loss noted and determine the amount of section loss to consider. After comparing new and existing section loss measurements, if no appreciable change is found, re-rating may not be warranted. For additional re-rating considerations, see Sections 1.8.2 and 1.8.4. Clearly document all assumptions corresponding to how section loss is implemented in the analysis to reduce the member capacity.

See MBE Article 4.3.5.6 and PUB 238 IE Article 4.3.5.6 for more discussion on the inspection procedures for steel superstructure components such as beams, trusses, connections, diaphragms/cross frames, bracing, pins and hangers, and bearings. These articles also provide more insight into what to pay particular attention to in the inspection report and photos. There are also discussions on what certain section loss or defects may indicate. See MBE Article C6A.6.5 for a discussion on how section loss severity and location affect tension members including eyebars and pin plates, compression members, built-up members with deteriorated lacing/batten bars, flexural members, and main truss gusset plates. Section loss and defects in non-redundant steel tension members (NSTM), which include tension members or members with tension elements such as those discussed in MBE Article 4.2.3.4 and connections to NSTMs such as gusset plates, are discussed later in this section.

If section loss is reported for a member, consider the critical stress locations along the member length and what will lead to the lowest ratings. If multiple locations of section loss are reported for one member and if it is initially unclear what section loss will control the ratings, multiple evaluations may be required. Depending on where the section loss is located, it can be overly conservative to consider localized section loss over the entirety of a steel component. This is especially concerning when utilizing a program that does not allow the user to input localized section loss. Be aware that PennDOT's BAR7 (which is used to determine ASR or LFR load ratings) program will allow varying section properties along the length of a member for most bridge models, but not all and it does not include inputs for localized section loss on a portion of a component in its cross section. Multiple evaluation runs may be required to evaluate different portions of the same member depending on model type selected. Because these programs have no input for localized section loss, the appropriate cross-section dimensions must be reduced to account for section loss. Alternatively, if necessary, AASHTOWare (BrR) may be used to input localized section loss for LRFR, LFR and ASR ratings. BrR is a 3D FEM program that can better capture the effects of localized imperfections. PennDOT's STLRFD program, which can be utilized for LRFR load ratings of flexural members, has a section loss tab that allows the user to define up to forty (40) section loss locations for the member being analyzed.

In the event that a vehicle collides with the bridge, damaged steel members must be evaluated based on the observed and measured section loss. Particularly when girders and truss members are struck by a vehicle traveling below or on the bridge, attention must be given to lateral deformation (see Figure 2.6.1.2-1). Damage may not need to be accounted for in a load rating if permanent localized (e.g., bulging, tearing) and global (e.g., twisting, buckling) deformations are found to be minor. If the permanent deformation is severe, a refined analysis may be required to capture the globally deformed shape with eccentric loading or loss of capacity due to local deformation. Further investigation may be required if cracking in welds connecting to primary members are found. If vehicle impact causes a primary member fracture or if deformations are likely to result in failure of the member, these scenarios are beyond the scope of this manual and the District Bridge Engineer should be notified immediately.


**Figure 2.6.1.2-1 – Example of Beam Distorted Due to Vehicular Collision**

See Section 1.9 for discussion on how and when to further reduce the capacity of the section based on its condition.

*Flexural Members – General*

Consider section loss in webs of steel flexural members when determining controlling shear ratings, especially near supports where shear is greatest. See Figure 2.6.1.2-2 for an example of stringer web section loss in an area where large shear force is expected. If BAR7 is used as a tool to determine ratings, web section loss at low shear locations (mid-span) should not be spread across the entire beam length. It is overly conservative to do so.

Section loss to all components of steel flexural members have an impact on controlling flexure ratings of flexural members, but which component's section loss has the most critical effect is not always apparent. Therefore, several runs and/or models may be required to accurately load rate a structure with section loss to determine the controlling rating for each load effect.


**Figure 2.6.1.2-2 – Section Loss in Stringer Web at Support**

A sensitivity analysis may be required to determine how sensitive the member capacity is to different section loss assumptions. To conduct a sensitivity analysis, change one variable (e.g., flange thickness or web thickness) and keep all other variables fixed. See how changing that variable affects the ratings. Then, move onto the next variable and follow the same procedure to better understand what variables have the most effect. If changing a variable significantly affects the controlling ratings, the ratings are said to be very "sensitive" to that variable. Sensitive variables must be handled with care. How section loss on this variable is accounted for in the load rating evaluation will have a significant impact on the results. It may be appropriate to reconsider section loss assumptions to obtain the most realistic results. This is an example of why clearly documenting assumptions is important.

When there is no input for local section loss in the web, such as when BAR7 is used for rating, its effects must be approximated. The following are two approaches to modeling section loss that is not constant along the web depth:

1. Average Thickness Method

   This is the most common method and should be checked first. The local section loss is spread across the entire web height and, thus, an average, reduced web thickness is determined. For BAR7 input, the average reduced web thickness is input along with the as-built web depth.

2. Modified Depth Method

Instead of spreading the section loss across the entire beam depth, simply reduce the beam depth by the height that is deteriorated. For most cases, this method will result in a lower rating than the Average Thickness Method, but there are some situations where it is more applicable. This method should be used when the ratio of the beam depth to the average reduced web thickness ( $D / t_{w\_reduced}$ ), as determined using the Average Thickness Method, controls the rating and otherwise would not control the rating in the As-Built condition. It may also be appropriate when the web buckling coefficient, C (per AASHTO Std. Spec. Article 10.48.8 and AASHTO LRFD Article 6.10.9.3.2) is less than 1.0 based on this ratio. If the section loss is severe and localized, such as a hole, it may be more appropriate and representative to use the Modified Depth Method. If the severe section loss is located at the bottom of the web at a support, web local yielding and crippling will be a concern. For more discussion on these failure modes, see below.

For BAR7 input, the as-built web thickness is entered along with the reduced web depth. Be aware that reducing the web depth will reduce the overall beam depth, which will inappropriately reduce the flexural capacity. Because of this, a separate run using the Average Thickness Method should be performed to determine the controlling flexural load rating.

If it is unclear which method to utilize, determine capacities following both methods to envelope the solution to compute a more accurate load rating. Also, there are situations, such as a hole in the web and minor section loss along the rest of the web, where both methods can be applied to the same cross-section within the load rating analysis run.

Local section loss in flanges is often spread across the cross-sectional width, similar to the Average Thickness Method explained above. There are two options for inputting the reduced flange thicknesses into BAR7.

1. Reduce the flange thickness

   Use the plate girder input and reduce the flange thickness accordingly. If the as-built steel member is a plate girder, this is a simple update to the flange thickness input. If the as-built steel member is a rolled shape, the girder dimensions will need to be input individually as a plate girder instead.

2. Input a negative cover plate

   This is only applicable to rolled shapes. Instead of inputting the girder dimensions as a plate girder, the user may elect to consider the as-built rolled shape with a negative cover plate. This method is applicable when the section loss is on the outer flange faces. BAR7 considers a positive value cover plate thickness on the bottom flange to start at the bottom of the bottom flange and extend downwards. Similarly, the program considers a positive value cover plate thickness on the top flange to start at the top of the top flange and extend upwards. BAR7 calculates section properties internally, so if the cover plate thickness is negative, the program will remove that area from the corresponding flange area.

However, if the section loss is severe or if there are large holes, it may be more appropriate to reduce the flange width instead. This will likely result in an asymmetric section, and it may be more appropriate to consider a different shape entirely when determining capacities. Asymmetric section loss may also induce load eccentricities that are not typically considered. Conventional methods of calculating capacities may no longer be applicable and a refined analysis will be required.

If the steel member has severe section loss or defects and appears visually unable to resist loads as intended, it should have been reported to the District Bridge Engineer immediately by the inspectors. However, for evaluation, it may be appropriate to consider the entire steel member ineffective and re-evaluate assuming it is providing no resistance or stiffness in the model, but it's dead load should still be included. If a steel member is considered ineffective and removed from the analysis, adjacent members must be evaluated to ensure they can adequately resist the additional load demand. Also, the deck will need to be evaluated to ensure it has adequate capacity to span over the "removed" member. Typical load rating programs (STLRFD, BAR7) and procedures are not applicable to steel members with severe section loss or defects as the methods utilized assume typical singly or doubly symmetric sections.

Severe section loss at the web/flange interface along a significant length may indicate a loss of connectivity and load transfer between web and flange. This is also true when there is severe web section loss adjacent to a support connection. Figure 2.6.1.2-3 shows a floorbeam that has web holes adjacent to its flanges and its connection to the girder. The web does not appear capable of properly distributing load to its flanges and girder. Refined analysis methods will be required to evaluate these steel members. This topic is considered to be outside the scope of this Manual.


**Figure 2.6.1.2-3 – Example of Severe Section Loss in Web Adjacent to Flange**

*Flexural Members – Beam Ends*

When evaluating steel beam ends above a support with section loss, adhere to the following guidelines:

- Beam webs with bearing stiffeners in good condition, that do not exceed slenderness limits and extend full height are considered stiffened. Web local crippling and web local yielding do not need to be checked.

- Beam webs with bearing stiffeners in good condition, that do not exceed slenderness limits, and are in contact with the loaded flange and extend at least 75% of the web depth are considered stiffened and are not susceptible to web crippling (Salkar et al. 2015). Web local yielding should still be checked.

- Beam webs with bearing stiffeners in good condition, that are in contact with the loaded flange, and extend between 50% and 75% of the web depth, are susceptible to web crippling. To determine the web crippling resistance for this condition, follow the guidelines presented in Salkar et al. (2015).

- Otherwise, the girder web is considered unstiffened and susceptible to web local yielding and crippling.

When checking the bearing stiffener height for the conditions above, it may be appropriate to discount the portion of bearing stiffener that has severe section loss. Minor section loss should be accounted for when checking bearing stiffener slenderness limits.

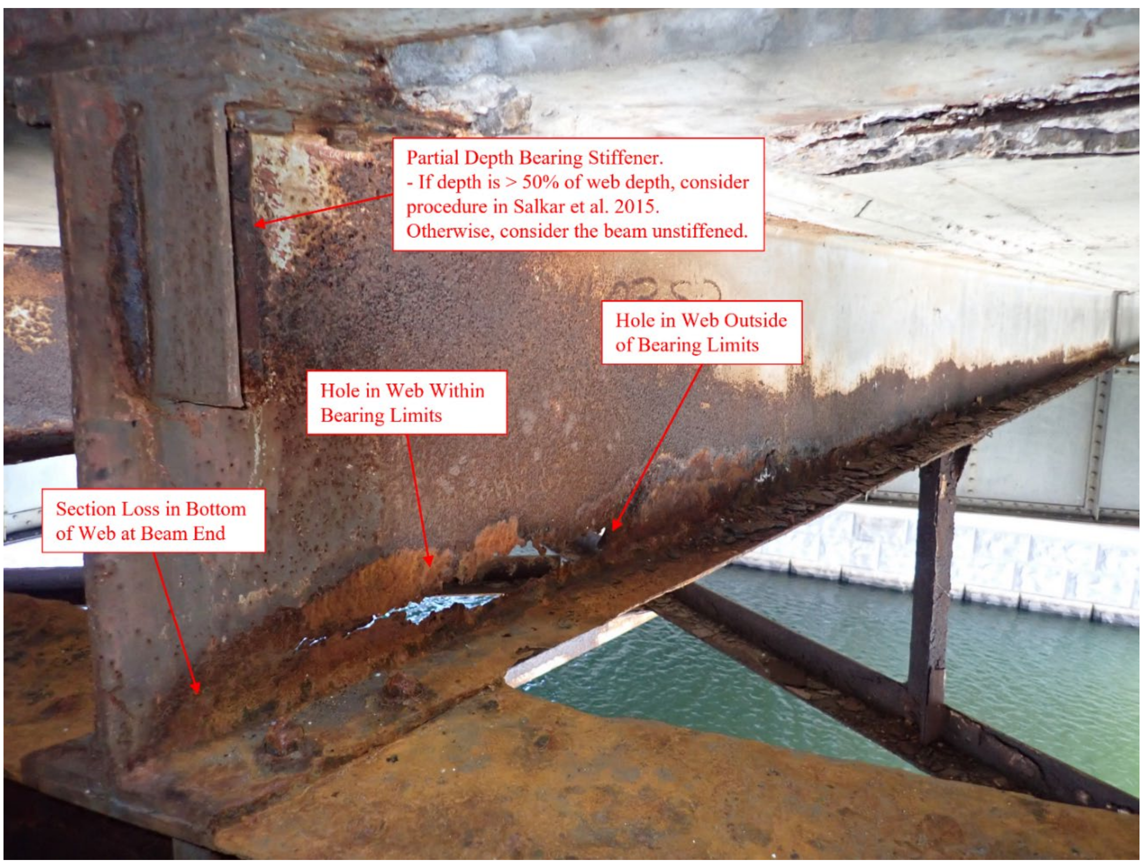



**Figure 2.6.1.2-4 – Example of Section Loss in Unstiffened Beam Web at Concentrated Load**

Section loss in the web and bottom flange of unstiffened beams directly above a support are especially concerning for steel flexural members (see Figure 2.6.1.2-4). AASHTO LRFD Article D6.5 provides guidance on how to evaluate unstiffened girders susceptible to web local yielding and crippling following LRFR method. For ASR method, follow AISC Steel Construction Manual Article J10-2 and J10-3. AASHTO LRFD Article D6.5 may be used for LFR load rating method if the resistance factors are not applied and the LFR load factors are applied. The important variables in the equations that are affected by section loss are the web depth (D), the web thickness ($t_w$), flange thickness ($t_f$), length of bearing (N), distance between outer face of flange/bearing to the web toe of fillet (k), and the web yield stress ($F_{yw}$). See Figure 2.6.1.2-5 for an example schematic of a girder with these components labeled.

Because this is a check of the web bearing on the bottom flange above the bearing, web and flange thicknesses immediately above the bearing are critical. Use engineering judgement to determine the height of web considered for the evaluation. As an example, another jurisdiction advises load raters to consider a web height within the bottom 4” above the bottom flange at the bearing to determine the remaining web thickness in their evaluations. Even in this case, the jurisdiction advises to use engineering judgement if advanced section loss occurs above the 4” height. Conservatively consider the minimum web and flange thicknesses over the bearing length. If this results in poor ratings, an average thickness may be appropriate if the range of section loss is reasonable. If there is a hole in the web within the bearing length, as shown in Figure 2.6.1.2-5, reduce N by the hole length. The hole indicates the web has a reduced length available to transfer the concentrated load, so it is accounted for by reducing N. Determine the web thickness based on the web remaining over the bearing length. Web holes in the beam end outside of the bearing influence (“X”k+N), do not have an impact on the web local yielding and local crippling capacity above the bearing (Gerasimidis, et al. 2019). Webs with large holes, such as ones that occupy a significant portion of the bearing length, require a refined analysis to evaluate and are beyond the scope of this Manual. The District Bridge Engineer should be notified immediately.

Flange section loss decreases the flange thickness, but depending on the severity of section loss, it may also decrease the overall bearing area of the flange. Severe section loss, such as holes in the flange will decrease $t_f$, k and/or N values. BAR7 and STLRFD can generate loads at supports, but BAR7 does not have the capability to evaluate web local yielding and crippling. STLRFD can only evaluate the required N value for rolled shapes given an applied load. Therefore, for the most part, capacity calculations and ratings will need to be developed outside of the program.

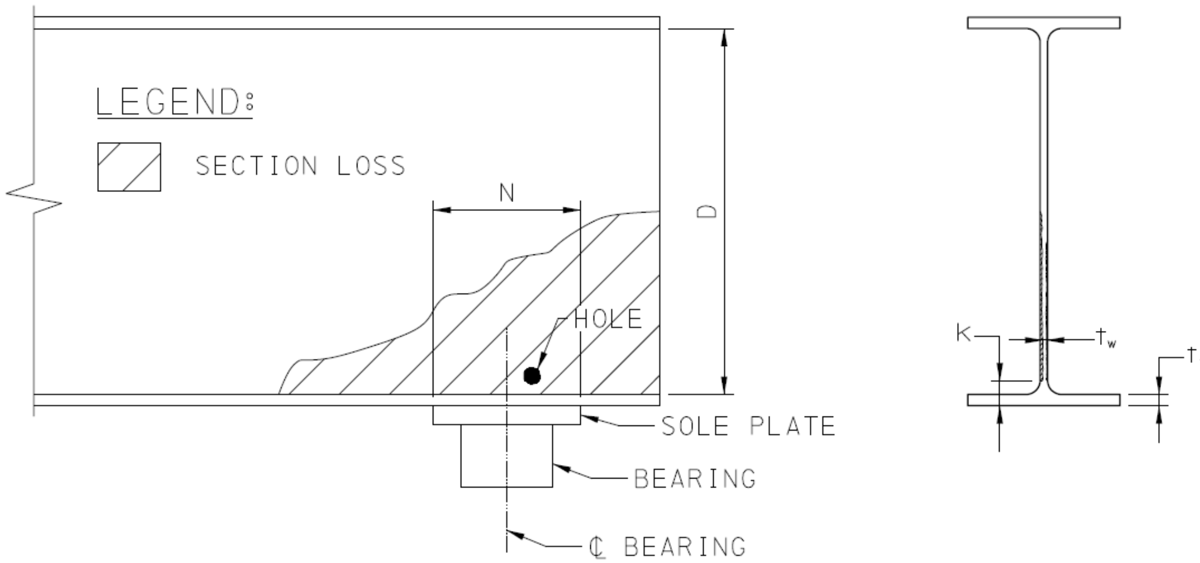



**Figure 2.6.1.2-5 – Variables for Determining Web Local Yielding and Web Local Crippling**

*Truss Members*

See MBE Article 4.3.5.6.6 and PUB 238 IE Article 4.3.5.6.6 for inspection techniques and defect concerns related to truss members. Because truss members are primarily loaded axially, section loss and defects are important regardless of where they are located along the truss length. In order to account for localized section loss, it is typical to determine an average thickness following a method similar to the "Average Thickness Method" detailed earlier in this Section. If BAR7 is used to determine the ratings, equivalent section property inputs considering the reduced thickness(es) will need to be calculated outside of the program.

Typical load rating programs (BAR7) and procedures are not applicable to steel members with severe section loss or defects as the methods utilized assume typical singly or doubly symmetric sections. Severe section loss typically leads to asymmetric members and possibly a different shape entirely depending on the section loss location and severity. A refined analysis will be required to evaluate such truss members. This topic is considered to be outside the scope of this Manual.

Truss member distortion indicates compression concerns that may lead to failure. If there is significant distortion, BAR7 and typical hand calculations will no longer be applicable. The District Bridge Engineer must be notified and a refined analysis must be used to evaluate the truss member. This topic is considered to be outside the scope of this Manual.

Lacing bars and batten plates that are missing or severely deteriorated have a major impact on truss local buckling capacity. MBE Article C6A.6.5 and Kulicki et al. (1990) provide guidance on how to evaluate built-up compression members with deteriorated lacing bars and batten plates.

See Section 2.6.3 for more discussion and best practices for load rating truss members.

Non-redundant steel tension members (NSTM)See MBE Article 4.3.7 and PUB 238 IP Article 2.4 for a list of NSTM, details on how important the inspection process is for them and the section loss and defects that are especially concerning for these members. Weld or base metal cracks and defects in the tension zone of NSTM are critical. For more information, MBE Article 4.3.5.6.10 and PUB 238 IP Article 2.4.11 discuss the inspection of welded connections.

PUB 238 IP Article 2.4.10.1 and PUB 238 IE Article 4.3.5.6.6 provide information on inspection procedures and notable section loss and defects for truss gusset plates. Figure 2.6.1.2-6 shows an example of section loss in a truss gusset plate. When evaluating gusset plates, section loss that intersects potential failure planes must be considered. Because the severity of section loss is based on its location and the failure mode considered, do not use an average thickness over the entirety of the gusset plate. Instead, follow the "Average Thickness Method" detailed earlier in this Section. An average gusset plate thickness within the failure plane is more appropriate. More discussion on load rating best practices for gusset plates and their failure planes are presented in Section 2.6.7.1.

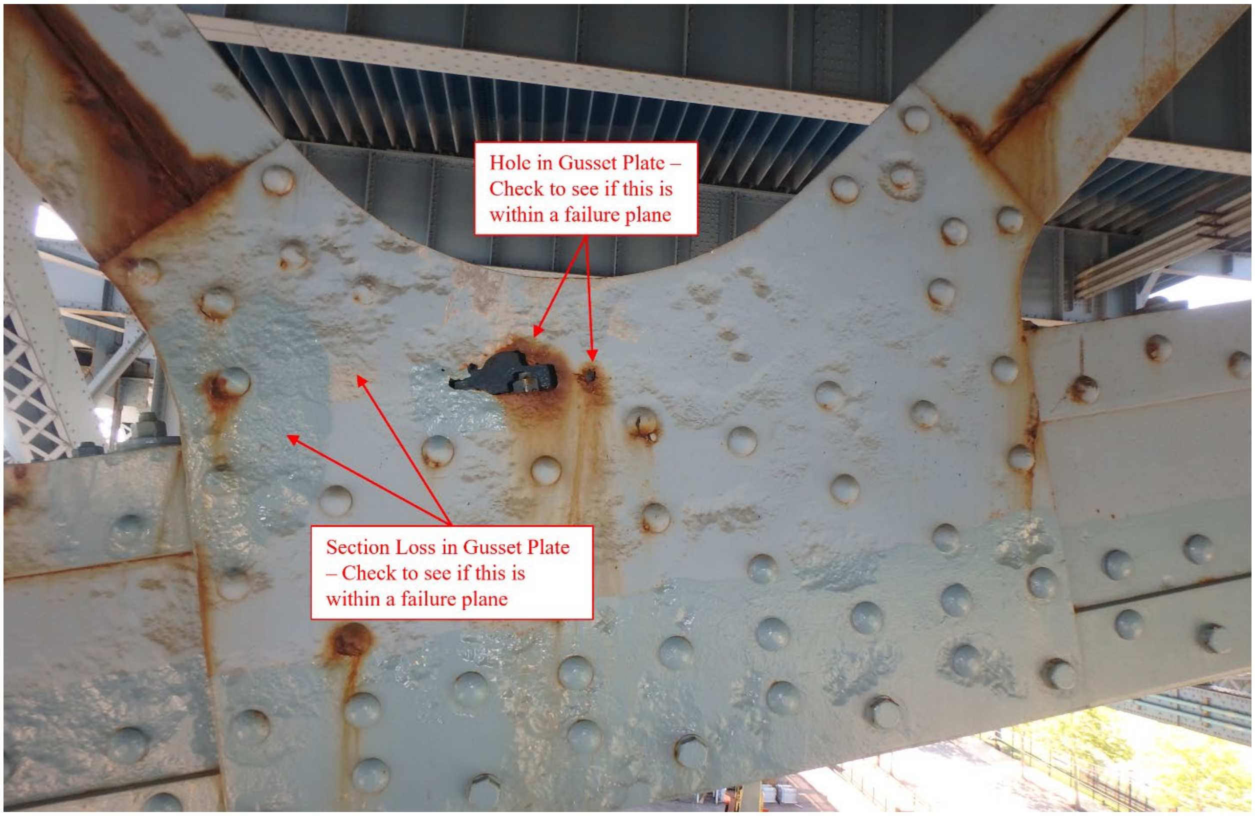



**Figure 2.6.1.2-6 – Example of Section Loss in Gusset Plate**

See MBE Article 4.3.6.2 for inspection details of eyebars in suspension bridges. MBE Article C6A.6.5 includes a discussion on how section loss severity and its location affects eyebars, pins, and hangers. Loose eyebars indicate a loss in tension and should be considered ineffective and removed from the evaluation. Cracked eyebars should be handled similarly. These defects, when found in NSTMs, indicate the bridge is susceptible to a sudden collapse. Therefore, notify the District Bridge Engineer immediately as lane restrictions or even full bridge closure may be required.

PUB 238 IP Article 2.4.10.2 and PUB 238 IE Article 4.3.5.6.11 provide information on inspection procedures and notable section loss and defects for pin and hanger assemblies. Section loss and defect concerns for pin and hanger assemblies are discussed in MBE Article C6A.6.6.1.

The District Bridge Engineer must be notified immediately if a NSTM load rating is below 1.0 operating or if there is concern that the NSTM is unable to act as intended.

*Metal Culverts*

See PUB 238 IP Article 2.5.2 and MBE Article 4.3.5.10 and PUB 238 IE Article 4.3.5.10 for discussions on inspection techniques, section loss, and defects associated with metal culverts such as metal box, corrugated metal plate (CMP), circular metal pipe, metal pipe arch, and metal arch culverts.

Section loss (particularly along longitudinal seams with missing/loose fasteners) and defects, including deflections or changes to the original global cross-sectional geometry, in metal culverts should be evaluated per NCSPA (1995) as these issues could lead to failure for the structure. Deflection indicates a loss of buckling strength and NCSPA (1995) provides guidance on how to reduce the culvert's capacity to account for this. See Section 2.6.6 for more discussion on load rating metal culverts.

In addition to what has been discussed in this section, Ream et al. (2019) provides some guidance for how to handle section loss and deterioration in steel tunnels and culverts.

*Miscellaneous Topics*

- Steel surface corrosion and pitting are not typically concerns and should not be considered as section loss unless there is measurable deterioration.

- Section loss of the concrete haunch and/or connection between the steel member and deck (deteriorated deck around shear studs, loss of clips attaching to timber deck, etc.) may indicate the top flange is no longer braced along that length. Increase the top flange unbraced length to account for this section loss. This may also indicate a loss of composite action between the two materials, which must be verified in the analysis.

- When evaluating concrete encased steel I-girders, consult the inspection report to determine the condition of the concrete. If there are signs of deterioration (cracks, unsound concrete, exposed steel) or separation between steel and concrete that may indicate loss of composite action between steel and concrete, the member should be considered non-composite. For additional information on concrete encased steel members, refer to Section 2.6.2.1.

- If the steel beam has at least one cover plate, check the quality of the welded connections between the flanges and the cover plates in the inspection report. Poor-quality welds indicate a loss of connection between the two members, and, thus, the cover plate should not be considered in section properties over that length.

- Section loss in transverse stiffeners does not typically need to be considered in load rating. However, if the transverse stiffener is completely lost or has lost the ability to stiffen the web, adjust the transverse stiffener spacing accordingly assuming that deteriorated stiffener is not there.

- Section loss in bolt/rivet heads may not need to be considered when checking pure shear in bolted/riveted connections. However, it is likely that the connection is still in-tact due to rust build-up. Therefore, the connection should not be relied upon long-term. Section loss in bolt heads indicates a loss in pretensioning, which means the bolt no longer has the ability to reliably contribute to the slip-critical connection resistance. These bolt/rivet groups should not be considered effective in resisting direct tension or tension that would result from applied moments. However, section loss to bolt/rivets should be considered when evaluating gusset plate connections.

- Depending on the severity and location of section loss, as-built end conditions of compression members may no longer apply. Members with as-built fixed end conditions are more susceptible to changes.

- Nicks and gouges in tension members can cause stress concentration concerns that could lead to long-term fatigue issues. This is not something to be included in a load rating, but it should be noted and monitored.

#### 2.6.1.3 Fatigue

Although not part of the load carrying capacity of the structure or required for a load rating, an evaluation of fatigue prone details on steel primary members with elements in tension is sometimes performed with a load rating analysis. Directly related to a load rating though, is the presence of cracks caused by fatigue, either load-induced or displacement-induced. If cracks are in primary members, their capacity may be affected, resulting in a lower load rating. In most cases, the ends of cracks are drilled out with crack arrest holes, which may further affect the member capacity. Load rating members with severe cracking is outside the scope of this document.

If performing a fatigue evaluation, refer to MBE Article 7 and DM-4 PP Article 5.1 (LRFR). Additional guidance is provided in DM-4 Article 6.6. Be aware that the outdated LFD method for fatigue evaluations is no longer used in PA except under special circumstances.

Several PennDOT programs are capable of performing fatigue evaluations for steel members. If these programs produce poor fatigue life results, consult DM-4 Article 5.1.1.1.4 that describes a quick method for reducing the conservatism of a design fatigue life evaluation that is used in those programs. Similar to load rating analysis, live load distribution in a 1D program is conservative, another option to likely improve a poor fatigue evaluation would be using a refined analysis to determine the live load fatigue stress. A more costly option for a poor fatigue life could be a strain gage analysis to determine stresses at fatigue prone details. Additional refinements can be achieved by load testing to determine live load stresses at fatigue prone details. Before any refinements to the approach outlined in this section are made, the PennDOT Assistant Chief Bridge Engineer - Inspection should be consulted and approval received.

### 2.6.2 Girder Superstructures

This section provides guidance for load rating longitudinal steel girders such as, I-girders, rolled beams, box and tub girders. Below guidance is specifically applicable to girder analysis methods and girder behavior.

#### 2.6.2.1 Steel I-Girders

This section is applicable to load rating analysis of steel girders consisting of either wide flange beams or built-up girders with flange and web plates welded, riveted or bolted together. These girders can be a part of a multi-girder bridge, two-girder bridge, girders with sub-stringer systems or through girder bridge. Evaluate the flexure and shear ratings of steel I-girders. Depending on the evaluation methodology, perform flexural ratings taking into account the provisions of MBE Articles 6A.6.9 for LRFR, 6B.5.2.1 for ASR, and 6B.5.3.1 for LFR. MBE Appendix L6B provides a summary of references from AASHTO Std. Specs. applicable to girder evaluation.

Analyze steel I-girders in accordance with the MBE and any other governing specification depending on the rating methodology (e.g., AASHTO LRFD, AASHTO Std. Specs.). General analysis guidance for common I-girder types is as follows.

- Straight non-skewed multi-girder bridges: 1D line girder analysis with distribution factors. Curved steel I-girder bridges meeting the requirements of AASHTO LRFD Articles 4.6.1.2.4b can be analyzed as straight girders using a simple line girder analysis as discussed in Section 1.7.1.1.

- Curved and skewed multi-girder bridges: 1D line girder analysis with distribution factor for skews ranging from 90° down to 70° (PennDOT skew angles) or 2D (i.e., grillage, grid, or PEB) or 3D analysis for lesser skews. On curved multi-girder bridges, the girder on the outside of the curve will typically control the load rating. See Section 2.2.2 for additional information. Refer to AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges with Design Examples for I-Girder and Box-Girder Bridges and AASHTO/NSBA guidelines for Steel Girder Bridge Analysis for more information on analysis of curved girders.

Refer to PUB 238 IP Article 3.3.2.1 for limitations of using the standard AASHTO distribution factors.

For simplified verification of curved multi-girder bridges, 1D analysis methods can be combined with approximate, hand analysis methods such as the V-Load method for analyzing horizontally curved I-girder bridges. Typically, this method is suitable for simple girder configurations (such as constant girder spacing) and can be less accurate for more sophisticated framing systems. Nevertheless, this hand-analysis method is a valuable tool for rating engineers to understand the complex behavior of curved systems, checking girder analysis and verification of results. For more information on the V-Load method, refer to AASHTO LRFD Article 4.6.2.2.4 and commentary; AASHTO/NSBA, 2019; and Coletti et al., (2005).

Refer to Section 1.7.1.1 for more information about different types of analysis.

Load rating of typical steel multi-girder bridges can be performed using the software listed below:

- PennDOT's STLRFD program can rate steel girders using LRFR methodology. STLRFD can evaluate beams for simple span or continuous spans using 1D simplified analysis. STLRFD's User Manual has detailed information on its capabilities, method of evaluation, input and output description, etc.

- PennDOT program BAR7 can analyze simple or continuous span straight steel multi-girder bridges per LFR method using 1D girder analysis.

- AASHTOWare program can perform analysis and specification checking of steel superstructure according to LRFR, LFR and ASR using 3D FEM analysis. It is suitable for straight and curved steel girder bridges.

Check interior and exterior girder ratings at the following locations:

- Cross-section change nodes such as cover plate cut-offs and flange plate transitions

- Support locations and beginning of unstiffened regions

- Brace points such as cross-frames or diaphragms and mid-point between the brace points

- Locations with section loss

- Unique locations such as change in superimposed load

See Section 2.6.1.2 for guidance on how to evaluate steel girder bridges with section loss and deterioration. MBE Appendix A5 provides an example load rating of a four-span continuous steel plate girder bridge using LRFR methodology.

Steel girders designed as simply supported for dead load and continuously supported for live load should be evaluated accordingly. See Azizinamini et. al. (2005) for more discussion on analysis of these girders. If continuity is completely or partially lost at interior supports, consider analyzing as simply supported for composite dead load and live loads.

In addition to major-axis bending stresses and shear stress which are typical of straight girders, curved and skewed bridges include St. Venant shear stress and warping normal stresses due to torsion. Some important considerations in I-girder behavior are discussed below:

- Torsional behavior: Due to the open cross-section, I-girders have low St. Venant torsional stiffness and resist torsion by warping. For the I-section, an internal couple due to the warping shear in the girder flange resists the warping torsion. When analyzing skewed and/or curved I-girders using a 2D grid analysis or PEB analysis, an equivalent St. Venant torsion constant should be used to prevent underestimation of the girder torsional stiffness. See DM-4 Article E6.2.2P, AASHTO LRFD Article 4.6.3.3.2, and White et al. (2012) for additional guidance.

- Flange lateral bending: Flange lateral bending stresses are due to out of plane bending of the flange. These stresses exist due to curvature of alignment, wind loads, seismic loads and cantilever loads due to formwork on exterior girders. For ratings, flange lateral bending stresses due to wind load and construction are not required. Lateral flange bending should be investigated in straight bridges with discontinuous cross-frames or diaphragm lines and skews less than 70 degree per MBE Article C6A.6.9.1. Calculation of flange lateral bending effects should follow the provisions in DM-4 Articles E6.1.2.3P and E6.1.3.1P

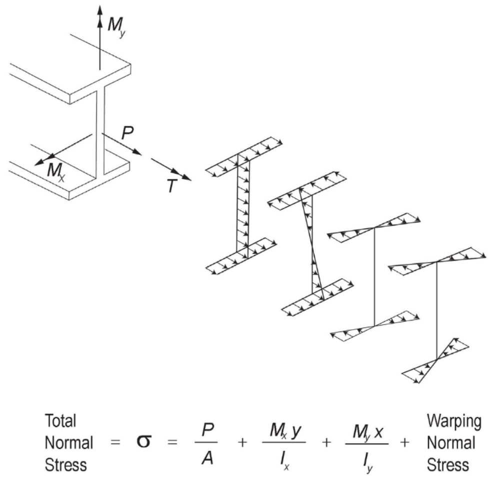



**Figure 2.6.2.1-1 – General I-Girder Normal Stresses (Used with permission from *Guidelines for Steel Girder Bridge Analysis, G13.1-2019*, developed by the AASHTO/NSBA Steel Bridge Collaboration)**

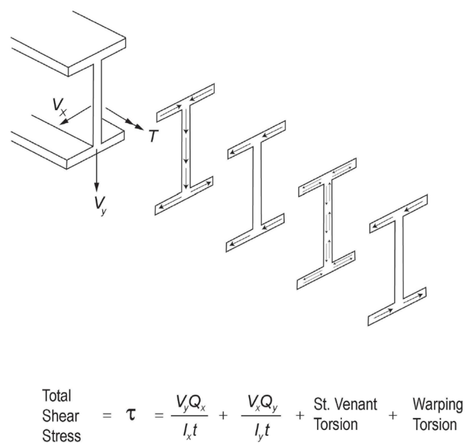



**Figure 2.6.2.1-2 – General I-Girder Shear Stresses (Used with permission from *Guidelines for Steel Girder Bridge Analysis, G13.1-2019*, developed by the AASHTO/NSBA Steel Bridge Collaboration)**

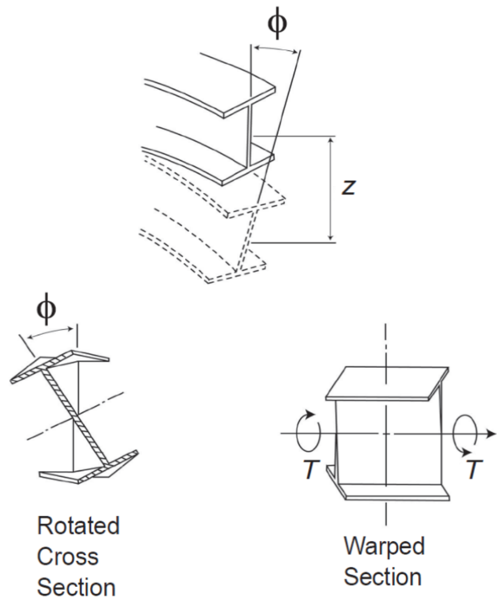



**Figure 2.6.2.1-3 – I-Girder Deformation (Used with permission from *Guidelines for Steel Girder Bridge Analysis, G13.1-2019*, developed by the AASHTO/NSBA Steel Bridge Collaboration)**

The total state of normal stresses and shear stresses in an I-girder are shown in Figure 2.6.2.1-1 and Figure 2.6.2.1-2 respectively. The normal stresses in an I-girder are a summation of:

- Axial stress
- Major axis bending stress
- Lateral bending stress
- Warping normal stress

The shear stresses in an I-girder are a summation of:

- Vertical shear stress
- Horizontal shear stress
- St. Venant torsional stress
- Warping shear stress

Steel I-girder bridges that are curved, heavily skewed, or have any other irregular framing have cross frames and diaphragms that are primary load carrying members and should be rated. See Section 2.12 for guidance on rating these members. Diaphragms of straight, non-skewed bridges are assumed not to provide any structural capacity and are not required to be rated. Consider diaphragm location as brace point. See Sections 2.4.2.2 and 2.12.3 for discussion on evaluating lateral bracing members as primary load rated members. NSBA (2022) has an in-depth discussion on the buckling behavior of steel girder systems, including lateral torsional buckling, torsional stiffness of I-girders and bracing of girders.

If poor ratings (<1.0 OR) are being observed, better ratings may be achieved for straight composite I-sections in negative flexure and straight non-composite I-sections, by using the flexural resistances from AASHTO LRFD Appendix A6 so long as the requirements set forth in AASHTO LRFD Article A6.1 are satisfied.

*Concrete Encased Steel I-beams*:
Encased I-beams are embedded in concrete throughout the beam length. The presence of concrete adds strength, stiffness and lateral bracing to the beam compression flange. Refer to PUB 238 IE Article 6B.6.1 for consideration of composite action in an encased I-beam for shored and unshored construction. In addition to the requirements of PUB 238, consider composite section between steel section and concrete encasement if there is sufficient shear transfer between the steel section and concrete, verifiable by calculation or presence of shear stirrups in existing plans or visually in the field. See discussion in Section 2.13.3 for evaluating the composite action between encased girder and concrete deck. The rating engineer should evaluate both the concrete encasement and concrete deck to determine the appropriate girder section for analysis.

In the absence of information about the use of shored construction for girders which yield unsatisfactory ratings, use engineering judgement and look for evidence in beam photographs which may indicate use of shored construction. For example, if suspended formwork was used, coil type hangers may remain in the beam. For guidance on consideration of section loss in encased beams, see Section 2.6.1.2. Figure 2.6.2.1-4 shows an example of formwork used for an encased I-beam.

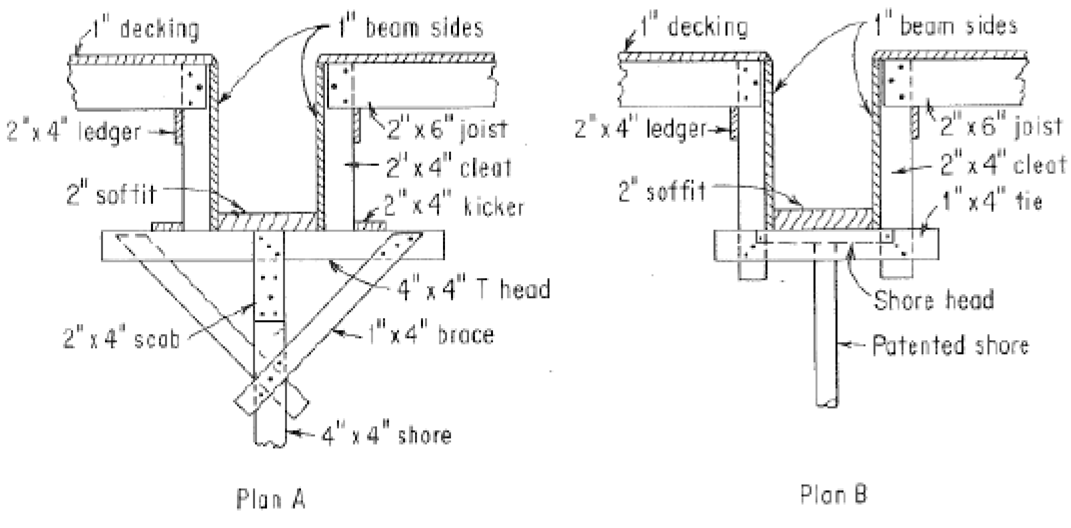



**Figure** 2.6.2.1**-4 – Formwork in Concrete Encased I-beam**

*Compression flange bracing*:
Follow MBE Article 6A.6.9.3 for guidance on compression flange bracing by the concrete deck. Compression flange brace points include locations where the deck is positively connected to the compression flange and at the location of diaphragms or cross-frames.

*Stress history tracking*:
During girder analysis, the rating engineer should be careful with the superposition of various stresses from the analysis model. It would be incorrect to use the maximum factored moment and calculate the stresses based on composite section or non-composite section only. Calculate the stresses in the girder at each stage based on the type of loading and effective section properties during that stage and consider the proper summation of stresses. For evaluating stresses in a steel member retrofitted with steel plates, caution should be observed in the method used to calculate stresses. The construction stage stresses will be locked-in to the existing member and the retrofitted steel members will participate in resisting changes to dead load forces at the time of strengthening and live load. The rating engineer should evaluate whether the strengthening plates were installed for condition repairs or strength repairs. Condition repair refers to the new steel plates installed at the locations with section loss. Steel members installed for condition repairs do not contribute to the member strength and can be neglected for calculating the member capacity.

PennDOT's load rating programs do not provide the capability to load rating considering stress history. Therefore, assume a properly designed repair (with sealed plans and/or calculations), can be considered as restoring the member to its original condition for load rating purposes.

*Moment Redistribution from Negative Moment Region*:
Moment redistribution is an alternate analysis procedure permitted by AASHTO LRFD and is applicable to Straight and Continuous Spans. Typically, the interior pier support moments are higher than the midspan moments and can control ratings This method redistributes negative moment from interior piers to the adjacent positive moment region by assuming that the section has sufficient ductility and inelastic rotation capacity higher than the moments at the interior pier. Consult MBE Article C6A.6.9.1 for more information on moment redistribution in negative movement regions for I-section members fulfilling the requirements of AASHTO LRFD Article B6.2. Use of this section is favorable to achieving higher load ratings in service and strength limit states.

*Composite action in negative moment region*:
For I-sections satisfying the criteria shown in AASHTO LRFD 6.10.4.2, the stresses for Service II load combination can be calculated by assuming that concrete is fully effective in the negative moment region. The well-distributed longitudinal reinforcement and shear connectors ensure that the deck can effectively resist the flexural stresses in the composite section.

*Section Properties for Evaluation of As-Inspected Ratings*:
Use the as-inspected section properties from field data to calculate the as-inspected ratings. Use measurements of section loss from the inspection report and refer to Section 2.6.1.2 to calculate the effective section properties. Typically, section loss is insignificant in modifying the load distribution to the girders in a multi-girder bridge, however high section loss areas should be closely evaluated for girder stresses. I-girders with severe section loss where redistribution of loads occurs is outside the scope of this document.

*Composite Action*:
Refer to Section 2.13.3 for more discussion on composite behavior of girders. Some old bridges which might have shear connectors on original plans or shop drawings should not always be assumed to behave as composite sections. The rating engineer should evaluate whether the girder behaves as a composite section or not. If the bridge is being evaluated for rehabilitation, check whether more shear connectors are required as per latest specifications.

*Partial length flange cover plates*:
When rating a multi-beam steel girder bridge in which a steel girder has a cover plate, reduce the length of cover plate by the development length equal to 1.5 times the width of the cover plate at both ends of the plate. For the analysis of a steel beam with section loss, see Section 2.6.1.2

*Splice plate analysis*:
Refer to Section 2.12.2 for guidance on rating analysis of splice plates.

#### 2.6.2.2 Steel Rolled Beams

For rating of steel rolled beams, follow guidance from Section 2.6.2.1 where applicable. Load rating of typical steel rolled beam bridges can often be performed using PennDOT's STLRFD program (LRFR), BAR7 (LFR) and AASHTOWare (LRFR, LFR). MBE Appendix A1 provides an example load rating of a single span rolled section using LRFR methodology.

For rating historical bridges without plans, use the beam information available from the book "Historical Record-Dimensions and Properties-Iron and Steel Beams 1873 to 1952", published by the American Institute of Steel Construction (AISC). This book lists several steel sections with the approximate year they were manufactured and is a useful guide to determine the section properties of historical sections. Steel section properties of rolled sections often change from time to time. Obtain the section properties corresponding to the bridge construction year and steel data records.

For section loss discussion on rolled shapes, see Section 2.6.1.2.

#### 2.6.2.3 Box and Tub Girders

Load rating of box and tub girders should be performed in accordance with MBE Articles 6A.6.11 for LRFR method. Follow MBE Article 6B.5.2.1 for ASR method. Follow MBE Article 6B.5.3.1 for LFR method.

Box and tub girders should be analyzed in accordance with the MBE and any other governing specification depending on the rating methodology (e.g., AASHTO LRFD, AASHTO Std. Spec.). General analysis guidance for common box and tub girder types is as follows.

- Straight non-skewed single or multi-girder (e.g., tangent tub girder) bridges: 1D analysis, i.e., line girder analysis with distribution factors.
- Curved and/or skewed multi-girder bridges: 2D (i.e., grillage, grid, or PEB) or 3D analysis.

For curved bridges, approximate analysis methods that extend the results of a 1D analysis to account for curvature are available for use in certain situations. The M/R method can be used for curved box and tub girders to account for the torsional moments caused by curvature. While the accuracy of this method can vary, it may be useful for checking or verifying results from a refined analysis. See AASHTO LRFD Article 4.6.2.2.4 and commentary, AASHTO/NSBA (2019), and Coletti et al. (2005) for discussions and limitations of the M/R method. The appropriate use of the M/R method for a specific load rating is the responsibility of the rating engineer.

Additional guidance on analyzing box and tub girder bridges can be found in Coletti et al. (2005), White et al. (2012), and AASHTO/NSBA (2019). See Section 1.7.1.1 for guidance on types of analyses. See Section 2.2 for guidance on curved and skewed bridges.

Depending on the rating methodology used, the capacity of a box girder should be determined in accordance with the MBE and AASHTO LRFD (LRFR), or the AASHTO Std. Spec. (ASR and LFR). See PUB 238 IP Article 3.8 for information on Department approved software that can be used for box and tub girder analysis and ratings. For a given bridge, there may not be software available that performs an all-encompassing load rating (i.e., analysis, capacity, and rating factor calculations) depending on the level of analysis required and rating methodology. In these instances, the use of software for analysis combined with hand calculations for capacities and rating factors (or vice versa) may be required to perform the load rating. For example, a straight non-skewed tangent tub girder bridge could be analyzed using PennDOT's CBA program to determine demands, followed by the hand calculation of the tub girders' capacities, and finally a hand calculation of the rating factors.

The following considerations should be taken into account when rating box and tub girders:

- Behavior. Closed sections have a much larger torsional stiffness compared to typical I-sections. It is because of this property that closed sections are often used in curved and/or heavily skewed bridges. Figure 2.6.2.3-1 and Figure 2.6.2.3-2 show the shear and normal stress contributions, respectively, on closed sections.

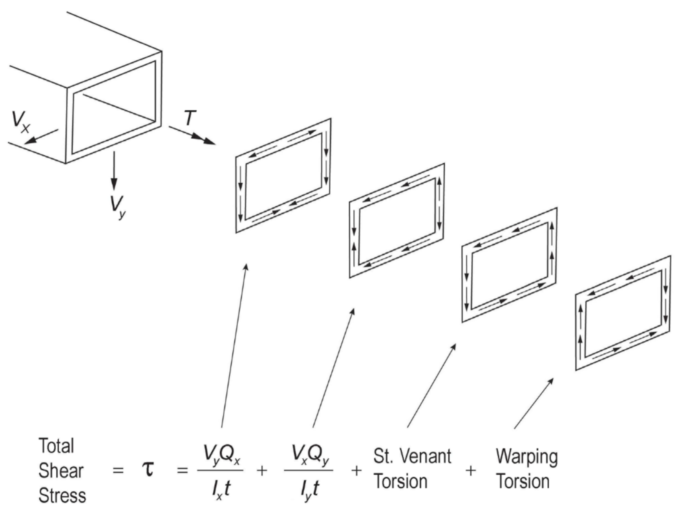



**Figure 2.6.2.3-1 – Closed Section Shear Stresses (Used with permission from *Guidelines for Steel Girder Bridge Analysis, G13.1-2019*, developed by the AASHTO/NSBA Steel Bridge Collaboration)**

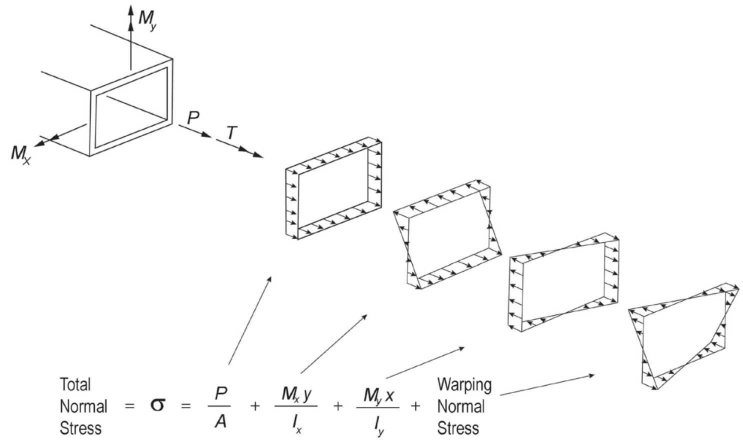



**Figure 2.6.2.3-2 – Closed Section Normal Stresses (Used with permission from *Guidelines for Steel Girder Bridge Analysis, G13.1-2019*, developed by the AASHTO/NSBA Steel Bridge Collaboration)**

- Curvature effects. On curved multi-girder bridges, the girder on the outside of the curve will typically control the load rating. See Section 2.2.2 for additional information.

- Torsion. All closed sections subject to eccentric loading carry torsion. Even straight non-skewed bridges are subject to torsion, although the magnitude may be negligible in these cases. Sources of torsion include asymmetric loading (e.g., noncomposite loading, live load), skew, and curvature. The combined effect of vertical shear and torsion increases the design shear in one web and decreases it in the other web (see Figure 2.6.2.3-1). Thus, whichever method is used for rating (e.g., manual calculations, software, etc.) should account for this behavior.

- Inclined webs. When the webs of closed sections are inclined, as often is the case for tub girders, the resultant web shear increases due to the slope (Figure 2.5.4.1-3). This is often accounted for in design specifications by increasing the shear demand (e.g., AASHTO LRFD Article 6.11.9) or by decreasing the web capacity. The rating engineer should ensure these shear effects are accounted for in the ratings.

- Flange lateral bending. Flanges, particularly top flanges in tub girders, can be subject to significant lateral bending stresses. Sources may include horizontal curvature, sloped webs, temporary support brackets for overhangs, and lateral bracing systems. Not all of these sources necessarily need to be included in a load rating. For example, live load induced lateral bending stresses in top flanges with a composite deck typically do not need to be considered. However, the rating engineer should ensure that flange lateral bending is accounted for in the ratings when appropriate.

- Other stress determinations. Depending on the bridge characteristics (e.g., straight, curved, skewed, single girder, multi-girder) and limit state, other sources of stress may need to be considered in a rating evaluation. Examples include cross-section distortion due to transverse bending and longitudinal warping of a box or tub section. See AASHTO LRFD Articles 6.11.1 and 6.11.1.1 and associated commentaries for detailed discussions.

- Shear Lag. The assumption within conventional beam theory that plane sections remain plane is not always applicable to closed sections. As the width of a box flange increases, normal stresses in the flange near the web become significantly larger than at mid-flange width due to the larger axial rigidity of the flange in comparison to its shear rigidity. This behavior is referred to as shear lag. The effect of shear lag is most pronounced when the moment changes rapidly along the length (i.e., shear forces are large) and as the flange width to span length ratio increases. It is because of shear lag that most design specifications place a limit on the flange width to span ratio (typically 0.2 or less). However, this cannot be controlled during a load rating. Depending on the rating methodology and corresponding specification(s), it may be appropriate to account for shear lag through increased demands or limiting the capacity of a section (i.e., using a reduced effective width for capacity calculations).

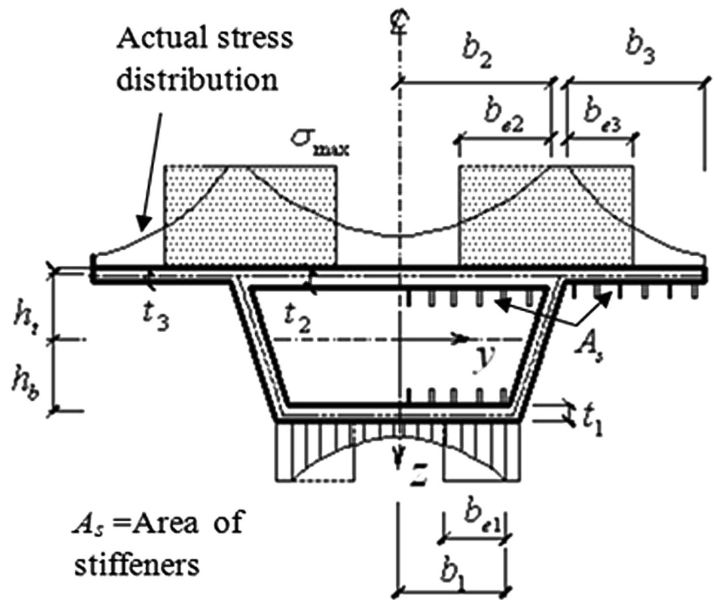



**Figure 2.6.2.3-3 – Comparison of Actual and Effective Flange Distribution in Tub Girder (With permission from ASCE, taken from *Journal of Bridge Engineering, Volume 16, Issue 6*, 2011)**

- Cross frames and diaphragms. On box and tub girder bridges that are curved, heavily skewed, or have any other irregular framing, cross frames and diaphragms are primary load carrying members and should be rated. See Section 2.12 for guidance on rating these members.

### 2.6.3 Trusses

Load rating of trusses should be performed in accordance with MBE Article 6A.6.6 for tension members, 6A.6.7 for non-composite compression members, and 6A.6.8 for members exposed to combined axial compression and flexure using LRFR methodology. MBE Article 6B.5.2.1 should be used for LFR methodology along with PUB 238 Article 6B.5.2.1. Due to the non-redundancy of trusses, the system factors specified in MBE Article 6A.4.2.4 should be used when evaluating truss bridges.

There are multiple main load carrying components that a load rating factor should be determined for a truss bridge. They include, but are not limited to, stringers, floorbeams (see Section 2.6.4), main truss members, gusset plates (see Section 2.6.7.1), and pins/plates (see Section 2.6.7.3). Note that lateral bracing is usually not evaluated as part of a load rating of a truss bridge as these components are often not main load carrying members, but wind load carrying members.

Truss members are often comprised of built-up plates in either an I-shape or box shape configuration, rolled shapes are also utilized. There are many different truss types, but all follow the same general principles for analysis and rating. Lateral bracing and batten plates should not be included when calculating section properties. If lacing bars or batten plates are missing on compression members, refer to PUB 238 IE Article 6.1.2. See Section 2.6.1.2 for explanation of section loss in truss members. When perforations are present in truss members, refer to AASHTO LRFD Article 6.8.5.2. If batten plates, lacing bars, or stay plates are present in compression members, refer to MBE Article 6A.6.7 for applying a length adjustment factor. The presence of perforations is also accounted for in the allowable stresses of the ASR method outlined in the MBE allowable stress tables.

Typically, trusses are rated per truss line due to unsymmetrical roadway cross sections, skewed supports, varying overhangs, sidewalk on one side, etc. Older bridges may also have truss members with different cross-sectional areas if their loads are different from one truss line to the other. See Section 2.4 for structural members of a truss bridge that require a load rating.

1D analysis in most cases is acceptable, however, in some circumstances, such as poor rating results, a refined analysis may be warranted to more accurately distribute the live load; especially when the trusses are skewed or if the roadway has curvature. A refined analysis can also model forces that a 1D truss analysis will not capture. When using refined analysis such as FEA modelling software to determine truss member forces, member bending moment may be induced in addition to axial forces, depending on beam end constraints defined by the user at panel points and the inclusion of member self-weights or other loads acting along the length of the member. Bending moments can be broken down into two categories, primary and secondary bending moments. Primary moments are caused by loads that are applied to truss members along the length of the member, not at panel points. When trusses are erected, typically the members are cambered for length such that dead load moments are eliminated. Cambering can be confirmed by consulting the original plans and shop drawings for the structure. Because long span trusses are dead load dominated, small live load moments can also often be neglected by modeling the structure using pinned joints. If the structure is modeled with fixed joints, the cambering should be included to remove the original dead load moments, and only live load and any changes to the dead load moments considered. Primary moments can also occur when the work lines of the truss members meet at a single point within the joint, but the centroid of the truss member does not meet at that same point creating an eccentric axial force. This is typically the case with horizontally unsymmetric members with symmetric bolt groups. See Figure 2.6.3-1 below for representation and Pub 238 IE Article 6B.5.2.1.1.

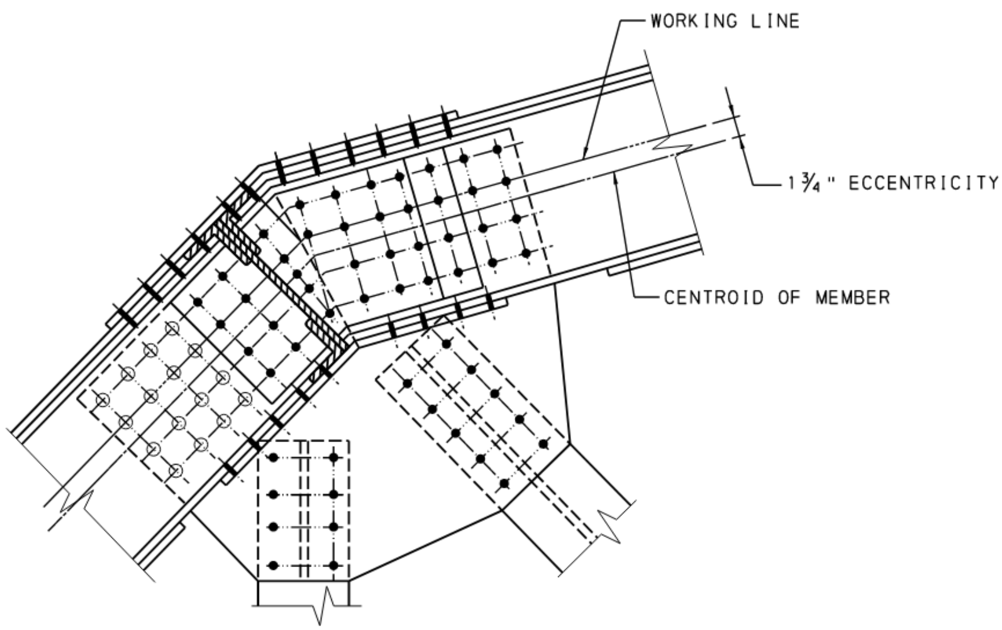



**Figure 2.6.3-1 – Moment Induced by Eccentric Forces**

Secondary bending moments are caused when the joint connections of the member, whether by bolts or rivets, are partially or fully constrained (fixed) and able to transmit moments through the joint. Inherently, trusses are typically designed as pure axial force members with pinned joints, but these primary and secondary moments may slightly increase stresses affecting the load rating factors.

PennDOT's BAR7 program is capable of 1D analysis of truss systems comprised of truss members only, or a combination of floorbeams and or stringers using LFR methodology. Eccentric end connections can also be input in BAR7 to analyze the members for axial plus bending stresses.

### 2.6.4 Floorbeams and Stringers

Steel floorbeams and stringers are typically components in long-span girder bridges, truss bridges, steel arch bridges, and suspended span bridges. Steel floorbeams span transversely, typically between trusses or between deep girders. Stringers span longitudinally between main longitudinal members and are supported by floorbeams. Not all bridges with floorbeams have stringers though, some may have closely spaced transverse members which are sometimes referred to as floor joists.

*Floorbeams:*

On the capacity side, floorbeams are the same as any other steel flexural member, although there may be catwalks or utilities through large holes in the floorbeam webs that should be accounted for in the gross and net section properties. Floorbeams may also be composite with the deck if the deck sits directly on the top flange and shear connectors are present. The brace points of the floorbeam top flange are typically at the stringer connections unless the deck sits directly on the floorbeam and the top flange is firmly embedded in the deck in accordance with DM-4, in which case the top flange can be considered fully braced. If the floorbeam is continuous, the bottom flange brace points are typically at the end connections only, unless bottom lateral bracing is present. Catwalks should be assumed to not provide any bracing. Knee braces at the floorbeam ends can be included when calculating the floorbeam shear capacity, although the original design may have conservatively ignored the knee braces. Note that the optional system factors in MBE Article 6A.4.2.4 for floorbeams rated using the LRFR method are determined based on the floorbeam spacing and the span configuration of the stringers.

Floorbeams can either be simple spans between girders or truss lines, or continuous with moment connections, especially if there are cantilevered floorbeam overhangs with tie-plates. Unless the floorbeam connection at the girder or truss clearly acts as a moment connection, it should be assumed to be a pinned connection for shear only. Some floorbeams have knee braces at their end connections that may be confused as moment connections.

When applying live load to floorbeams with stringers, the live load can be applied directly to the top flange of the floorbeams within the limits of the curbs or applied through the stringers and then to the floorbeams. The distribution factor method for moment and shear may be different for floorbeams without stringers when the floorbeams are closely spaced, see AASHTO LRFD Article 4.6.2.2.2f-1.

Load rating floorbeams can be done using Bar7 for the LFR method or using FBLRFD for the LRFR method. Floorbeams with the same section properties and load effects can be grouped together in a load rating. Significant section loss may affect the section properties requiring less grouping and a larger number of floorbeams to be load rated. The limit states for LRFR load ratings are summarized in DM-4 Table 3.4.1.1P.5.

*Stringers:*

Steel stringers are load rated similar to steel girders. Stringers either sit on top of the floorbeam top flanges or frame into the floorbeams. Stringers are typically rolled steel sections and their section properties can be easily obtained from published tables. Note that older bridges may be rolled sections that are either no longer rolled or have slightly different section properties compared to modern rolled shapes; historic shapes can be found on AISC's Historic Shapes database. Most modern bridges have composite stringers, but composite action should be verified with the presence of shear connectors. Some older bridges have had deck replacements with shear connectors added to the stringers; this should be verified with any available deck replacement plans.

Most floorbeam-stringer bridges have stringers comprised of multiple continuous span configurations, although on some older bridges the stringers may be simply supported at each floorbeam. The distribution factors for stringers are the same as steel girders.

Load rating stringers can be done using Bar7 for the LFR method or using FBLRFD for the LRFR method with the option to combine them with a floorbeam rating. STLRFD is another option for rating steel stringers using the LRFR method.

### 2.6.5 Arches

This section includes guidance on steel arch bridges. The main load carrying members of a steel arch bridge are arch ribs, which are usually box shaped.

Arch ribs, hangers, diaphragms, struts, gusset plates, pins, and connections should be load rated, as applicable. See the appropriate sections in Section 2 of this Manual for guidance on rating these elements. Figure 2.6.5-1 below shows an elevation view of an arch with some main member callouts. Figure 2.6.5-2 shows a cross-section of an arch rib.

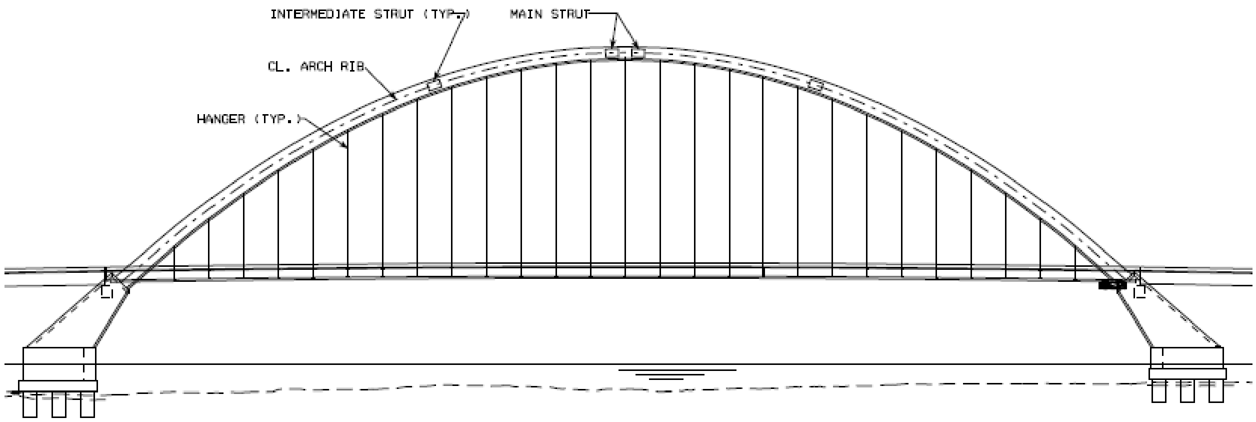



**Figure 2.6.5-1 – Elevation View of an Arch Truss with Main Components Labeled**

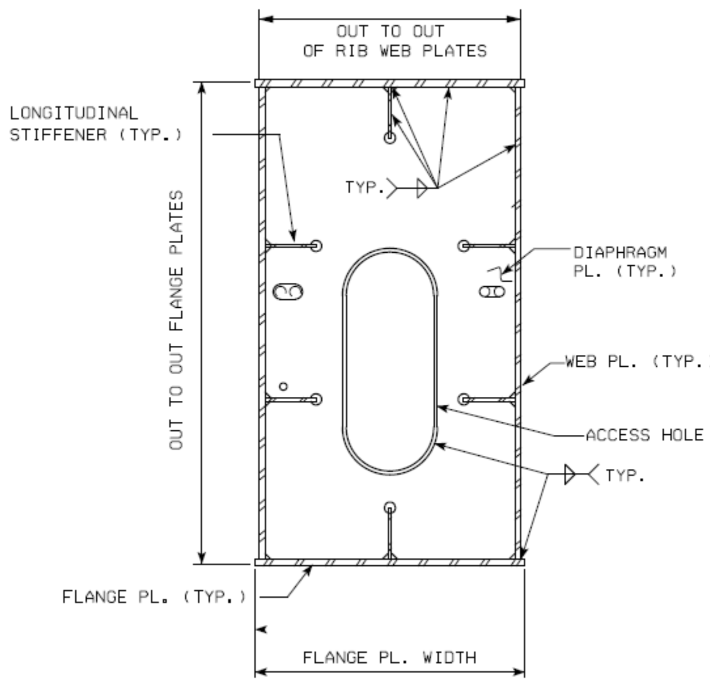



**Figure 2.6.5-2 – Typical Cross Section of an Arch Rib**

Steel spandrel arches are typically fixed, 2-hinged or tied, all of which are statically indeterminate, which complicates the analysis. Arches typically resist compression and bending. Capacities should be determined per MBE Article 6A.6.8 and AASHTO LRFD Articles 6.9.2.2 and 6.14.4 for LRFR method. For ASR method, follow AASHTO Std. Spec. Articles 10.36 and 10.37. For LFR method, follow AASHTO Std. Spec. Articles 10.54.2 and 10.55. MBE Appendix H6A provides a helpful list of equation references for calculating capacities when following LRFR method. Also, if applicable, MBE Article I6A provides an example of rating steel compression members with eccentric connections.

Below is a list of items that should be considered regarding loading and geometry for steel spandrel arches:

- Controlling moments often occur at the 1/4 point or the base (depending on the arch type).
- Arches are susceptible to in-plane and out-of-plane stresses. If the arches are not continuously braced laterally along the arch, an out-of-plane loading may be controlling the buckling behavior. For this case, a refined analysis is suggested to accurately determine the out-of-plane loads. Except for tied arches, all types of arches are susceptible to in-plane bending amplifications due to excessive deflections. Amplification factors can be evaluated by either using simplified methods included in the AASHTO LRFD Articles 4.5.3.2.2b and 4.5.3.2.2c for LRFR method. For ASR method, follow AASHTO Std. Spec. Articles 10.37. For LFR method, follow AASHTO Std. Spec. Article 10.55. Alternatively, in-plane bending amplifications can be estimated with a refined analysis per AASHTO LRFD Article 4.5.3.2.3. A refined analysis will determine a more accurate estimate of the loads, since it includes the actual stiffness, loading and boundary conditions of the arch.
- Although not typically controlling, shear ratings of the arch members should be evaluated in accordance with the appropriate articles in MBE, AASHTO LRFD and AASHTO Std. Spec., as applicable.

### 2.6.6 Metal Culverts

This section provides guidance on load rating flexible metal culverts, which include CMP, metal pipe arch, structural plate pipe, long-span structural plate, and metal arch culverts. Figure 2.6.6-1 shows a myriad of different metal culvert types.

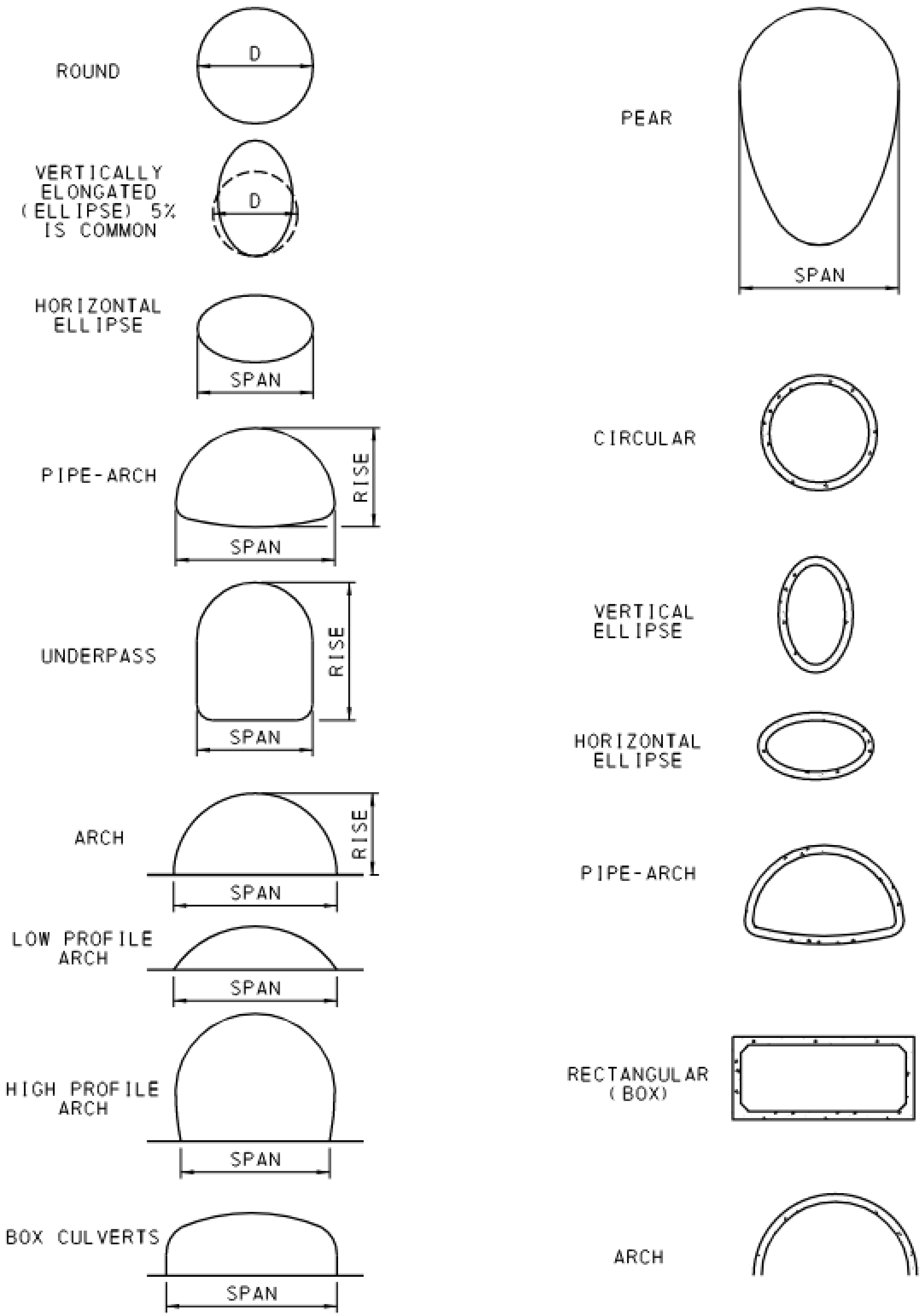



**Figure 2.6.6-1 – Typical Culvert Shapes**

For LRFR method, follow the capacity guidelines for metal culverts provided in AASHTO LRFD and DM-4 Articles 12.5 and 12.7. For ASR method, follow AASHTO Std. Spec. Article 12.2. For LFR method, follow AASHTO Std. Spec. Article 12.3.

Specific metal culvert types should be evaluated accordingly per AASHTO LRFD and DM-4 Articles 12.7 through 12.9 and 12.3 for LRFR method. For ASR and LFR methods, follow AASHTO Std. Spec. Articles 12.4 through 12.8.

Determine applicable loads per AASHTO LRFD and DM-4 Articles 12.6 for LRFR method. For ASR and LFR methods, follow AASHTO Std. Spec. Article 12.1. Regardless of which load rating method is used, metal culverts must be checked for the following failure modes:

- Wall Resistance
- Buckling Resistance
- Seam Resistance for Structures with Longitudinal Seams

AASHTO Culvert and Storm Drain System Inspection Guide (2020) provides additional guidance on evaluating existing metal culverts. McGrath et al. (2002) has more guidance on common practices for evaluating long span metal culverts (spans between 10 ft. and 50 ft.). NCSPA (1995) is another reference for guidance on how to load rate metal culverts. See Section 2.6.1.2 of this Manual for more information on metal culverts with section loss and/or defects.

At a minimum, culvert ratings should be checked at the following locations along the culvert length:

- Cross-section dimensions change
- Culvert type or material changes
- Location of maximum soil cover (maximum earth pressure load)
- Location of minimum soil cover, if applicable for maximum live load effects
- Unique locations such as change in soil type, special designs, and obstructions

While analytical methods exist for rating existing metal arch culverts, engineering judgement is an acceptable method and typically used to rate these structures. There are currently no approved PennDOT programs to rate metal arch culverts.

AASHTO and NCHRP sponsored software CANDE-2022 (or latest version) along with the CANDE Tool Box can also be used to perform a 2D finite element structural analysis and rating of metal culverts using LRFR and ASR methods. The latest version of this software, Tool Box, User's Manual, Solutions and Formulations Manual and tutorials are available for free to download at http://www.candeforculverts.com/.

### 2.6.7 Connections

There are three mechanical connection types on primary members that may need to be included in a load rating. To some degree, each is dependent on the connection type, the redundancy of the member (lack of), and whether the member is in tension.

- Truss Gusset Plate Connections
- Bolted (or Riveted) Splice Connections
  - Non-redundant Longitudinal Flexural Members
  - Tension Members
  - Cantilevered Floorbeam Overhang Tie-Plates
- Pin Connections

Mechanical connections using bolts or rivets are checked for shear in the fastener and bearing on material at the fastener hole. Bolts in slip-critical connections are also checked for slip using service loads. Any tension in the bolts due to an applied moment, prying, and/or direct tension will reduce the slip capacity of the connection and should be included in the analysis. Conservative assumptions should be made for the faying surfaces when determining the slip coefficient if design plans or shop drawings are unavailable. Be aware that the presence of the following will reduce the capacity/resistance of a mechanically fastened connection:

- Oversized or Slotted Holes
- Undeveloped Fill Plates
- Long Bolt Groups (in the direction of applied stress)

It is uncommon to include welded connections in a load rating analysis, as most of these are shop welded and meet or exceed the capacity of the member being spliced. It may also be difficult to determine the location of shop welds. A welded connection with measurable section loss may need its capacity evaluated and compared to the capacity of the primary members in the connection. Web-to-flange welds of built-up plate girders are typically not included in a load rating analysis. Primary member welded connections with cracks are outside the scope of this Manual and likely require immediate repair or retrofit.

Load rating evaluations of connections is outlined in MBE Article 6A.6.12 for the LRFR method. The LFR method is outlined in MBE Article L6B.2.6.1 under the gusset plate sections, although the provided information is also applicable to other critical connections. Also follow the specifications for mechanically fastened and welded connections provided in DM-4 Article 6.13. Further discussion of gusset plates, bolted splices, and pin connections is provided in the following sections.

#### 2.6.7.1 Truss Gusset and Splice Connections

As outlined in Pub 238 IP Article 2.4.10.1, truss gusset plate connections are to be included as separate members in the bridge load rating analysis since they connect non-redundant truss members. Gusset connections are at least two vertical plates at truss joints connecting multiple truss members framing into each other. In most situations, the forces are assumed to be evenly shared by the two gusset plates. Each member's connection to the gusset plates is to be included in the analysis. Note that some truss verticals may have zero live load and their connection to the gusset plate will not need evaluated for a live load rating. It is good practice to at least check the capacity-to-demand (C/D) ratio for that member. The C/D ratio, when evaluated, should yield a number greater than 1.0 which indicates the member can carry the dead load forces.

When performing a load rating evaluation of gusset plates, include changes in the plate condition (as determined by recent field inspection), and changes in dead load and live load.

PennDOT's BAR7 program can evaluate gusset plates following LFR methodology. Refer to MBE Appendix L6B.2.6 when load rating gusset plates with LFR. Refer to MBE Article 6A.6.12.6 for the LRFR method. A gusset plate load rating example is also provided in MBE Appendix A for both LFR and LRFR methods, although be cognizant of PennDOT's requirements in Pub 238 and DM-4 compared to the MBE when following those examples. A load rating analysis of gusset plate connections include the following checks to determine the controlling capacity or resistance at each member's connection to the gusset plate.

- Fastener Shear Resistance/Capacity
- Bolt Slip Resistance/Capacity
- Bearing Resistance/Capacity of Fastener Holes
- Gusset plate checks, including:
    - Gusset Plate Shear Resistance/Capacity including block shear as well as net and gross yielding
    - Gusset Plate Compressive/Buckling Resistance/Capacity at the Whitmore section
    - Gusset Plate Tensile Resistance/Capacity
    - Chord splice if present

When calculating the Whitmore section of the gusset plate at each connected member, verify the typical 30-degree angle of distribution is not outside the limits of the gusset plate. If that situation occurs, reduce the 30-degree angle on one or both sides of the member to stop at the edge of the gusset plate.

The load rating engineer should be aware of any rivets/bolts that may be subject to "double shear". Double shear can be utilized when two cross-sections are effective in resisting the load. The bolt/rivet in double shear will effectively have twice the shear strength. See Figure 2.6.7.1-1 2.7.7.1-1 below for further clarification.

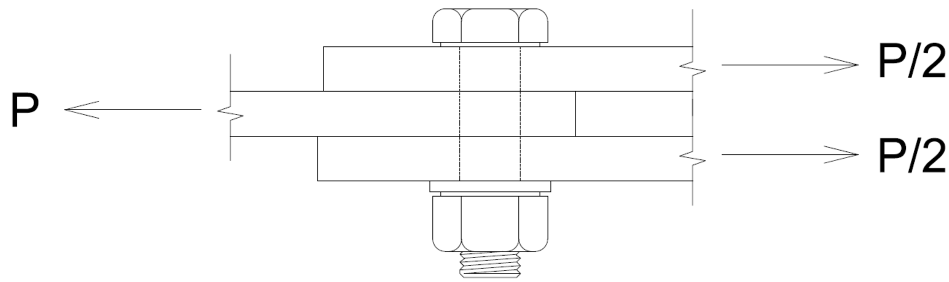



**Figure 2.6.7.1-1 – Bolt/Rivet in Double Shear**

If the gusset plate is also being utilized as a tension chord splice, evaluate the connection in accordance with MBE Article 6A.6.12.6.9. The resistance of individual plates may be added together when determining the factored resistances per MBE Article 6A.6.12.6.5. When a chord is continuous through the gusset plate, i.e., no splice is present, that chord member does not need its connection to the gusset plate evaluated because load is not transferred through the gusset plate from the chord member.

2.6.7.2 Bolted Splices

All field splices that are located on nonredundant members including main truss chord members and fracture critical members must be load rated. Any splice (regardless of redundancy) must be load rated if deterioration is noted, if there is an indication that a slip critical connection has slipped and is transferring load as a bearing type connection, or if requested by the Department. For further LRFR splice load rating guidance, see MBE Article 6A.6.12 and AASHTO LRFD/DM-4 Article 6.13. For further ASR and LFR splice load rating guidance, see MBE Articles 6B.5.2.1 and 6B.5.3.1, respectively, and AASHTO Std. Spec. Article 10.18.

The rating engineer should be aware of a fill plate reduction factor specified in MBE Article 6A.6.12.5.1 for LRFR method. For LFR method, follow MBE Article 6B.5.3.1.

The splices of nonredundant tension members should be investigated for gross section yield and net section fracture in accordance with MBE Article 6A.6.6 and Section 2.6.3 of this Manual.

For load rating guidance for gusset plates that splice chord sections together, see MBE Article 6A.6.12.6.9 for LRFR method. For LFR method, follow MBE Appendix L6.B.2.6.6. For ASR method, follow MBE Article 6B.5.2.1.

2.6.7.3 Pins

Pins should be evaluated in accordance with MBE Article 6A.6.12.4 and evaluated for combined flexure and shear as specified in AASHTO LRFD Article 6.7.6.2.1 and for bearing as specified in AASHTO LRFD Article 6.7.6.2.2. LFR methodology does not provide straightforward guidance on how to load rate pins. Refer to Guide Specifications for Strength Design of Truss Bridges (1985) Article 1.10 for guidance as well as Kulicki et al. (1983). For ASR methodology, utilize tables in MBE Article 6B.5.2.1 for allowable stresses.

It is important to note that field measurements of pin size (including a reduction for any corrosion and/or wear found) should be recorded and the alignment of plates and members bearing on the pin should be field verified as the placement of the bearing plates has a significant impact on the load capacity. See MBE Article C6A.6.12.4 for further clarification.

## 2.7 WOOD STRUCTURES

Wood members carrying live load should be load rated using the LRFR (MBE Article 6A.7) or ASR (MBE Article 6B.5.2.7) method. The LFR method is not applicable for wood bridges (MBE Article C6B.5.3). Similar to steel and concrete, at a minimum, wood primary superstructure members are to be load rated. Per Section 2.4, wood decks or wood secondary members may sometimes require a load rating. Note that bridges with wood decks on steel stringers are classified as steel structures; however, load rating of the deck should follow the specifications for wood structures.

The calculation of static forces and stresses in wood structures is the same as determining static forces in a typical steel or concrete structure. The material properties of wood can vary significantly depending on the species of wood, quality/grade, moisture content, orientation of the grain, and other factors.

General considerations when load rating wood bridges include the following:

- Actual member net dimensions should be used rather than nominal dimensions.
    - For LRFR method, follow AASHTO LRFD Article 8.4.1.1.2 for sawn lumber and LRFD Article 8.4.1.2.2 for glue-laminated timbers.
    - For ASR method, follow AASHTO Std. Spec. Article 13.2.1.2 for sawn lumber and AASHTO Std. Spec. Article 13.2.2.2 for glue-laminated timbers.
    - If the bridge has other timber grades, refer to the National Design Specification for Wood Construction (NDS).
- The load rating engineer should be cognizant of the wood deck type, such as plank, stressed laminated, spike laminated, or glue laminated, when determining the distribution of live load in interior longitudinal beams. For LRFR method, follow AASHTO LRFD Table 4.6.2.2.2a-1 and DM-4 Article 4.6.2.2.2a. For ASR method, follow AASHTO Std. Spec. Table 3.23.1. The Lever Rule is typically used for determining the distribution of live load in exterior longitudinal wood beams.
- If a field inspection indicates loss of bar force in a post-tensioned stress laminated wood slab bridge, differential deflection between the longitudinal timber planks may be occurring, which causes a reduction in transverse distribution of live load (i.e., greater concentration of live load) due to loss of “slab” action. The capacity should be reduced to account for this loss of “slab” action by assuming less timber planks in the cross-section are contributing to load resistance. For more information, see PUB 238 IE Article 4.3.5.6.4.
- The size, type, and orientation of imperfections in wood members, such as cracking, splitting, or checking, will influence the structural capacity of wood members. See Section 2.7.1 for more details.

- In addition to checking vertical shear in wood beams, horizontal shear caused by bending should be checked when the horizontal shear stress is parallel to the grain as it may control the bending strength.

- For the load rating of critical connections using LRFR method, evaluate shear at the strength limit state (MBE Article 6A.7.6).

- Dynamic load allowance should not be applied to wood members for the LRFR method (MBE Articles 6A.4.3.3, 6A.4.4.3, and 6A.7.5) nor impact applied for the ASR method (AASHTO Std. Spec. Article 13.1.3).

- Timber decks with excessive deflections should be load rated and often control the overall structure rating (MBE Article 6.1.5.1).

- For the load rating of timber decks using the ASR method, alternate HS20 axle loads may be used per MBE Figure 6B.6.2-1.

- For the load rating of timber piles following LRFR method, see AASHTO LRFD Articles 8.4.1.4, 8.8, and 8.10. Following ASR method, see AASHTO Std. Spec. Articles 13.6 and 13.7.

An example of an interior timber stringer rating is provided in MBE Appendix A4.

### 2.7.1 Material Properties, Defects and Section Loss

Wood material is orthotropic, meaning the material properties vary in each direction depending on the grain. Whether hardwood or softwood, the species of wood also significantly affects the material properties. The presence of defects such as decay, rotting, splitting, cracking, and crushing reduce the material strength. MBE Article 4.3.5.6.4, as supplemented by PUB 238 IE Article 4.3.5.6.4 provide additional details on section loss and signs of distress for timber structures. As-inspected load ratings must account for these defects by reducing section properties accordingly. The following material-related items should be considered:

- Material properties are to be determined in accordance with PUB 238 IP Article 3.7.2.3 and MBE Articles 6A.7.2 and C6A.7.2 for the LRFR method. For the ASR method, follow AASHTO Std. Spec. Article 13. Also, see Section 1.10 of this Manual, as applicable. Check the existing plans for wood species, grade, and other relevant material properties, if available and applicable.

- Condition factors per MBE Article 6A.4.2.3 (LRFR) and safety factors per MBE Article 6B.5.2.7 (ASR) may be adjusted when a substandard grade or degradation is encountered (MBE Article 6.1.1).

- As-Inspected load ratings must account for decay/section loss/defects in timber members by reducing the cross-sectional area at minimum (MBE Article 6.1.2). Broken timber members or those that are cracked, decayed, or rotted over a significant length may need to be considered ineffective and removed from the evaluation. If this occurs, loads must be distributed assuming the wood member is not there. Note, the deck may need rated if a beam is considered ineffective and removed from the analysis. The type and extent of decay can be difficult to detect during inspection and often extends beyond visible areas; therefore, all decay should be assumed to be significant (Ritter 1990).

- For timber pile foundation load rating evaluations:
    - See AASHTO LRFD Article 8.4.1.4 for material specifications of timber piles when following the LRFR method. See AASHTO Std. Spec. Article 13.2.4 when following the ASR method.
    - To account for section loss, reduce the pile diameter.
    - Significantly decayed timber piles should be omitted from the pile group in the foundation capacity calculations.
    - If the top of pile is visible, check to see if as-designed end conditions at the connection to the pile cap are still valid given section loss in the timber pile.
    - If scour is present, exposing the piles, the lateral stability of the piles may be reduced and should be included in the pile moment analysis (PUB 238 IE Article 4.3.5.7.3).

## 2.8 MASONRY STRUCTURES

Masonry bridges, reinforced or unreinforced, should be rated using the ASR method (MBE Article 6B.5.2.6) when possible. The LRFR and LFR methods cannot be used for masonry bridges (MBE Article C6B.5.3). When sufficient information is unavailable and unmeasurable (e.g., dimensions, details, etc.), it may be appropriate to rate old masonry structures using the engineering judgement method (PUB 238 IP Article 3.6.1.1 and Appendix IP 03-B). See Section 1.4 for additional guidance on rating structural members following the engineering judgement method.

### 2.8.1 Material Considerations

Material properties should be determined per Section 1.10. The following material related items should be considered:

- Mortar used to bind masonry units should be classified in accordance with ASTM C270 (MBE Articles 6A.9.1.1 and 6B.5.2.6).
- The inventory allowable compressive stress for various masonry assemblies is provided in MBE Table 6B.5.2.6-1.
- MBE Article 6B.5.2.3 may be considered for reinforced masonry members if reinforcement properties are unknown.
- The condition of the mortar affects the masonry strength. Consider mortar deterioration caused by water, weathering, and high frequency dynamic loads when evaluating material strength (Chajes 2002).

### 2.8.2 Rating Considerations

General considerations when rating masonry bridges include the following:

- Masonry should be evaluated at the inventory level only when completing an analytical rating.
- The condition of both the masonry and mortar should be considered when determining capacity.

Dead and live loads should be included in the analysis as described in Section 1.6. For closed spandrel masonry arches with fill material, vertical and horizontal earth pressure loads (EV and EH) should also be included in the analysis. Other loads may need to be considered for arches as described in MBE Article 6A.5.10 and Sections 1.6.3.2 and 1.6.3.3 of this Manual.

Historically, the most common application of unreinforced masonry for bridges is the filled spandrel arch. This structure type carries live load in multiple directions (Chajes 2002). Compressive thrusting forces are carried through the arch barrel in the longitudinal (along the span) direction to the abutments. In addition, a portion of the live load is applied through the fill material as a lateral surcharge pressure against the spandrel walls and wing walls. These lateral forces can lead to longitudinal cracks in the arch barrel or outward displacement of the spandrel walls.

Approximate and/or classical arch analysis methods may be used for load rating; however, these methods tend to overestimate demands (or underestimate capacities). The fill material provides passive restraint to the arch, which limits live load deflection and increases the stiffness of the structure. Therefore, in some cases, a refined analysis that considers soil-structure interaction including lateral earth pressure effects may be needed to determine accurate ratings. A refined 3D analysis is also needed to accurately determine transverse effects.

Unreinforced masonry arches should be evaluated per MBE Article 6A.9.1.2. The modes of failure of the arch that are typically investigated when load rating these structures are:

1. Overturning of adjacent masonry units
2. Sliding or shear failure
3. Compressive failure of the masonry

The following are miscellaneous topics to consider while load rating these structures (Chajes 2002):

- A lack of longitudinal cracks in the arch is typically an indication that the rating analysis only needs to be considered in the span direction.
- Thick arch barrels are less susceptible to adverse transverse effects.
- Bridges with thin arch barrels and deep fills are more susceptible to adverse transverse effects.
- Large magnitude dead loads are more likely to cause failure (and influence ratings) than live loads.

See PUB 238 IP Article 2.5.3 and PUB 238 IE Article 4.3.5.6.15 and MBE Article 4.3.5.6.15 for guidance on notable section loss and defects in stone masonry structures. Common signs of potential distress that may need to be considered in load rating masonry arches include:

- Wide cracks and deep mortar losses. Cracking is common in mortar and not typically concerning. However, previous inspection reports should be consulted to see if the cracking has changed. If there is significant change in the crack, it must be accounted for in the load rating evaluation.

- Occasional missing or slipped masonry stones. These defects should be noted and monitored but are not typically concerning. Slipping of masonry unit is common in un-mortared masonry arches.

- Foundation movement. This can be caused by scour or soil settlement and can lead to failure.

- Spandrel wall bulging, tilting, or sliding. These can be precursors to spandrel wall collapse. When the roadway fill is poorly drained, these walls are susceptible to failure. Failures typically start at the third or quarter points and extend in both directions (Chajes 2002).

- Masonry displacement or deformation of arch. This is indicative of structural failure. The District Bridge Engineer should be notified immediately. The structure should be evaluated to determine the safe load carrying capacity.

PennDOT Stone Arch Bridge Maintenance Manual (2007) provides additional commentary on the types of structural defects and deterioration in stone masonry arch bridges.

If the structural behavior under normal traffic significantly differs from what was predicted through the analysis, experimental load testing discussed in Section 1.7.1.4 or a more refined analysis may be required. Additional guidance on load rating masonry arch bridges is provided in Chajes (2002).

## 2.9 SUBSTRUCTURES

Unless otherwise required by the DBE or project rating criteria, substructures typically need not be load rated. See Section 2.4.3 for guidance on when to rate substructures.

For LRFR ratings, dynamic load allowance (i.e., impact factor) is to be applied to substructures in accordance with MBE Article 6A.2.3.3, DM-4 Articles 3.6.2.1.1P and 3.6.2.1.2P, and AASHTO LRFD Article 3.6.2.1. For ASR and LFR ratings, the impact factor is to be applied to substructures in accordance with MBE Article 6B.6.4 and AASHTO Std. Spec. Article 3.8.

Unless otherwise directed, braking and centrifugal forces should be included in the live load analysis of substructures (MBE Article 6.1.5.2). Longitudinal forces including braking forces need not be included in the live load analysis when rating reinforced concrete pier caps (PUB 238 IE Article 6.1.5.2.1I).

### 2.9.1 Load Sharing

There are several scenarios in which substructures carry loads from multiple spans or separate superstructures. When load rated, the demands on the substructure must reflect this load sharing.

One of the most common forms of load sharing is when an individual substructure (e.g., pier, bent) supports multiple simple or continuous superstructure spans (Figure 2.9.1-1). When determining demands, positioning of live load should be investigated to produce maximum force effects in the substructure. Live load positioning will vary depending on if the spans are simple or continuous, the bearing conditions (e.g., fixed, expansion, integral), and the force effect (e.g., bending in a pier cap, axial force in a column). Positioning should be determined in accordance with the MBE, as outlined below.

- LRFR: For Design Live Load rating, reactions at interior supports should be determined using the provisions of MBE Article 6A.4.3.2.1 and Appendix C6A. Legal Load rating should follow the provisions of MBE Article 6A.4.4.2.1a and Appendix D6A related to interior support reactions and increased span lengths. For Permit Load rating, follow the provisions of MBE Article 6A.4.5.4.1 related to span lengths.

- ASR and LFR: For Design Live Load rating, it is recommended that the provisions of MBE Figure 6B.6.2-2 for negative moment in continuous spans be used to also determine reactions at interior supports. Legal Load rating should follow the provisions of MBE Article 6B.7.2 related to span lengths. For Permit Load rating, follow the provisions of MBE Articles 6B.8.2 through 6B.8.4 related to mixing with other traffic, lane positioning, and impact.

In addition, load sharing and load distribution amongst components within an individual substructure should be considered. For example, columns within multi-column piers and bents will carry a portion of the total load on the substructure. In these cases, loads on individual columns should be maximized in order to capture the critical ratings.

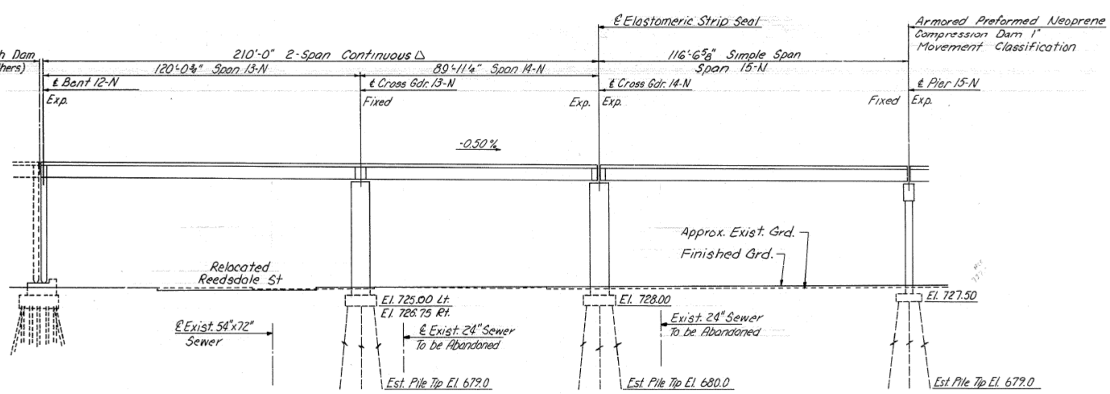



**Figure 2.9.1-1 – Example of Piers Supporting Multiple Spans**

A second less common type of load sharing that is encountered is when a substructure such as a pier or bent supports multiple superstructures, either on different levels (Figure 2.9.1-2) or on one cap/cross girder (Figure 2.9.1-3). In such instances, live load positioning may vary or be more critical for individual members of the substructure. For example, the cross girders in Figure 2.9.1-2 only support live load from one roadway whereas the lower portions of the columns support live load from both roadways. The substructure shown in Figure 2.9.1-3 differs in that the top cross girder supports two roadways and the column bases support 3 roadways. Such scenarios introduce live load complications (e.g., appropriate multiple presence factors to use) that may require project-specific rating criteria to be agreed upon. Otherwise, conservative assumptions should be made.

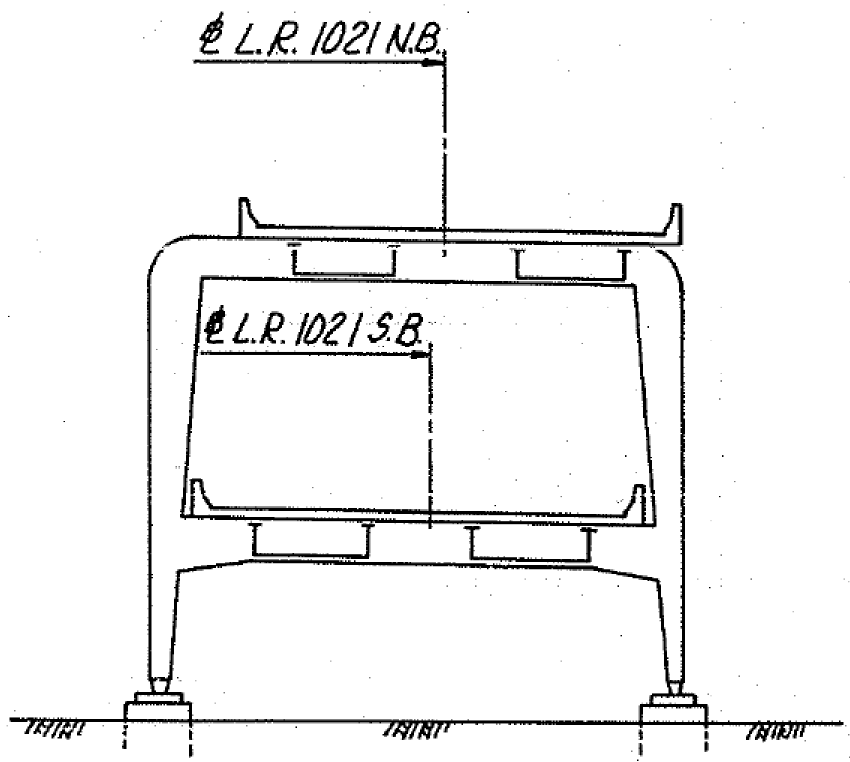



**Figure 2.9.1-2 – Example of a Bent Supporting Multiple Superstructures**

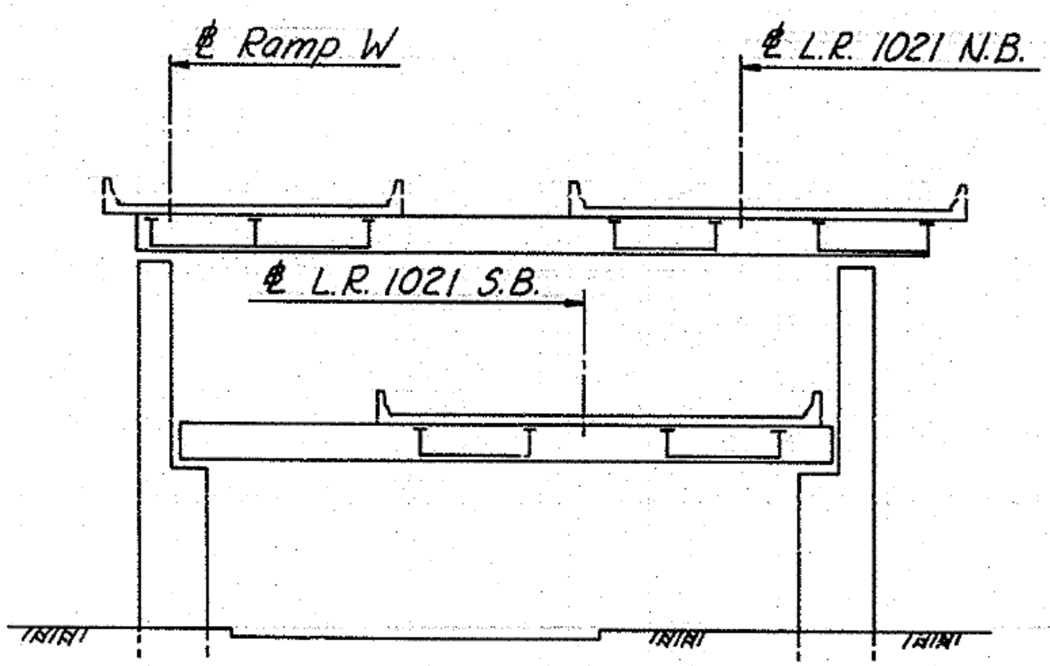



**Figure 2.9.1-3 – Example of Cross Girders and Piers Supporting Multiple Superstructures**

Another example of load sharing occurs when abutments support slab loads from both the bridge deck and approach slab (Figure 2.9.1-4). While this is a common detail in bridge design and construction, it rarely comes up in load rating since it is uncommon to rate abutments. In the event an abutment is rated, load contributions (both dead and live) from the deck and approach slab should be considered in the rating evaluation.

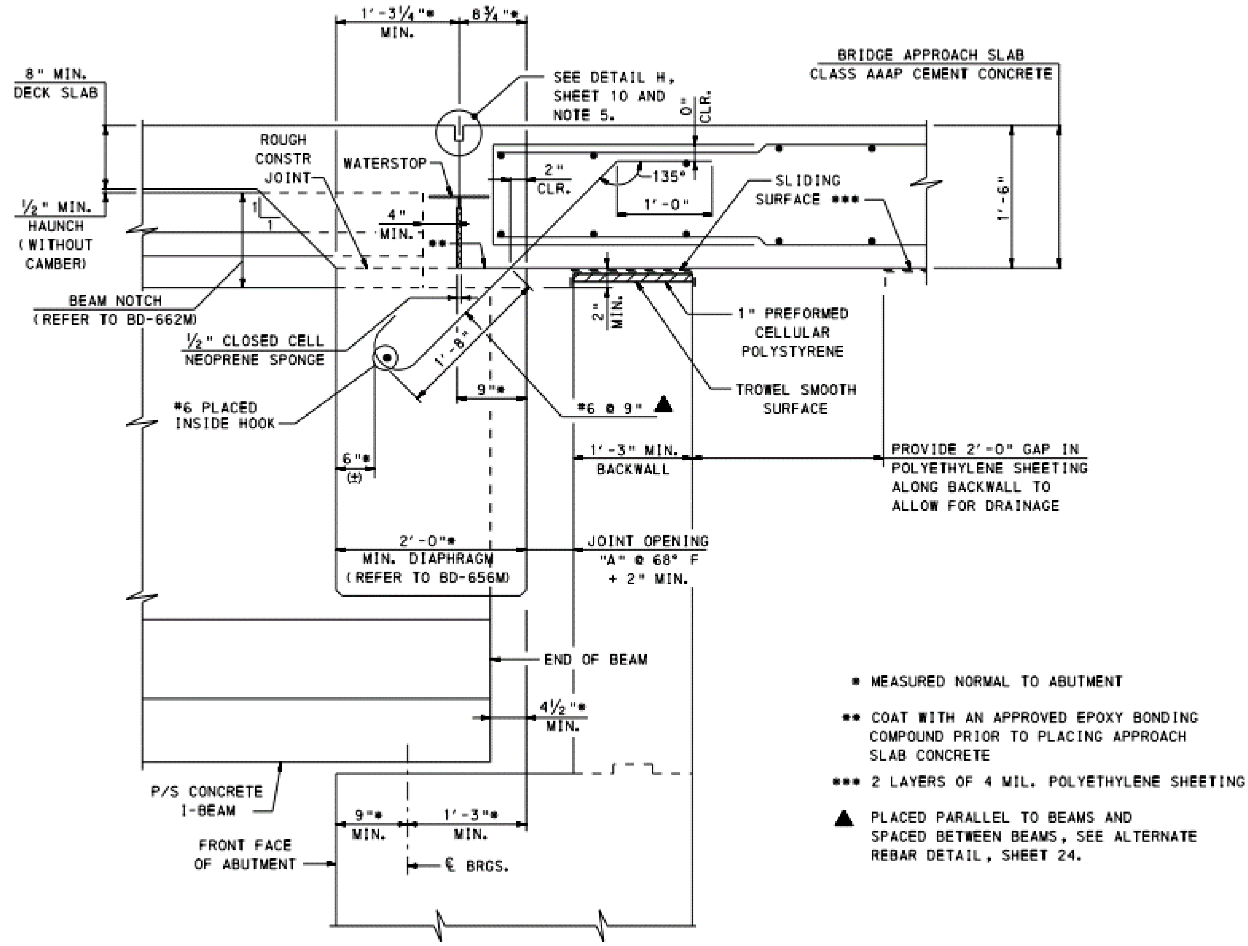



**Figure 2.9.1-4 – Example of an Abutment Supporting a Deck Slab and Approach Slab (Taken from PennDOT BD-628M)**

### 2.9.2 Piers

Concrete piers should be rated in accordance with PUB 238 IE Article 6.1.5.2. VBent may be used to determine load effects and capacity of various pier components.

#### 2.9.2.1 Pier Caps

Concrete pier caps should be analyzed and rated in accordance with PUB 238 IE Article 6.1.5.2.1I.

Steel or concrete cross girders should be rated in accordance with PUB 238 IP Article 3.4.5.

Tables of live load reactions on transverse beam members (e.g., pier caps) are provided in MBE Appendices D6B and E6B. These tables may be convenient for determining live load reactions on pier caps granted all criteria for using the tables are met. Reactions are provided for AASHTO Std. Spec. vehicles only; therefore, the tables can only be used for ASR and LFR ratings.

2.9.2.2 Bents

Concrete compression members that are part of bents (e.g., columns) should be rated in accordance with MBE Article 6A.5.7 and MBE Appendix G6A for LRFR ratings. For ASR and LFR ratings, concrete compression members should be rated in accordance with MBE Article 6B.5.2.4. Provisions for bending and columns are provided in MBE Articles 6B.5.2.4.1 and 6B.5.2.4.2, respectively.

Steel bent compression members should be rated in accordance with MBE Articles 6A.6.7 and 6A.6.8, and MBE Appendix H6A for LRFR ratings. For ASR and LFR ratings, steel compression members should be rated in accordance with MBE Article 6B.5.2.1. Provisions specific to combined stresses and batten plate compression members are provided in MBE Articles 6B.5.2.1.1 and 6B.5.2.1.2, respectively. Steel column capacity formulas for ASR and LFR ratings are provided in MBE Appendices K6B and L6B, respectively.

Wood bent members should be rated in accordance with MBE Article 6A.7 for LRFR ratings. Follow MBE Article 6B.5.2.7 for ASR and LFR ratings.

### 2.9.3 Abutments

Abutments are typically not load rated (DM-4 Article 3.4.1.1P). Live load carrying abutment caps and backwalls are susceptible to deterioration and if severe, a load rating should be considered. When load rated, the capacity of these members should be reduced to account for the deterioration.

## 2.10 FOUNDATIONS

As mentioned in Section 2.4.4, foundations do not typically need to be load rated. When performed, ratings should be done in accordance with the MBE. Any loss of soil (e.g., scour), settlement, or pile deterioration should be accounted for in the ratings.

## 2.11 NON-TYPICAL BRIDGES

In the context of this rating manual, non-typical bridges refer to those that have been altered from their original as-built condition or those that carry elevated significance including widened bridges, historical bridges, and bridges that have been strengthened, repaired, or rehabilitated. These bridge types are discussed in the following sections.

### 2.11.1 Widened Bridges

Bridges that are widened are required to be re-rated to reflect any changes in load carrying capacity (PUB 238 IP Article 2.2.2).

- Widened portions should be rated in accordance with PUB 238 IP Article 3.6.2 and specifically Table IP 3.6.2-1. Refer to DM-4 PP Article 5.5.5 for additional requirements for rehabilitated bridges.

- The analysis method used for load rating should consider the connection (or lack thereof) between the original and widened portions of the bridge. If there is no continuity between the original and widened portions, then the widened portion should be analyzed as a separate structure. An example of this scenario is when a longitudinal deck joint is used between the original deck and widened deck portion, and the original and widened superstructure members are not connected via cross frames. If there is continuity between the original and widened portions, then the entire bridge should be analyzed as one structure.

- Consider load distribution during analysis and capacity evaluations. In order to assume load distribution between the existing and widened portions, the widened deck should be adequately connected to the existing deck via lap splicing, dowel, or similar method. In beam-slab bridges, to use live load distribution factors in an approximate analysis, the widened superstructure beams should have a similar stiffness as the existing beams (AASHTO LRFD Article 4.6.2.2.1).

- Exterior beams on older bridges may not have been designed to have the same capacity as the interior beams. Therefore, the previous exterior beam (now an interior beam) should be rated as an interior beam. This can result in different rating factors for the interior beams on a widened bridge.

- Prior to widening, existing portions of substructures should be load rated if demands on these members will increase as a result of widening. If widened portions of the superstructure are supported by new substructure members, then load rating of the existing substructure members may not be necessary if demands do not increase.

### 2.11.2 Strengthened, Repaired, and Rehabilitated Bridges

Bridges that undergo strengthening, repairs, or rehabilitation are required to be re-rated to reflect any changes in load carrying capacity (PUB 238 IP Article 2.2.2). In addition, when bridges that have been strengthened, repaired, or rehabilitated in the past are required to be load rated as part of an inspection, the rating engineer should account for historical changes to member load carrying capacity as part of the load rating.

Review historical documents such as rehabilitation plan sets to determine repair history that could affect the load rating. If such documentation is not available, compare the current conditions and recent inspection reports to original plan sets (e.g., design, as-built, shop drawings) to determine if and what changes have occurred. For bridges without plans, engineering judgement may be needed to recognize past strengthening, repairs, or rehabilitation changes. Common strengthening techniques include the following (Klaiber et al., 1987; Dorton and Reel, 1997; Chajes et al., 2019):

- *Reducing dead load*. This is often achieved by replacing an existing deck with a lighter one (e.g., lightweight concrete, grid deck, FRP deck)

- *Developing composite action*. Most commonly performed for steel beams supporting a concrete deck when the deck slab is due for replacement.

- *Increasing the transverse stiffness*. This is done to improve the live load distribution to the main longitudinal members and can be achieved using a number of methods such as adding cross frames, increasing the deck thickness, and transverse post-tensioning.

- *Increasing member strength*. Examples include adding plates to steel members, adding reinforcement to reinforced concrete members, and jacketing or encasement using concrete, steel, or FRP materials.

- *Adding or replacing members*. Adding members can reduce the demands on existing members. Replacing members due to damage or deterioration can restore the original capacity, and using modern higher strength materials can provide even greater capacity.

- *Strengthening connections*. Examples include replacing existing rivets with high strength bolts, adding material at the connection (e.g., plates, reinforcement), and replacing pins.

- *Post-tensioning members*. This method is typically used to relieve tension stresses and consequently increase capacity. Other applications include increasing transverse stiffness as mentioned above.

- *Developing continuity*. Adding supports to increase the number of spans and decrease member demands is one way to improve live load carrying capacity. A more common method is to connect adjacent simply supported spans together to create a single continuous support, which decreases the maximum positive moment demand in the beam.

Special considerations when load rating a bridge for an existing or proposed repair or rehabilitation include the following:

- *Construction sequence and locked-in stresses*. Depending on the timing of repair operations, there can be significant locked-in stresses in the existing member under evaluation, typically from dead load. Any strengthening material only carries a portion of the load applied after the repair operations are complete (e.g., live load). The construction sequence used to apply loads both before and after a repair has an effect on the final stress state. Construction sequencing should be considered for all bridge types, but can be particularly important for bridges or members with geometrically nonlinear responses where deflections have a significant effect on demands. Locked-in stresses can be reduced or eliminated by relieving load before performing strengthening repairs. Ways to accomplish this include jacking, temporary supports, and performing repairs while dead load is minimized, such as during a deck replacement. When rating an existing bridge, locked-in stress relief should only be assumed if explicitly documented in existing records. See MBE Article 6A.6.9.2 and DM-4/LRFD Article 6.10.1.1.1 for related discussions specific to steel I-sections in flexure.

- *Original versus repair materials*. Differences in mechanical properties can occur due to the employment of different material types (e.g., steel versus concrete), different material grades or specifications (e.g., historical lower strength steels versus modern high strength steels), or other similar reasons. These differences should be accounted for in the analysis and capacity evaluations, if appropriate.

- *Existing damage*. When a bridge is damaged (e.g., damage caused by a vehicular impact, deterioration, etc.), it often necessitates a load rating evaluation. A load rating may be required for the damaged state to determine if repairs are necessary and/or for the repaired state to determine the change in live load capacity post-repair. Special considerations for damaged members include changes in section properties and changes in mechanical properties, particularly if the repair method involves heat straightening of damaged steel.

- *Changes in demands or structural behavior*. Changes in demands on a bridge or member due to repair or rehabilitation operations (e.g., widening) must be accounted for in the load rating. See Section 2.11.1 for guidance on widened bridges. Similarly, changes in structural behavior such as making a simply supported beam continuous must be considered in the load rating analysis. In addition, these changes can have subsequent effects on supporting substructure load carrying capacity. See Section 2.9 for guidance on rating substructures.

Load rating strengthened, repaired, or rehabilitated bridges should follow PUB 238, the MBE, DM-4, AASHTO LRFD, the AASHTO Std. Spec., and any other governing specifications. Follow PUB 238 Table IP 3.6.2-1 to determine the appropriate rating methodology and refer to DM-4 PP Article 5.5.5 for load carrying capacity requirements for rehabilitated bridges. Load rating provisions specific to members strengthened with FRP are provided in the AASHTO *Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Members* (2012). As part of an FHWA study on bridge strengthening (Chajes et al., 2019), new design examples were developed and existing design examples were identified for inclusion in the study. Relevant examples to load rating include strengthening a steel truss (Ahlskog, 2018a), shear and flexural strengthening of a steel plate girder with section loss due to corrosion (Ahlskog, 2018b), developing composite action and continuity in a stringer floorsystem (Ahlskog, 2018c), post-tensioning a concrete pier cap (Storlie, 2018), and two separate examples on shear and flexural strengthening of a concrete girder using externally bonded FRP (Zureick et al., 2010).

### 2.11.3 Historic Bridges

Historic bridges are those that meet the National Register of Historic Places' evaluation criteria. The historical significance of PennDOT's bridges is tracked in BMS2 under Item 5E04. Historic bridges should be rated in accordance with MBE Article 6A.9.2. Also refer to AASHTO's *Historic Bridge Preservation Guide* (2020), particularly Section 4, for guidance on analysis and evaluation.

Due to their age, historic bridges often do not have sufficient information (e.g., plans, material properties) to perform a computational load rating. In these cases and depending on the missing information, rating by engineering judgement (see PUB 238 IP Article 3.6.1.1), the use of non-destructive testing (see PUB 238 IP Article 3.7.1), or the use of instrumentation and load testing (see PUB 238 IP Article 3.7.3) may be appropriate. See Section 1.1.3.4 for guidance on noncomputational load ratings.

Refer to PennDOT's Standards for Old Bridges (1983) for select standard bridge details from 1918 through 1960 that may be applicable depending on the bridge type and age. Other resources containing useful historical information such as section properties and material specifications include Brockenbrough and Schuster (2018), Ferris (1953), Waddell (1889), AISC's historic shape references (AISC, 2022), and Meinheit and Felder (2014).

## 2.12 SECONDARY MEMBERS

Refer to Section 2.4 for guidance on when secondary members should be rated. The sections below provide specific rating guidance on cross frames, diaphragms, splices and connections, and lateral bracing.

### 2.12.1 Cross Frames and Diaphragms

Boundary conditions can have a significant effect on the cross frame or diaphragm load rating results and should be carefully considered when developing a model for rating these members. The end connection boundary conditions assumed for analysis should accurately replicate the actual boundary conditions within reason. Diaphragm or cross frame members with only one element of the cross section connected at the end (e.g., webs of W or channel shape diaphragms, webs of T shapes and single angle legs of cross frame members) are typically assumed to be pinned at their ends. For members with all elements of the cross section connected at the ends and for concrete diaphragms, it may be more appropriate to assume fixed boundary conditions. If available, consult diaphragm and cross frame design information within the original design plans to determine the designer's assumptions and intentions related to boundary conditions.

See Section 2.4, and specifically Section 2.4.2.2, for guidance on when to rate cross frames or diaphragms.

#### 2.12.1.1 Concrete

There are no explicit guidelines in the MBE or other load rating specification on when diaphragms in concrete superstructure bridges should be rated. However, several design specifications have provisions for when diaphragms should be analyzed and designed. These scenarios include:

- When subject to shear or torsion loads (AASHTO LRFD Article 5.12.4)
- When transferring loads from the superstructure to the substructure at abutments, piers, or other bearing locations (AASHTO LRFD Article 5.12.4, AASHTO Std. Spec. Articles 8.12 and 9.10)
- On horizontally curved superstructures (AASHTO LRFD Article 5.12.4, AASHTO Std. Spec. Articles 8.12 and 9.10)

It is recommended that diaphragms that fit any of the above scenarios be investigated. If determined to be primary load carrying members subject to live load, then they should be load rated.

The capacity of a diaphragm should be determined based on the type, properties, and geometry of the member. Appropriate methods to determine capacity may include conventional beam design (AASHTO LRFD Articles 5.6 and 5.7 and AASHTO Std. Spec. Articles 8.15 and 8.16), or the strut-and-tie method (AASHTO LRFD Article 5.8). Corven (2016) provides guidance on designing diaphragms in post-tensioned box girder bridges that can be adapted to load rating capacity calculations. From this manual, specific scenarios to consider for diaphragms at supports (i.e., end diaphragms) to transfer vertical shear forces include the following:

- Horizontal and vertical diaphragm forces due to eccentricity between the girder web shear force line of action and the bearing (see Figure 2.12.1.1-1)
- The strut-and-tie method can be used to determine diaphragm forces. In addition, localized effects should be checked, including shear friction between the web and diaphragm and direct tension at the bottom of the web (see Figure 2.12.1.1-2).

- Increased horizontal diaphragm forces for girders with inclined webs (see Figure 2.12.1.1-3).
- Further increase in horizontal diaphragm forces for girders with inclined webs and large eccentricity between the web and bearing (see Figure 2.12.1.1-4), which can necessitate diaphragm post-tensioning.

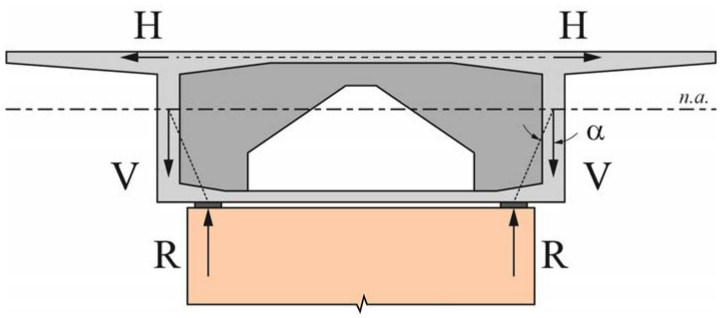



**Figure 2.12.1.1-1 – Diaphragm Forces Caused by Eccentric Girder Webs and Bearings (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

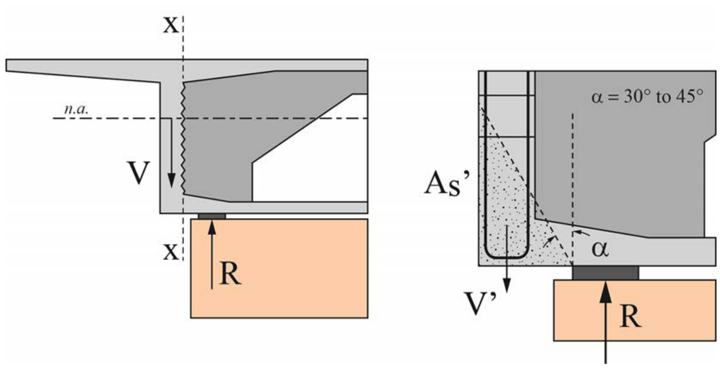



**Figure 2.12.1.1-2 – Localized Force Transfer Mechanisms: Shear Friction (Left) and Web Direct Tension (Right) (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

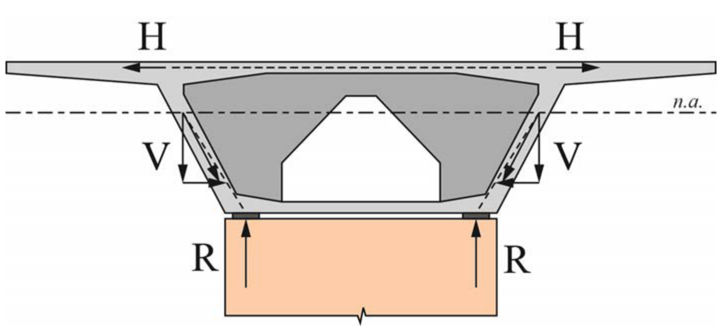



**Figure 2.12.1.1-3 – Diaphragm Forces for a Girder with Inclined Webs (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

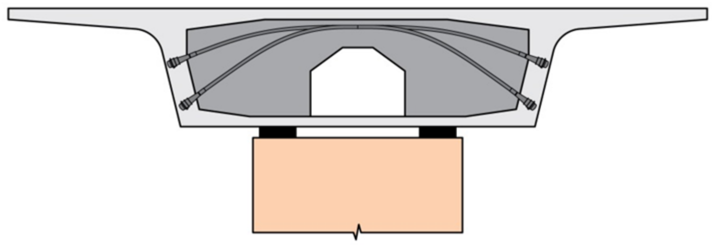


**Figure 2.12.1.1-4 – Diaphragm Post-Tensioning Used in a Girder with Large Web/Bearing Eccentricity and Inclined Webs (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

Similar guidance is provided for end diaphragm transfer of torsional forces, including:

- Forces on an A-shaped torsion diaphragm (Figure 2.12.1.1-5).
- Forces on a V-shaped torsion diaphragm (Figure 2.12.1.1-6).

For both cases above, diaphragm capacities and ratings should be determined using a suitable method, such as the strut-and-tie method.

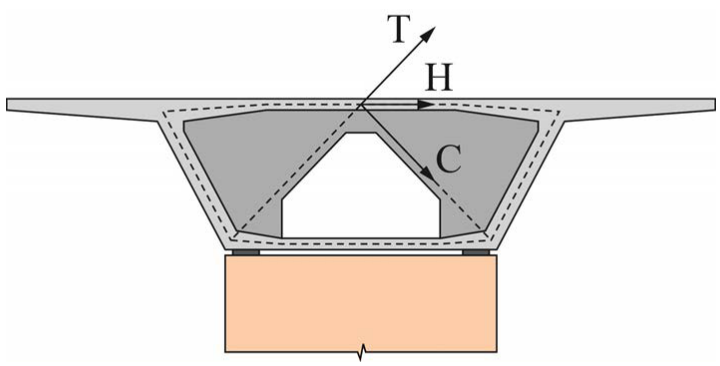



**Figure 2.12.1.1-5 – A-Shaped Torsion Diaphragm (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

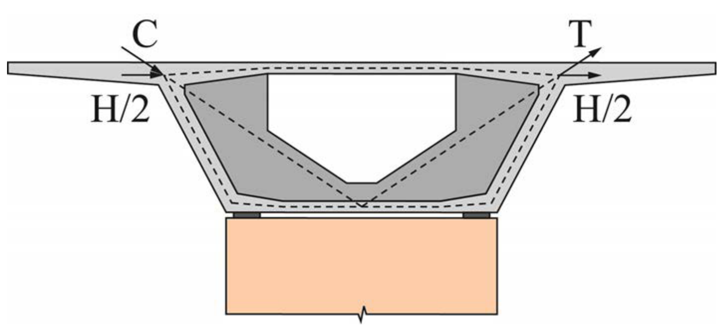



**Figure 2.12.1.1-6 – V-Shaped Torsion Diaphragm (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

In addition to the above guidance on single-cell box girder diaphragms, Corven (2016) discusses demand and capacity considerations for multi-cell box girder diaphragms. The strut-and-tie method is typically most appropriate for analyzing and rating these members. Two potential strut-and-tie layouts for an example bridge are shown in Figure 2.12.1.1-7 and Figure 2.12.1.1-8.

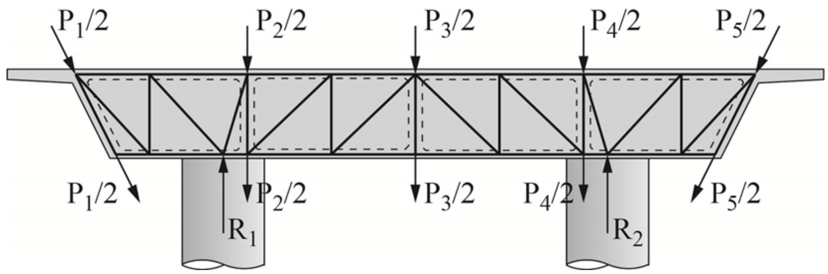



**Figure 2.12.1.1-7 – Potential Strut-and-Tie Layout for Diaphragms Considering Simply Supported Column Connection (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

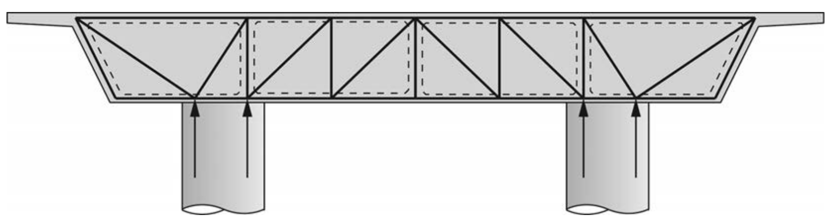


**Figure 2.12.1.1-8 – Potential Strut-and-Tie Layout for Diaphragms Considering Monolithic Column Connection (Taken from FHWA, *Post-Tensioned Box Girder Design Manual*, 2016)**

2.12.1.2 Steel

Cross frames and diaphragms that are part of horizontally curved steel superstructures are considered primary load carrying members and should be rated in accordance with MBE Articles 6A.6.9.7 and 6A.6.11.1 for LRFR method. Follow AASHTO Std. Spec. Article 10.20.1 for ASR and LFR methods. In addition, PennDOT considers cross frames and diaphragms on steel bridges with skew angles less than 70° as primary members that should be rated (see PUB 238 IE Article 4.3.5.6.8, and DM-4 Articles 4.6.2.2.1, 6.7.4.1, and Appendix E6P).

When cross frame and diaphragm members are required to be load rated, a suitable analysis should be performed to accurately determine force effects on these members. As outlined above, cross frames and diaphragms are considered primary load carrying members in superstructures with atypical framing plans (e.g., horizontally curved, heavily skewed). These structures often require an enhanced analysis method (i.e., 2D or 3D analysis) to determined force effects for load rating. Therefore, it is typically most convenient to model the cross frames or diaphragms within the same global superstructure model. Special considerations include the following:

- Diaphragms (e.g., plate diaphragms, rolled W or C shapes) in 2D and 3D models can be modeled directly using beam elements that capture the appropriate behavior (i.e., are of an appropriate finite element formulation).

- Conversely, cross frames (e.g., truss structures oriented in X, K, or inverted K configurations) in 2D grid or grillage or PEB models cannot be modeled using individual elements for each cross frame member and therefore should be modeled using a single member with equivalent stiffness. Appropriate methods for modeling equivalent stiffness of cross frames in 2D analyses are provided in DM-4 Appendix E6.2.1.P, and discussed in detail in AASHTO/NSBA (2019), Adams et al. (2019), and White et al. (2012). Methods to transform equivalent single member results back into force effects on each cross frame member are provided in Adams et al. (2019).

- In 3D models, individual cross frame members can be explicitly modeled, and the force effects on each member can be taken directly from the analysis results.

- For both 2D and 3D analysis of cross frames, it may be appropriate to use a reduced axial rigidity for single angles and flange-connected T-sections as discussed in AASHTO LRFD Article C4.6.3.3.4. See AASHTO/NSBA (2019) for additional discussions on cross frame stiffness.

- The fit condition used to detail the diaphragms and cross frames can result in locked-in forces in these members. The three most common fit conditions are:

    - No Load Fit (NLF): the girder webs are plumb and the diaphragms/cross frames are unstressed under unloaded conditions.

    - Steel Dead Load Fit (SDLF): the girder webs are plumb and the diaphragms/cross frames are unstressed under the steel dead load.

    - Total Dead Load Fit (TDLF): the girder webs are plumb and the diaphragms/cross frames are unstressed under the total dead load.

  The fit condition assumed for load rating should match that used for original construction when known. Consult design documents such as the design plans or shop drawings when available to determine the fit condition used for original fabrication and construction. If the fit condition assumed for rating differs from the original construction, the result can be either unconservative or conservative depending on the fit condition and bridge type. See DM-4 Article E6.4P for PennDOT guidance on fit conditions for skewed and/or horizontally curved steel I-girder bridges. Additional guidance and discussions can be found in AASHTO/NSBA (2019), Adams et al. (2019), and White et al. (2012).

The capacity and rating factor calculations for diaphragm and cross frames members should carried out appropriately depending on the force effects applied to each member (e.g., axial, shear, moment). When load rating older structures with complex framing plans, it is not uncommon for the cross frames or diaphragms to control the rating because they may not have been considered primary members at the time of design and construction. In scenarios when cross frames or diaphragms do not rate sufficiently, it may be appropriate to consider the insufficient members ineffective, remove those members from the analysis, and reevaluate the entire structure. If the remaining structure rates sufficiently, the bridge does not need to be posted provided DBE approval. Strengthening and/or retrofit should still be considered for the members with low ratings.

### 2.12.2 Splices and Connections

For guidance on when to rate splices and connections, see Section 2.4.2.2.

For guidance on rating gusset plates and connection plates in trusses, see Section 2.6.7.1. For guidance on rating bolted splices, see Section 2.6.7.2.

### 2.12.3 Lateral Bracing

See Section 2.4.2.2 for when lateral bracing members should be load rated.

When load rating lateral bracing members, follow MBE Article 6A.6.9.7 and AASHTO LRFD Article 6.7.5.1 for LRFR ratings. Follow AASHTO Std. Spec. Article 10.21 for ASR and LFR ratings. Lateral bracing members typically carry axial load alone, but this should be confirmed on a case-by-case basis. All applicable load cases (e.g., dead load, live load) should be evaluated, but the demands are typically live load dominated. Permanent bottom flange lateral bracing in I-girder bridges can be subject to significant live load forces (AASHTO LRFD Article C.6.7.5.1).

For axially loaded lateral bracing members in compression, the unbraced length can be conservatively assumed to be the full length of the member. If the ratings are unfavorable, the load rater has the following options to increase the load ratings:

- The unbraced length can be reduced to the last line of bolts in the connection or end of weld.
- If the lateral bracing crosses another lateral bracing in the “X” shape configuration, a brace point at the center of the “X” can be assumed if it is shown through a refined analysis that the other lateral brace member can brace the compression member in-plane and out-of-plane at this location.

Lateral bracing can be evaluated using AASHTOWare (BrR) for applicable bridge types. For other bridge types where the use of BrR is not appropriate that use an enhanced analysis method (i.e., 2D or 3D analysis), it is typically most convenient to model the lateral bracing within the same global superstructure model. In this case, the force effects can be combined with hand calculations to develop the load rating results.

## 2.13 COMMON MISTAKES

Common mistakes when performing load rating evaluations can be avoided with guidance from experienced load rating engineers and from the guidance within this Manual. A common mistake which occurs on the loading side of a rating is determining the appropriate live load distribution factors. Additional common mistakes on the capacity/resistance side often occur when determining the unbraced length for components in compression, or when determining whether a flexural member should be considered to act compositely with the deck. General guidance to avoid typical common mistakes regarding these topics is provided in the following sections.

### 2.13.1 Live Load Distribution Factors

LLDF's are typically calculated by the analysis programs such as BAR7, STLRFD, and AASHTOWare or they can be input as user defined values. Some programs require these values be input, such as stringers in any stringer-floorbeam systems. The engineer should understand the concepts behind the LLDF calculation. When using analysis programs that determine LLDF's, it is important to check the LLDF's being output from the program as the live load scenarios that need to be evaluated may differ than the programs default loaded lanes. This section discusses common mistakes and misunderstandings associated with the calculation of LLDF. The objective is to highlight these issues and provide clarity.

LLDF calculated per AASHTO Std. Spec. Article 3.23 is the fraction of wheel load. Refer to AASHTO Std. Spec. Article 3.12 to determine the reduction in live loads for various loaded lanes.

LLDF calculated per AASHTO LRFD Article 4.6.2.2 is the fraction of the lane or axle load that is carried by the member. The LLDF's can be used for bridges which meet the specific applicability requirements defined in the design specification and noted in PUB 238 IP Article 3.3.2.1. It is a common misinterpretation to apply multiple presence factors in addition to the LRFD distribution factor. Multiple presence factors have been implicitly included in the live load distribution factors in AASHTO LRFD Article 4.6.2. See Section 2.3.3 for discussion on multiple presence factors.

The rating engineer should also be aware that distribution factors are calculated separately for moments and shears. The term L, Span Length, which is used for the calculation of distribution factors in AASHTO LRFD Article 4.6.2.2.1 can sometimes be mistaken as the total length of the bridge. The length L is different for positive and negative moment regions and for different sections of continuous span bridges. An example is the situation when the interior span of a continuous bridge does not have a point of contraflexure (i.e., loading condition in which there is no live load on the interior span), extend the negative moment region of interior support to the centerline of the adjacent span (AASHTO LRFD Article 4.6.2.2.1).

Typically, LLDF's are calculated in accordance with the corresponding load rating methodology. However, when LFR distribution factors/load ratings are performed and found to be inadequate, LRFD distribution factors may be used in accordance with PUB 238 IP Article 3.4.3.3. The distribution factors in AASHTO LRFD Article 4.6.2.2 consider different parameters like girder moment of inertia, girder eccentricity, girder area, slab thickness, span length, etc., and are generally more accurate than AASHTO Std. Spec. distribution factors.

When computing the number of lanes for rating calculations, follow MBE Article 6A.2.3.2. AASHTO LRFD Article 3.6.1.1.1 is appropriate to compute the number of lanes for the LRFD design calculations, but should not be used for rating calculations.

MBE Article 6A.2.3.2 provides an alternate method for instructions on transverse positioning of vehicles on the roadway to evaluate LLDF's. The wheel can be placed on the outside edge of lane, but no closer than 2.0 ft. from the curb. If the rating of the exterior or first interior girder is less than 1.0, the rating engineer can use a modified rating method of limiting the vehicle placement as specified in MBE Articles 6A.2.3.2 and 6B.6.2.2. The vehicles can be placed in the actual travel lanes on the bridge instead of the loaded lanes to achieve better ratings. This provision should only be considered for exterior beams, which are still functioning as designed with minimal section loss and no signs of distress. Use of this provision must be approved by the Assistant Chief Bridge Engineer - Inspection. The alternate method should not be considered for EVs.

See guidance in Sections 1.8.1 and 1.8.2, on placement of emergency vehicles along with vehicular traffic in the other lanes. The distribution factor should be calculated to account for this placement of live loads. Instead of using a multi-lane distribution factor for the emergency vehicle and design truck, apply the single lane live load distribution factor to the emergency vehicle. The multi-lane live load distribution factor minus the single lane live load distribution factor will be applied to the adjacent vehicle. Similarly, for live load distribution of a heavy permit truck using the distribution factor tables may be overly conservative. Consider using a refined approach for calculation of the distribution factors if the use of LRFD LLDF's result in unacceptable ratings.

For the purpose of calculating the live load distribution factors and multiple presence factors with pedestrian load, sidewalks should be considered as an additional loaded lane. See further discussion on sidewalks in Section 2.3.1.

An important concern with skewed bridges is the calculation of the skew correction factors. In skewed bridges, due to the bridge geometry and redistribution of bridge stiffness, the loads tend to concentrate more at the obtuse bridge corner (stiffer) as compared to the acute bridge corner. These effects are inherently captured in 3D bridge analysis. However, when a line girder analysis is used, the torsional effects from skew become large and need to be applied to the corresponding non-skewed bridge. Skew correction factors should be applied as discussed in Section 2.2.3. Shear force due to dead load should not be modified for skew effects.

When the shear key connection in adjacent box-beam bridges becomes degraded due to reasons such as thermal effects, moisture seepage, misalignment, relative movement of beams, cracking of wearing surface or poor construction methods, the distribution of live load will need to take these effects into account. Refer to Section 2.5.1.2 for further direction on calculation of live load distribution in box beams with a degraded shear key. For analyzing prestressed or reinforced concrete adjacent box beam bridges with functional shear keys and fully effective tie rods or post-tensioning using LFD methodology, use AASHTO Std Spec. Article 3.23.4. See PUB 238 IP Article 3.3.2.2 for additional guidance.

When bridges are widened to increase shoulder width or to add more lanes, do not use the live load factors corresponding to the original bridge cross-section. Live load distribution factors should be recalculated. Adding new longitudinal girders or stringers in a cross-section or modifying the girder spacing will change the LLDF's. In some situations, the existing girders/stringers may be retained and additional exterior girders are placed. In this case, the fascia girder of the existing bridge should be considered as an interior girder and be evaluated for adequate strength due to redistribution of live loads. See Sections 2.2.1 and 2.11.1 for detailed discussion on non-standard bridges and widened bridges.

AASHTO LRFD provides guidance on the live load distribution factor calculation of grid decks (fully filled, partially filled or unfilled) with thickness greater than 4.5 inches. For Load Factor ratings, refer to AASHTO Std. Spec. Table 3.23.1 for steel grid deck.

Another common error which has been observed for evaluation of permit trucks is the multiple presence factor not being divided out from the single lane distribution. This is applicable only to LLDFs computed using LRFR methodology. When a 3D refined analysis is used, place permit truck in one lane and legal load vehicle in other lanes. Dividing out the multiple presence factor is not required for live load results obtained from 3D analysis.

An interior beam underneath a longitudinal deck joint should be assessed as a fascia or exterior beam using guidance from PUB 238 IP Article 3.3.

### 2.13.2 Unbraced Length for Components in Compression

The unbraced length is the length between brace points at which the member is braced against movement. The capacity of a compression member depends on the slenderness ratio, KL/r, where K is the effective length factor, L is the unbraced length and r is the radius of gyration.

Structural connections can have different boundary conditions for translation and rotation. Use AASHTO LRFD Table C4.6.2.5-1 for effective length factors for various boundary conditions. If a member has intermediate bracing, the unbraced length should be calculated for each segment of the member and the controlling value used in capacity calculations.

MBE Article 6A.6.9.3 provides guidance on determination of unbraced length of girder compression flange braced by concrete deck. Even in the absence of shear connectors, if there is adequate contact between the uncracked deck and girder, it is reasonable to assume that the top flange of the girder is fully braced by the deck for the buckling check.

### 2.13.3 Composite Action

There are some common mistakes associated with the evaluation of composite action in steel and concrete girders. Composite action is considered when the deck is designed to act compositely with the supporting member. The most common way to achieve this is through shear connectors. If field inspection data shows there is movement between the bottom of deck and top of supporting member, it may indicate that the member is not fully composite with the deck and the ratings should not account for composite action (MBE Article 4.3.5.5).

Concrete Encased Steel I-beams do not require mechanical connectors to furnish composite action. The bond between steel and concrete ensures composite action for flexural and axial members for service and fatigue limit states. Sufficient shear transfer must be provided to consider composite at the strength limit state. See Sections 2.6.1.2 and 2.6.2.1 and MBE Article 6A.6.9.4 for further discussion on composite action in Concrete Encased I-beams. If the deck is found to be in poor condition per the most recent Inspection Report, the rating analysis should be performed using non-composite section properties regardless of the method of construction (PUB 238 IE Article 6B.6.1).

In unshored construction, the girder or stringer is non-composite with the deck and supports the self-weight, weight of deck and formwork. For evaluation of the composite section, see MBE Article 6A.6.9.2. In the absence of information about shored or unshored construction used during construction, assuming unshored construction will be conservative and is recommended.

In the case of girders detailed without shear connectors, sometimes it can be incorrectly assumed that no composite interaction exists between the deck and girder. This assumption can lead to conservative ratings. Girders without shear connectors can still utilize some level of composite action if there is sufficient shear transfer between the girder and concrete. Reference guidance in MBE Article C6A.6.9.3

for more information. It should be noted, the steel mesh shown on PennDOT's standard plans is not sufficient to allow the bridge to be considered fully composite.

For rating of a composite beam in negative moment regions, the deck longitudinal reinforcement can be included in the rating analysis.

For girders with unknown composite action with the deck, conservatively assume non-composite section properties for ratings. If the rating factor for Legal Loads is below 1.0, follow guidance provided in MBE Article C6A.6.9.4 to gauge the amount of composite action that can be considered. Unless the inspection or testing shows that there is slip or deterioration between the bottom of deck and top of girder in which case proceed with the evaluation of the section as non-composite. Using the concrete deck properties in girder composite section capacity will yield less conservative results and should only be used if the engineer can evaluate and verify that composite action is occurring.

In some older bridges, it is possible that despite the presence of shear connectors in contract plans, composite section properties were not considered in the original girder design. The details of girder, deck and shear connectors should be carefully evaluated to ascertain the level of composite action available.

If the bridge is being rehabilitated for higher deck loads or present-day live loads, follow guidance from latest standards to evaluate composite section behavior and provide additional shear connectors as required.

**REFERENCES**

AASHTO. 2018. A Policy on Geometric Design of Highways and Streets, 7th Edition with Errata, GDHS-7, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2020. Historic Bridge Preservation Guide, 1st Edition, HBP-1, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2020. Culvert and Storm Drain System Inspection Guide, 1st Edition, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO/NSBA. 2019. G13.1 Guidelines for Steel Girder Bridge Analysis, Third Edition. AASHTO/NSBA Steel Bridge Collaboration.

Adams, A., N. Galindez, T. Hopper, T. Murphy, P. Ritchie, V. Storlie, and J. Weisman. 2019. Manual for Refined Analysis in Bridge Design and Evaluation, FHWA-HIF-18-046, Federal Highway Administration, Washington, DC.

Ahlborn, T. M., Gilbertson, C. G.;Kiefer, John A.;Colling, T. K. 2013. Assessment of ODOT culvert load rating spreadsheets for use in Michigan.

AISC. 2022. "Historic Shape References". https://www.aisc.org/publications/historic-shape-references/#28363.

Azizinamini, A., Yakel, A., Farimani, M. 2005. Development of a Steel Bridge System - Simple for Dead Load and Continuous for Live Load

Brockenbrough, R. L., and J. Schuster. 2018. Design Guide 15: Rehabilitation and Retrofit, Second Edition. American Institute of Steel Construction (AISC), Chicago, IL.

Chajes, M. 2002. Load Rating of Arch Bridges, Delaware Center for Transportation, University of Delaware, Newark, DE.

Chajes, M., T. Rollins, H. Dai, and T. Murphy. 2019. Report on Techniques for Bridge Strengthening: Main Report. Office of Infrastructure – Bridges and Structures, Federal Highway Administration, Washington, DC.

Coletti, D., Z. Fan, W. Gatti, J. Holt, and J. Vogel. 2005. Practical Steel Tub Girder Design, National Steel Bridge Alliance, Chicago, IL.

Corven, J. 2016. Post-Tensioned Box Girder Design Manual, FHWA-HIF-15-016, Federal Highway Administration, Washington, DC.

Dorton, R. A., and R. Reel. 1997. NCHRP Synthesis 249: Methods for Increasing Live Load Capacity of Existing Highway Bridges. Transportation Research Board, Washington, DC.

Ferris, H.W. 1953. Historical Record Dimensions and Properties Rolled Shapes. American Institute of Steel Construction (AISC), New York, NY.

References

FHWA. HIBS-1. 2016. Load Rating for the FAST Act's Emergency Vehicles, Federal Highway Administration, Washington, DC.

Gerasimidis, S., S. Brena, and B. Knickle. 2019. Development of Load Rating Procedures for Deteriorated Steel Beam Ends, University of Massachusetts Amherst UMass Transportation Center, Amherst, MA.

Ghosn, Michael and Fred Moses. 1998. NCHRP Report 406: Redundancy in Highway Bridge Superstructures. Department of Civil Engineering, The City College and University of New York, NY.

Klaiber, F. W., K. F. Dunker, T. J. Wipf, and W. W. Sanders, Jr. 1987. NCHRP Report 293: Methods of Strengthening Existing Highway Bridges. Transportation Research Board, Washington, DC.

Kulicki, J.M. 1983. Load Factor Design of Truss Bridges with Applications to Greater New Orleans Bridge No. 2, Transportation Research Record 903. Transportation Research Board, National Research Council, Washington, DC.

Kulicki, J. M., Z. Prucz, D. F. Sorgenfrei, D. R. Mertz, and W. T. Young. 1990. NCHRP Report 333: Guidelines for Evaluating Corrosion Effects in Existing Steel Bridges, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

Lin, Z., & Zhao, J. 2011. Revisit of AASHTO Effective Flange-Width Provisions for Box Girders. Journal of Bridge Engineering, 16(6), 881-889.

Meinheit, D. F., and A. L. Felder. 2014. Vintage Steel Reinforcement in Concrete Structures, 1st Edition. Concrete Reinforcing Steel Institute (CRSI), Schaumburg, IL.

Naito, C., L. Jones, and I. Hodgson. 2010. Inspection Methods & Techniques to Determine Non Visible Corrosion of Prestressing Strands in Concrete Bridge Components Task 3 – Forensic Evaluation and Rating Methodology. Advanced Technology for Large Structural Systems (ATLSS) and Lehigh University, Bethlehem, PA.

National Corrugated Steel Pipe Association (NCSPA). 1995. Load Rating and Structural Evaluation of In-Service, Corrugated Steel Structures, NCSPA Design Data Sheet No. 19, NCSPA, Dallas, TX.

NSBA. 2022., Steel Bridge Design Handbook, Chapter 13 Bracing System Theory and Design for I-Girders and Tub Girders.

PennDOT. 1983. Standards for Old Bridges, Volumes 1 through 3, Bridge Division, Bureau of Highway Design, Department of Transportation, Commonwealth of Pennsylvania, Harrisburg, PA.

Ream, A., B. Frankl, B. Chavel, B. Leshko, B. Schiller, D. Paterson, P. Wilson, H. Law, C. Felice. 2019. Reference Guide for Load Rating of Tunnel Structures, FHWA-HIF-19-010, Federal Highway Administration, Washington, DC.

Ritter, M. A. 1990. Timber Bridges: Design, Construction, Inspection, and Maintenance, US Forest Service, Washington, DC.

Salkar, R., A. Salkar, and W. Davids. 2015. Crippling of Webs with Partial-Depth Stiffeners under Patch Loading. AISC Engineering Journal, Volume 52(4), 221-231.

Waddell, J. A. L. 1889. General Specifications for Highway Bridges of Iron and Steel. S.G. Spencer, Kansas City, MO.

White, D. W., D. Coletti, B. W. Chavel, A. Sanchez, C. Ozgur, J. M. Jimenez Chong, R. T. Leon, R. D. Medlock, R. A. Cisneros, T. V. Galambos, J. M. Yadlosky, W. J. Gatti, and G. T. Kowatch. 2012. NCHRP Report 725: Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges, National Cooperative Highway Research Program, Transportation Research Board, Washington, DC.

WisDOT. 2022. Bridge Manual, Chapter 36, Box Culverts, Bureau of Structures, Department of Transportation, State of Wisconsin, Madison, WI.

Zureick, A., B. R. Ellingwood, A. S. Nowak, D. R. Mertz, and T. C. Triantafillou. 2010. NCHRP Report 655: Recommended Guide Specification for the Design of Externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements. Transportation Research Board, Washington, DC.

# OVERVIEW OF TYPICAL BRIDGE TYPES AND EXAMPLES

## 3.1 GENERAL

This Section provides an overview of the typical bridge types in PennDOT's inventory. In addition, complete load rating examples are provided for several bridge types.

## 3.2 CONCRETE T-BEAMS

This Section covers the rating of reinforced concrete T-beam bridges.

### 3.2.1 Policies and Guidelines

Most T-beam bridges can be rated by analytical methods based on design plans and field measurements. If design plans are not available, PennDOT standard plans can be utilized if field measured values are in reasonable agreement with one of the standard plans.

If plans are not available and field measurements are not in reasonable agreement with PennDOT standard plans, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.2.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating.

#### 3.2.2.1 LFR or ASR Method

Based on the age of these bridges, the typical load rating method will be LFR. PennDOT's BAR7 program (Bridge Type CTB) should be utilized to analyze and rate these bridges when the necessary information is available to complete an analytical rating.

#### 3.2.2.2 LRFR Method

Most T-beam bridges were built prior to 2011 and designed by load factor or allowable stress methods; therefore, the LRFR rating method would rarely be utilized.

### 3.2.3 Live Load and Dead Load Distribution

#### 3.2.3.1 LFR or ASR Method

Pub. 238 Section 3.3 provides guidance on the distribution of live load to longitudinal girders. Typically, the live load distribution factor for exterior beams is in accordance with the AASHTO Standard Specification.

The typical distribution factors are as follows when AASHTO distribution factors are applicable:

| Exterior | | Interior | |
|---|---|---|---|
| Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.3.1.2) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.2 and Table 3.23.1) |
| Lever Rule* | Lever Rule* | Lever Rule* | • S/6.5 (one lane)<br>If S > 6' use Lever Rule<br>• S/6.0 (2 or more lanes)<br>If S > 10' use Lever Rule |

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

Concrete T-Beam bridges built to the PennDOT standards (listed below) have beam spacings less than 6'.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2.2.3 and Pub. 238 IP 3.3.3.1 for additional discussion of skew.

Dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all beams (AASHTO Standard Specification 3.23.2.3.1.1).

3.2.3.2 LRFR Method

Most T-beam bridges were built prior to 2011 and designed by load factor or allowable stress methods; therefore, the LRFR rating method would rarely be utilized.

### 3.2.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1918-1930 Volume 1 | Reinforced Concrete Bridge | 2/27/1922 (Latest Revision 8/5/1925) | S-201-30 |
| Online document not available | Reinforced Concrete Bridge | 1926-1930 | S-195 through S-310 |
| Standards for Old Bridges 1931-1940 Volume 2 | Reinforced Concrete Bridge | 12/15/1939 | S-195 through S-310 |
| Standards for Old Bridges 1931-1940 Volume 2 | Reinforced Concrete T-Beams | 12/15/1939 | All S-515 drawings |
| Standards for Old Bridges 1961-1965 Volume 4 | Standard Reinforced Concrete T-Beam Bridges | 2/25/1965 | ST-101 |

### 3.2.5 Modeling Section Properties and Deterioration

See Section 2.5.1.2 for a discussion of modeling section loss.

Bent up bars, aka trussed bars, were typically provided to supplement shear stirrups at the beam ends. BAR7 applies the bent up bars from the centerline of bearings to a distance of (BEAM DEPTH)/Tan α; however, this rarely applies to where the actual bars are in the PennDOT Standard T-beams. S-195 for example, has an exterior beam depth of 22.5" and α = 45°, therefore BAR7 would apply the bent up bar from 0.00' to 1.88'. However, the bend in "bar b" in the standard drawing is from 0.98' to 2.04'. Additionally, BAR7 does not evaluate shear within D/2 from the support, where D is the distance from the tension reinforcement to the top of the section. In the S-195 example, D = 16" for an exterior beam, which means BAR7 will not evaluate shear in the first 0.67'. Therefore, where shear is first checked at 0.67', BAR7 considers a bent up bar where none is present. Because of this issue, and that BAR7 cannot consider bars bent up in multiple locations, do not use this method to enter the bars.

For BAR7 to properly account for the vertical stirrups combined with bent up bars, the vertical stirrup spacing must be reduced to account for the bent up bars. The reduced spacing, s', is calculated as shown below, where s is the vertical stirrup spacing, $A_{vb}$ is the area of the bent up bars, and $A_{vs}$ is the area of vertical stirrup (both legs).

$$s' = \frac{1}{\left(\frac{1}{s} + \frac{A_{vb} sin\alpha}{A_{vs} d}\right)}$$

Per AASHTO Std. Spec. 8.16.6.3.4:

$$V_s = A_{vb} f_y sin\alpha \leq 3\sqrt{f_c{}'} b_w d$$

Therefore:

$$A_{vb} \leq \frac{3\sqrt{f_c{}'} b_w d}{f_y sin\alpha}$$

Note, if the vertical stirrups are deteriorated, the spacing distance in the 1 / s factor can be replaced by $s_i + s_{i+1}$ $(1-A_{s,current}/A_{s,orginal})$ which creates a linear relationship between no section loss as the same spacing, and 100% section loss is twice the spacing. In this formula, $s_i$ is the larger distance to the adjacent stirrup and $s_{i+1}$ is the smaller distance. If the stirrup spacing is constant, then $s_i = s_{i+1}$.

Because the vertical stirrup spacing changes frequently and the "b" and "c" bars apply at different and sometimes overlapping areas, the reduced spacing should be calculated at multiple locations. BAR7 allows only 6 stirrup spacing ranges per beam, choose spacings and ranges that conservatively represent the more detailed actual ranges. If shear controls, ensure the spacing is accurate at the location which controls.

Note, BAR7 has the option to use the 1973 or earlier AASHTO Specifications for shear ratings of a reinforced concrete member. This option should only be used for the rating of an existing bridge that was designed using the 1973 or earlier AASHTO Specifications.

### 3.2.6 Standard Practices

In the original standard drawings, square reinforcement is identified with a □ and round reinforcement has a ∅. Most of the primary reinforcement in the T-beams are square bars.

The centerline of bearing to centerline of bearing span length is typically taken as the clear span distance plus 9” at each end. The original decks were 11” thick with a 2.5” integral wearing surface. The 2.5” of integral concrete wearing surface shall not be considered structural.

The sides of the T-beam stems were typically tapered to facilitate removal of the forms in each bay. The width of the exterior beams is ½” wider at the top than the bottom, and the interior are normally 1” wider at the top. When determining the beam sizes and spacing, field measurements must be consistently taken from the top or the bottom. For instance, if the bay widths are measured under the deck and the beams widths at the bottom the summation will not equal the out-out of the bridge. The average between the top and bottom of the stem should be entered into BAR7.

In most cases, $f'_c$ = 3.0 ksi and $f_y$ = 33.0 ksi.

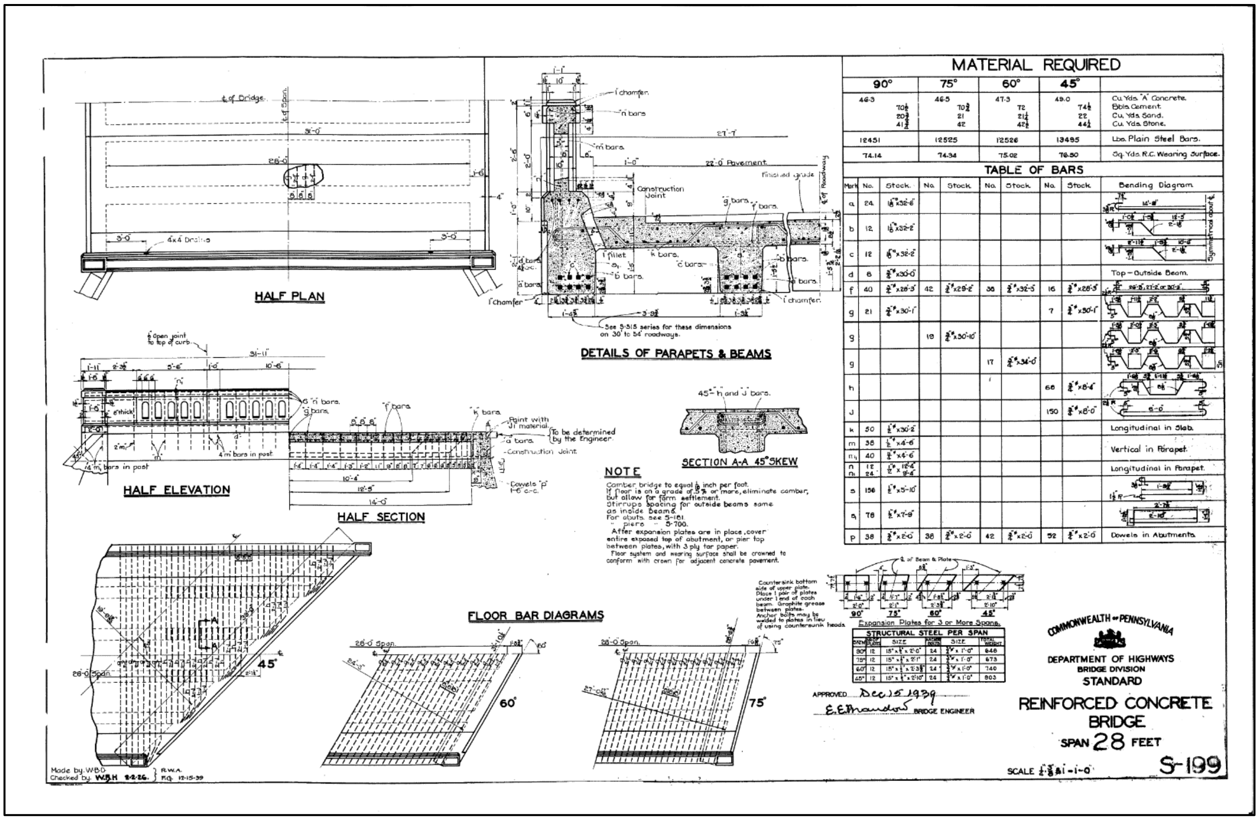
### 3.2.7 Common QA Findings

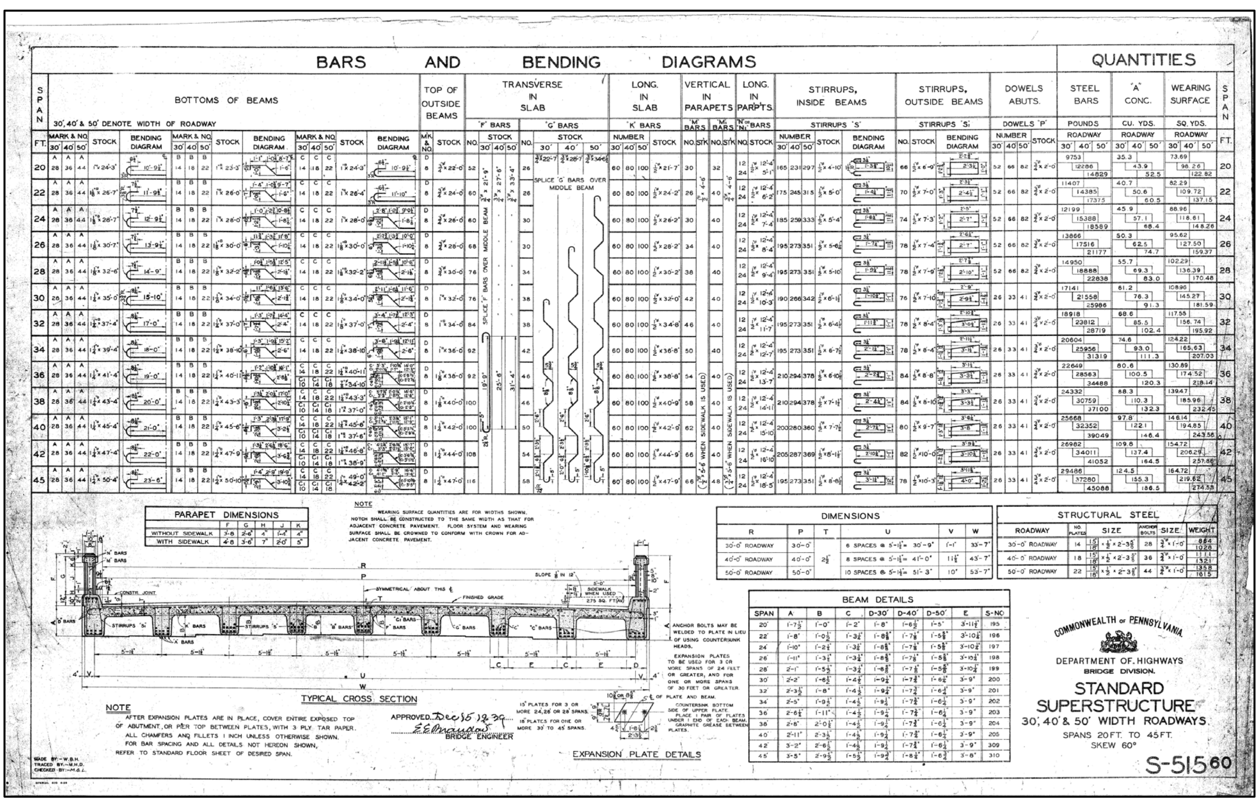
This Section is in development and will be provided in future editions of this manual.

### 3.2.8 Sample Load Rating

This Section contains sample load rating calculations for an interior and exterior beam for a single span non-composite reinforced concrete T-beam structure shown below. The analysis will be performed using PennDOT’s Bridge Analysis and Rating (BAR7) Program, Version 7.15.0.0 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part IV (DM-4).

The single span superstructure consists of an 8 ½” reinforced concrete deck supported on seven (7) reinforced concrete T-beams with a PennDOT skew angle of 60°. The bridge information is based on the Pennsylvania Department of Highways Reinforced Concrete Bridge Standard S-199 and Standard Superstructure S-515-60. The existing pigeon hole barriers were replaced with 3’-6” x 1’-0” vertical wall barriers.


MATERIAL REQUIRED
TABLE OF BARS
HALF PLAN
DETAILS OF PARAPETS & BEAMS
HALF ELEVATION
HALF SECTION
SECTION A-A 45° SKEW
FLOOR BAR DIAGRAMS
COMMONWEALTH of PENNSYLVANIA
DEPARTMENT OF HIGHWAYS
BRIDGE DIVISION
STANDARD
REINFORCED CONCRETE BRIDGE
SPAN 28 FEET
S-199


BARS AND BENDING DIAGRAMS
QUANTITIES
PARAPET DIMENSIONS
DIMENSIONS
STRUCTURAL STEEL
BEAM DETAILS
TYPICAL CROSS SECTION
EXPANSION PLATE DETAILS
COMMONWEALTH of PENNSYLVANIA
DEPARTMENT OF HIGHWAYS
BRIDGE DIVISION.
STANDARD
SUPERSTRUCTURE
30', 40' & 50' WIDTH ROADWAYS.
SPANS 20 FT. TO 45 FT.
SKEW 60°
S-515 60

3.2.8.1 Load Rating Summary Form Including Section Loss

LOAD RATING SUMMARY FORM

| | | | |
|---|---|---|---|
| Done By: | ABC | Date: | |
| Checked By: | XYZ | Date: | |

| | |
|---|---|
| Structure ID (5A01): | 02-3070-0080-0205 |
| Inspection Date (7A01): | 11/18/2021 |
| Facility Carried (5A08): | McClaren Road |
| Feature Intersected (5A07): | Montour Run |
| Structure Type (6A26 - 6A29): | Single Span Reinforced Concrete T-Beam Bridge |
| Spans / Members Analyzed: | Span 1/Interior Girders including Section Loss |
| Analysis Method: | LFD |
| PennDOT Program / Version: | BAR7 Version 7.15.0.0 |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 0.80 | 16.1 | 1.34 | 26.8 | 1.34 | 26.8 | * | * | M | M |
| HS20 | 0.73 | 26.2 | 1.21 | 43.7 | 1.21 | 43.7 | * | * | M | M |
| ML80 | 0.51 | 18.6 | 0.85 | 31.0 | 0.85 | 31.0 | * | * | M | M |
| TK527 | 0.56 | 22.2 | 0.93 | 37.0 | 0.93 | 37.0 | * | * | M | M |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 0.76 | 21.8 | 1.26 | 36.3 | 1.26 | 36.3 | * | * | M | M |
| EV3 | 0.49 | 20.9 | 0.81 | 34.8 | 0.81 | 34.8 | * | * | M | M |

Comments/Assumptions*:

Superstructure and substructure condition ratings are 5 and 6, respectively with isolated section loss. Therefore, for an ADTT < 500, the SLC Factor equals 1.0 per PennDOT Publication 238, Table IP 4.3.2-1.
Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.
BAR7 analysis includes section loss for the flexural and shear reinforcement.
* Controlling Member is Interior Beam in Span 1.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

3.2.8.2 Interior Beam Load Rating Analysis Including Section Loss

3.2.8.2.1 BAR7 Input Parameters

A typical interior T-beam girder was selected and will be rated using BAR7. Many of the BAR7 input parameters can be left blank. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

Project Identification

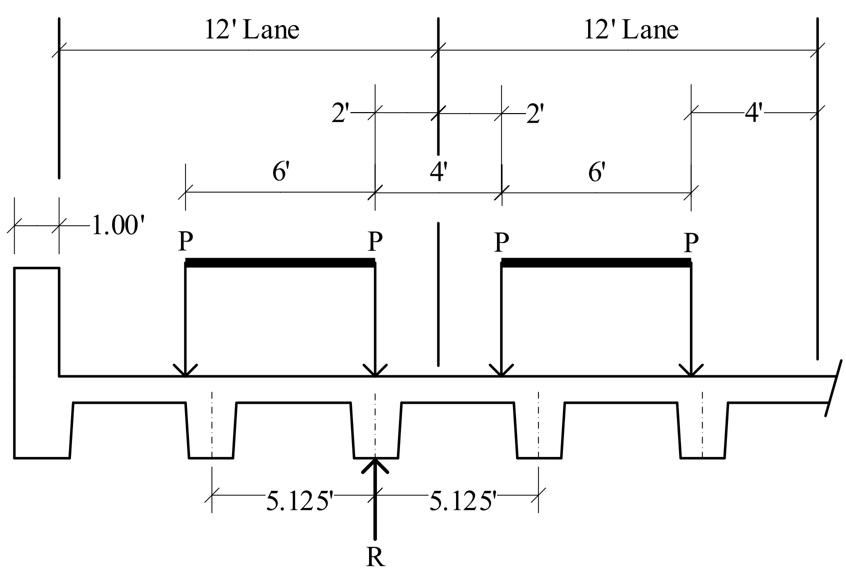
- Project Identification = "=BRRAT"
- Structure ID = 02307000800205
- Description = TYP INTERIOR BEAM
- Bridge Type = CTB
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 0 for Normal Output

Bridge Cross Section and Loading

- Beam Spacing, S = 5' – 1 ½" per S-515 = 5.125 ft, Use 5.13 ft due to 2 decimal BAR7 limitation.

- Distribution Factor – Shear

  Per AASHTO 3.23.1, the interior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the two-lane condition shown below, the reduction in load intensity factor is 1.0.




$DF_V = P[1 + (5.125' - 4')/5.125'] \times 1.0 = 1.220$ wheels
$DF_V = 1.220$ wheels / (2 wheels/axle) = 0.610 axles

- Distribution Factor – Moment

  Per AASHTO Table 3.23.1, the interior beam distribution factor for moment is calculated as S/6 for bridges with two or more traffic lanes. In accordance with AASHTO 3.12.2, the reduction in load intensity is not applicable when Table 2.23 if used for moment in longitudinal beams.

  $DF_M$ = S / 6.0 = 5.125 ft / 6.0 = 0.854 wheels x (1 axle/2 wheels) = 0.427 axles

- Distribution Factor – Deflection
  Per AASHTO 8.13.2, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
  Roadway Width = 30.00 ft per Standard Drawing S-515
  $N_L$ = 30.00 ft / 12 ft = 2.50 ==> Use 2 lanes
  Reduction Factor = 1.0 for 2 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 2 lanes x 1.0 / 7 beams = 0.286 lanes/beam

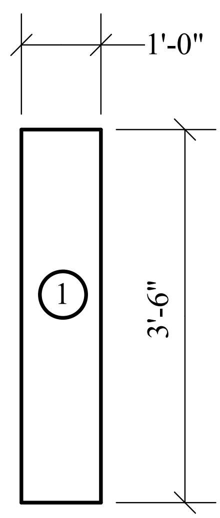
- Deck Thickness = 8.5 in

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight and deck slab weight, which are calculated by BAR7. Weight due to SIP forms, haunch concrete, intermediate diaphragms, and stiffeners, etc. are equal to zero.

  Total Dead Load 1 = 0 kips/ft

- Dead Loads – DL2
  Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  Roadway Barriers




Area 1 = 12 in x 42 in = 504 $in^2$

Barrier Weight = 504 $in^2$ / 144 $in^2/ft^2$ x 0.150 kcf = 0.525 kips/ft

Asphalt Wearing Surface
Per S-199 and S-515 Dimension T, the asphalt wearing surface is 2 ½" thick. However, a recent inspection indicates that the total wearing surface is 11 ½".
Overlay Weight = (11.5 in /12 in/ft) x 30 ft x 0.140 kcf = 4.025 kips/ft

Total Dead Load 2 = (2 x 0.525 kips/ft + 4.025 kips/ft) / 7 beams = 0.725 kips/ft

- Concrete Compressive Strength, $f'_c$ = 3 ksi for Class A Concrete per Table IP 3.7.2.2-1.
- Modular Ratio, n = 9 for Class A Concrete per DM-4 8.2

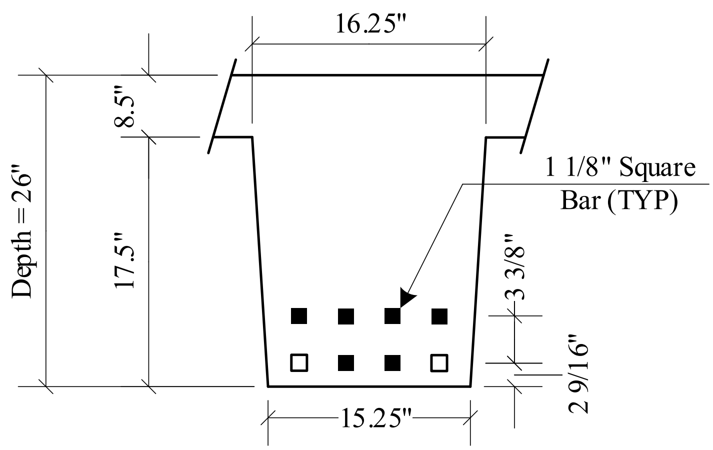
Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = 28.0 ft clear span + 2 x (9 in/12 in /ft) from front face to centerline of bearing = 29.50 ft per Standard Drawings S-199

Concrete Member Properties

During the field inspection of the structure, section loss and deterioration were noted for the beams' flexural and shear reinforcement. Over the middle portion of the span length, the outer bars in the bottom row of flexural reinforcement have 100% section loss. These two bars will be removed from the rating analysis. In addition, the first stirrup beyond the front face of abutment is broken on both sides of the web. Therefore, assume any remaining stirrups over the abutment seat are ineffective.

- Type = T for T-beam Bridge
- Depth from top of slab to bottom of web, Depth = Dimension B per S-515 + Slab Depth per S-199 = 17.5 in + 8.5 in = 26 in
- Width of the Web, B = 1'-3 ¼" for Dimension C per S-515 + ½" for sloped faces = 15.75 in (avg)
- Distance of the tension reinforcement (AS) from the top of section, D = 26 in – 4.49 in = 21.51 in
    - Distance to 1st Row (4 "a" bars) = 2.5625 in per S-199
    - Distance to 2nd Row (2 "b" & 2 "c" bars) = 2.5625 in + 3.375 in = 5.9375 in per S-199
    - All bars are 1 1/8" square bars, Bar Area = 1.125 in x 1.125 in = 1.266 $in^2$
    - Total Reinforcement Area = 7 x 1.266 $in^2$ = 8.859 $in^2$
    - Bar Group CG = [1.266 $in^2$ x (3 x 2.5625 in + 4 x 5.9375 in)] / 8.859 $in^2$ = 4.49 in

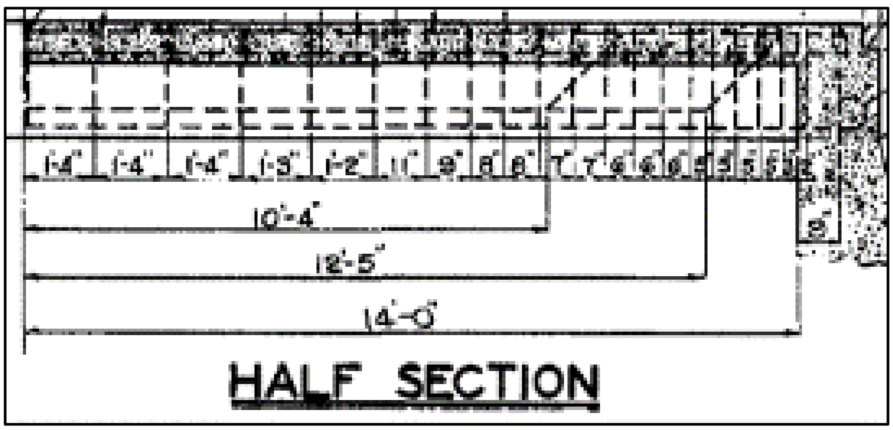


□ - 50% Removed Bar Due to Section Loss

- Area of tension reinforcement, AS = 8.86 in$^2$
- Compression reinforcement not considered. D' = 0, A'S= 0
- Reinforcement yield strength, $f_y$ = 33 ksi per AASHTO MBE Table 6B.5.2.3-1
- Stirrup details = Y indicating stirrup reinforcement will be entered
- Stirrup area of bent up bars, AV = 2 "b" bars per S-199 = 2 x 1.125 in x 1.125 in = 2.53 in$^2$
- Specs = 3 for the program to use the 1973 or earlier AASHTO Specifications for shear ratings of a reinforced concrete memberOK
- Alpha = Blank to default to 45 degrees.
- Integral wearing surface = 0 in.

Stirrup Details

The sketch below is taken from PennDOT's S-199 Standard Drawing.




- Stirrup Area = 0.20 in$^2$ for one leg of a #4 (1/2" diameter) stirrup
- Yield Strength of Reinforcement Stirrups, $f_y$ = 33 ksi per AASHTO MBE Table 6B.5.2.3-1
- Location = 0.00 ft
- Spacing = 9 in (centerline of bearings to abutment front face) + 3 in space + 5 in space = 17 in (Removes 1st 2 bars shown above, one on each side of abutment front face)
- Location = 17 in = 1.42 ft
- Spacing = 5.0 in
- Location = 1.42 ft + 3 x 5 in/12 = 2.67 ft per S-199
- Spacing = 6.0 in
- Location = 2.67 ft + 3 x 6 in/12 = 4.17 ft per S-199
- Spacing = 7.0 in
- Location = 4.17 ft + 2 x 7 in/12 = 5.33 ft per S-199
- Spacing = 8.0 in
- Location = 5.33 ft + 2 x 8 in/12 = 6.67 ft per S-199
- Spacing = 16.0 in maximum spacing to centerline of span due to limitations of BAR7 locations

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 02-3070-0080-0205 | | |
| Description: | | | |
| Bridge Type: | CTB | | Concrete T-Beam |
| SLC Level: | | | Super = 5 & Sub = 6; ADTT = 23 therefore SLC = 1.0 |
| Lanes: | | | N/A for CTB |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 0 | | |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | | | |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | | | Leave blank and calculate DF |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | | | |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for CTB |
| Overhang or Spacing: | 5.13 | ft | |
| CL of Girder: | -- | ft | Leave blank for CTB |
| Roadway Width: | -- | ft | Leave blank for CTB |
| Distr Factor -Shear: | 0.610 | | See Hand Calcs |
| Distr Factor -Moment: | 0.427 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.286 | | See Hand Calcs |
| Slab Thickness: | 8.50 | in | Per plans |
| Haunch: | | in | |
| Bridge DL1: | 0.000 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.725 | kip/ft | See Hand Calcs |
| F'c: | 3.0 | ksi | Per plans |
| N: | 9 | | Class A per DM-4 8.2 |
| Symmetry: | | | |
| LL Location: | | | Only applies to truss |
| Number of Panels: | | | Only applies to truss |
| End Connections: | | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | | | Only applies to truss |
| Stringer DL: | | kip/ft | N/A for CTB |
| Floorbeam DL: | | kip/ft | N/A for CTB |
| Unit weight of Concrete: | | lb/cf | N/A for steel grid |
| Gusset Plate Analysis: | | | N/A for CTB |
| Patch Load Analysis: | | | N/A for CTB |
| Unsymmetrical Pier Support: | | | N/A for CTB |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 29.50 | ft | See Hand Calcs |

**Concrete Member Properties**

| | | | |
|---|---|---|---|
| Type: | T | | T for T-beam |
| Depth: | 26.00 | in | Per Contract Drawings |
| B: | 15.75 | in | See Hand Calcs |
| D: | 21.51 | in | See Hand Calcs |
| AS: | 8.86 | $in^2$ | See Hand Calcs |
| D': | 0 | in | |
| A'S: | 0 | $in^2$ | |
| Fy Reinf: | 33 | ksi | See Hand Calcs |
| Allowable Fs - IR: | | ksi | |
| Allowable Fs - OR: | | ksi | |
| St. Det: | Y | | Stirrup Details |
| Av: | 2.53 | $in^2$ | See Hand Calcs |
| Specs: | 3 | | For 1973 or earlier AASHTO Specs |
| Alpha: | | | |
| IWS: | 0 | in | |

**Stirrup Details**

| | | | |
|---|---|---|---|
| Area: | 0.200 | $in^2$ | Per Contract Drawings |
| fsy: | 33 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 17.000 | in | Per Contract Drawings |
| Location 2: | 1.42 | ft | Per Contract Drawings |
| Spacing 2: | 5.000 | in | Per Contract Drawings |
| Location 3: | 2.67 | ft | Per Contract Drawings |
| Spacing 3: | 6.000 | in | Per Contract Drawings |
| Location 4: | 4.17 | ft | Per Contract Drawings |
| Spacing 4: | 7.000 | in | Per Contract Drawings |
| Location 5: | 5.33 | ft | Per Contract Drawings |
| Spacing 5: | 8.000 | in | Per Contract Drawings |
| Location 6: | 6.67 | ft | Per Contract Drawings |
| Spacing 6: | 16.000 | in | Per Contract Drawings |

### 3.2.8.2.2 BAR7 Output

```
*****************************************************************************
*                                                                           *
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*                                                                           *
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                    BRIDGE ANALYSIS AND RATING (BAR7)               330427

PROGRAM P4353000                                              06/04/2024 11:45
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: ... r T-Beam BAR7 Analysis - With Section Loss.dat




                            PROJECT IDENTIFICATION


STRUCTURE ID - 02307000800205 - INT BEAM WITH LOSS

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 CTB                0   0.00  0.0  0.0                           0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     0.000        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    5.13       0.00        0.00   0.610  0.427  0.286

   SLAB              DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   8.50      0.00   0.000   0.725   3.000    9.

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00


                         SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     29.50


            CONCRETE MEMBER PROPERTIES

                                             FY
TYPE  DEPTH    B      D    AS     D'   A'S   REINF
 T    26.00  15.75 21.51  8.86  0.00  0.00    33.


     ALLOWABLE FS    ST                             INTEGRAL
       IR    OR      DET   AV   SPECS  ALPHA  WEARING SURFACE
       0.0   0.0      Y   2.53    3      0.          0.0


                              STIRRUP DETAILS
```

| STIRRUP AREA | FSY | LOCATION | SPACING | LOCATION | SPACING | LOCATION | SPACING |
|---|---|---|---|---|---|---|---|
| 0.200 | 33 | 0.00 | 17.000 | 1.42 | 5.000 | 2.67 | 6.000 |
| | | 4.17 | 7.000 | 5.33 | 8.000 | 6.67 | 16.000 |

DEFAULT VALUES

| SLC LEVEL | GAGE DISTANCE | PASSING DISTANCE | UNIT WEIGHT DECK | FY REINF | ALLOWABLE FS IR | ALLOWABLE FS OR | ALPHA | INTEGRAL WEARING SURFACE | STIRRUP AREA | FSY | SKEW CORR FACTOR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| I | 6.0 | 4.0 | 150.0 | --- | 18.0 | 25.0 | 45. | --- | --- | --- | 1.000 |

CONCRETE SECTION PROPERTIES

| DEPTH OF N.A. | MOMENT OF INERTIA | SECTION MODULUS CONCRETE | SECTION MODULUS TENSION STEEL | SECTION MODULUS COMPRESSION STEEL |
|---|---|---|---|---|
| 6.28 | 23578.2 | 3753.81 | 172.03 | 0.00 |

LIVE LOAD IMPACT FACTOR FOR REACTION, MOMENT AND DEFLECTION : 1.30

```
*******************************
*  T-BEAM  -  LIVE LOAD  H20   *
*******************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 23.0 | 27.7 |
| 2 | 23.0 | 27.7 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 23.0 | 27.7 | 1.30 | 0.000 | 0.000 | 0.068 | 0.082 | 0.000 | 0.000 |
| 0.90 | 20.0 | 17.4 | 21.6 | 19.4 | 1.30 | 0.119 | 2.607 | 0.064 | 0.057 | 0.033 | 0.014 |
| 2.95 | 61.0 | 52.7 | 18.4 | 17.9 | 1.30 | 0.364 | 7.932 | 0.054 | 0.053 | 0.105 | 0.046 |
| 5.90 | 108.4 | 92.4 | 13.8 | 15.7 | 1.30 | 0.642 | 14.006 | 0.041 | 0.046 | 0.200 | 0.088 |
| 8.85 | 142.3 | 118.9 | 9.2 | 13.4 | 1.30 | 0.835 | 18.220 | 0.027 | 0.040 | 0.273 | 0.121 |
| 11.80 | 162.6 | 132.3 | 4.6 | 11.2 | 1.30 | 0.943 | 20.575 | 0.014 | 0.033 | 0.320 | 0.141 |
| 14.75 | 169.4 | 132.7 | -0.0 | 9.0 | 1.30 | 0.966 | 21.070 | -0.000 | 0.027 | 0.336 | 0.145 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | ALLOW. STRESS SHEAR | LOAD FACTOR MOMENT | LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 258.0 | 358.4 | 451.3 | 67.4 | 86.7 | 93.1 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.90 | 258.0 | 358.4 | 451.3 | 67.4 | 86.7 | 93.1 | IR | 13.68 | 2.36 | 11.28 | 1.54 |
| | | | | | | | OR | 19.44 | 3.35 | 18.80 | 2.57 |
| 2.95 | 258.0 | 358.4 | 451.3 | 56.3 | 74.6 | 71.8 | IR | 3.74 | 2.12 | 3.26 | 1.24 |
| | | | | | | | OR | 5.64 | 3.14 | 5.43 | 2.06 |
| 5.90 | 258.0 | 358.4 | 451.3 | 49.8 | 65.6 | 61.7 | IR | 1.62 | 2.30 | 1.55 | 1.29 |

```
                                                   OR   2.71   3.31    2.58   2.15
  8.85  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.97   2.31    1.03   1.19
                                                   OR   1.82   3.20    1.72   1.99
 11.80  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.72   3.17    0.84   1.67
                                                   OR   1.48   4.24    1.39   2.79
 14.75  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.67   4.47    0.80   2.39
                                                   OR   1.42   5.80    1.34   3.99


                       *******************************
                       *  T-BEAM  -  LIVE LOAD  HS20  *
                       *******************************

                                  MAXIMUM REACTIONS
                        SUPPORT    D.L.      LL+I
                           1      23.0       34.9
                           2      23.0       34.9


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
      ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
      SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

         D.L.   LL+I   D.L.    LL+I      FLEX STRESS SHEAR STRESS DEFLECTION
   X    MOMENT MOMENT SHEAR   SHEAR I.F.  CONC   STEEL  D.L.  LL+I  D.L.  LL+I
  0.00    0.0    0.0   23.0   34.9  1.30 0.000  0.000 0.068 0.103 0.000 0.000
  0.90   20.0   23.4   21.6   26.1  1.30 0.139  3.025 0.064 0.077 0.033 0.020
  2.95   61.0   69.5   18.4   23.5  1.30 0.417  9.099 0.054 0.069 0.105 0.064
  5.90  108.4  117.9   13.8   20.0  1.30 0.724 15.790 0.041 0.059 0.200 0.120
  8.85  142.3  145.5    9.2   16.4  1.30 0.920 20.074 0.027 0.049 0.273 0.163
 11.80  162.6  152.1    4.6   12.9  1.30 1.006 21.950 0.014 0.038 0.320 0.186
 14.75  169.4  139.3   -0.0    9.3  1.30 0.987 21.535-0.000 0.028 0.336 0.194

                                                           RATING FACTORS
          MOMENT CAPACITY    SHEAR CAPACITY     ALLOW. STRESS    LOAD FACTOR
   X     IR     OR     ULT    IR    OR    ULT     MOMENT  SHEAR  MOMENT  SHEAR
  0.00  258.0  358.4  451.3  67.4  86.7  93.1 IR   0.00           0.00
                                              OR   0.00           0.00
  0.90  258.0  358.4  451.3  67.4  86.7  93.1 IR  10.17   1.76    8.39   1.15
                                              OR  14.46   2.50   13.98   1.92
  2.95  258.0  358.4  451.3  56.3  74.6  71.8 IR   2.84   1.61    2.47   0.94
                                              OR   4.28   2.39    4.12   1.56
  5.90  258.0  358.4  451.3  49.8  65.6  61.7 IR   1.27   1.80    1.21   1.01
                                              OR   2.12   2.59    2.02   1.69
  8.85  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.80   1.88    0.84   0.97
                                              OR   1.49   2.61    1.41   1.62
 11.80  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.63   2.76    0.73   1.46
                                              OR   1.29   3.69    1.21   2.43
 14.75  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.64   4.30    0.77   2.31
                                              OR   1.36   5.59    1.28   3.84


                       *******************************
                       *  T-BEAM  -  LIVE LOAD TK527  *
                       *******************************

                                  MAXIMUM REACTIONS
                        SUPPORT    D.L.      LL+I
                           1      23.0       35.8
                           2      23.0       35.8


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
```

ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 23.0 | 35.8 | 1.30 | 0.000 | 0.000 | 0.068 | 0.106 | 0.000 | 0.000 |
| 0.90 | 20.0 | 26.6 | 21.6 | 29.7 | 1.30 | 0.149 | 3.248 | 0.064 | 0.088 | 0.033 | 0.024 |
| 2.95 | 61.0 | 79.1 | 18.4 | 26.8 | 1.30 | 0.448 | 9.772 | 0.054 | 0.079 | 0.105 | 0.076 |
| 5.90 | 108.4 | 133.9 | 13.8 | 22.7 | 1.30 | 0.775 | 16.904 | 0.041 | 0.067 | 0.200 | 0.145 |
| 8.85 | 142.3 | 168.1 | 9.2 | 18.6 | 1.30 | 0.992 | 21.653 | 0.027 | 0.055 | 0.273 | 0.199 |
| 11.80 | 162.6 | 190.8 | 4.6 | 14.8 | 1.30 | 1.130 | 24.655 | 0.014 | 0.044 | 0.320 | 0.230 |
| 14.75 | 169.4 | 192.1 | -0.0 | 11.4 | 1.30 | 1.156 | 25.217 | -0.000 | 0.034 | 0.336 | 0.239 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 258.0 | 358.4 | 451.3 | 67.4 | 86.7 | 93.1 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.90 | 258.0 | 358.4 | 451.3 | 67.4 | 86.7 | 93.1 | IR | 8.95 | 1.55 | 7.38 | 1.01 |
| | | | | | | | OR | 12.72 | 2.19 | 12.30 | 1.68 |
| 2.95 | 258.0 | 358.4 | 451.3 | 56.3 | 74.6 | 71.8 | IR | 2.49 | 1.41 | 2.17 | 0.82 |
| | | | | | | | OR | 3.76 | 2.10 | 3.62 | 1.37 |
| 5.90 | 258.0 | 358.4 | 451.3 | 49.8 | 65.6 | 61.7 | IR | 1.12 | 1.59 | 1.07 | 0.89 |
| | | | | | | | OR | 1.87 | 2.28 | 1.78 | 1.48 |
| 8.85 | 258.0 | 358.4 | 451.3 | 40.2 | 52.2 | 46.6 | IR | 0.69 | 1.67 | 0.73 | 0.86 |
| | | | | | | | OR | 1.29 | 2.31 | 1.22 | 1.44 |
| 11.80 | 258.0 | 358.4 | 451.3 | 40.2 | 52.2 | 46.6 | IR | 0.50 | 2.40 | 0.58 | 1.27 |
| | | | | | | | OR | 1.03 | 3.21 | 0.97 | 2.11 |
| 14.75 | 258.0 | 358.4 | 451.3 | 40.2 | 52.2 | 46.6 | IR | 0.46 | 3.54 | 0.56 | 1.89 |
| | | | | | | | OR | 0.98 | 4.59 | 0.93 | 3.16 |

```
********************************
*  T-BEAM  -  LIVE LOAD  ML80  *
********************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 23.0 | 37.5 |
| 2 | 23.0 | 37.5 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 23.0 | 37.5 | 1.30 | 0.000 | 0.000 | 0.068 | 0.111 | 0.000 | 0.000 |
| 0.90 | 20.0 | 28.1 | 21.6 | 31.3 | 1.30 | 0.154 | 3.352 | 0.064 | 0.093 | 0.033 | 0.026 |
| 2.95 | 61.0 | 83.9 | 18.4 | 28.4 | 1.30 | 0.463 | 10.103 | 0.054 | 0.084 | 0.105 | 0.082 |
| 5.90 | 108.4 | 143.0 | 13.8 | 24.2 | 1.30 | 0.804 | 17.536 | 0.041 | 0.072 | 0.200 | 0.157 |
| 8.85 | 142.3 | 181.9 | 9.2 | 20.0 | 1.30 | 1.036 | 22.616 | 0.027 | 0.059 | 0.273 | 0.215 |
| 11.80 | 162.6 | 208.4 | 4.6 | 15.9 | 1.30 | 1.186 | 25.879 | 0.014 | 0.047 | 0.320 | 0.251 |
| 14.75 | 169.4 | 210.1 | -0.0 | 12.5 | 1.30 | 1.213 | 26.472 | -0.000 | 0.037 | 0.336 | 0.261 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 258.0 | 358.4 | 451.3 | 67.4 | 86.7 | 93.1 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.90 | 258.0 | 358.4 | 451.3 | 67.4 | 86.7 | 93.1 | IR | 8.48 | 1.46 | 6.99 | 0.96 |
| | | | | | | | OR | 12.05 | 2.08 | 11.65 | 1.60 |

```
  2.95  258.0  358.4  451.3  56.3  74.6  71.8 IR   2.35   1.33    2.05   0.78
                                              OR   3.55   1.98    3.41   1.30
  5.90  258.0  358.4  451.3  49.8  65.6  61.7 IR   1.05   1.49    1.00   0.83
                                              OR   1.75   2.14    1.67   1.39
  8.85  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.64   1.55    0.68   0.80
                                              OR   1.19   2.14    1.13   1.33
 11.80  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.46   2.23    0.53   1.18
                                              OR   0.94   2.99    0.89   1.96
 14.75  258.0  358.4  451.3  40.2  52.2  46.6 IR   0.42   3.21    0.51   1.72
                                              OR   0.90   4.17    0.85   2.87


                ++++++++++++++++++++++++++++++++++++++++++++++
                +                                            +
                +        R A T I N G   S U M M A R Y         +
                +                                            +
                ++++++++++++++++++++++++++++++++++++++++++++++

                        ALLOWABLE STRESS RATING         LOAD FACTOR RATING
 LOAD                    FACTOR    TONS     X          FACTOR    TONS     X
 H20     IR (DESIGN)      0.67 M   13.4   14.75         0.80 M   16.1   14.75
         OR (DESIGN)      1.42 M   28.5   14.75         1.34 M   26.8   14.75
 HS20    IR (DESIGN)      0.63 M   22.6   11.80         0.73 M   26.2   11.80
         OR (DESIGN)      1.29 M   46.4   11.80         1.21 M   43.7   11.80
TK527    IR (DESIGN)      0.46 M   18.5   14.75         0.56 M   22.2   14.75
         OR (DESIGN)      0.98 M   39.4   14.75         0.93 M   37.0   14.75
 ML80    IR (DESIGN)      0.42 M   15.5   14.75         0.51 M   18.6   14.75
         OR (DESIGN)      0.90 M   33.0   14.75         0.85 M   31.0   14.75

      RATING FACTOR CODES:
      M - MAXIMUM MOMENT STRENGTH GOVERNS
      V - MAXIMUM SHEAR STRENGTH GOVERNS

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

3.2.8.3 Exterior Beam Load Rating Analysis

3.2.8.3.1 BAR7 Input Parameters

The exterior T-beam was selected and will be rated using BAR7. Many of the BAR7 input parameters can be left blank. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02307000800205
- Description = EXTERIOR BEAM
- Bridge Type = CTB
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 0 for Normal Output
- Skew Correction Factor = 1.084 (See below)

  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

$$\text{SCF} = 1.0 + 0.2\left(\frac{12Lt_s^3}{K_g}\right)^{0.3} \tan\,\theta$$

where,

L = Span Length = 29.50 ft
$t_s$ = Deck Thickness = 8.5 in
θ = AASHTO Skew Angle = (90° – 60° 00' 00") = 30.0° (skew angle at obtuse corner of bridge)

$e_g$ = Distance between the c.g. of the deck and c.g. of the beam = 8.75 in + 4.25 in = 13.0 in

Beam c.g. from Top of Beam = 17.5 in / 2 = 8.75 in
Deck c.g. from Top of Beam = 8.5 in / 2 = 4.25 in

n = Modular Ratio Between the Deck and Beam = 9 per DM-4 5.4.2.1 for $f'_c$ = 3 ksi (Class A Concrete per S-199)

I = Moment of Inertia of the Beam ≈ bh3/12 = 20.625 in x (17.5 in)$^3$ / 12 = 9211.4 in$^4$ (use stem only)

A = Stem Area ≈ b x h = 20.625 in x 17.5 in = 360.94 in$^2$ (Use stem only for non-composite beam per AASHTO 4.6.2.2.1)

$K_g$ = Longitudinal Stiffness Parameter = n(I+A$e_g^2$) = 9[9211.4 in$^4$ + 360.94 in$^2$ x (13.0 in)$^2$] = 631892 in$^4$

The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

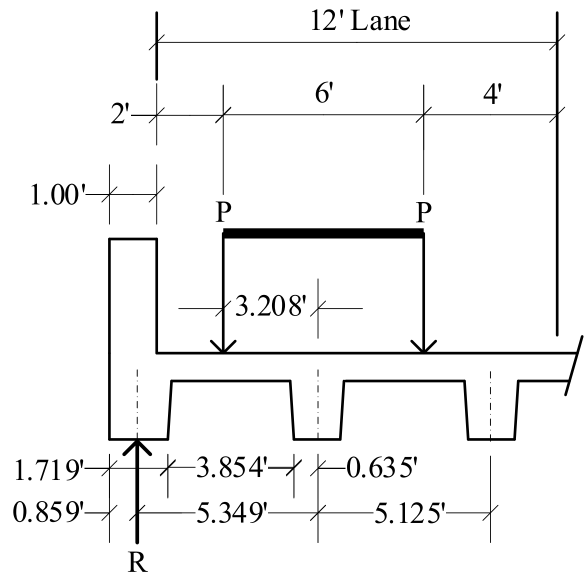
| | |
|---|---|
| 0° ≤ θ ≤ 60° | θ = 30.0°, OK |
| 3.5 ft ≤ S ≤ 60 ft | S = 5.349 ft (see below), OK |
| 20 t ≤ L ≤ 500 ft | L = 29.50 ft, OK |
| $N_b$ ≥ 4 | $N_b$ = 7, OK |

Bridge Cross Section and Loading

- Overhang/Spacing = 3' – 10 ¼" (Dimension E per S-515) / 2 + 1' – 8 5/8" (Dimension D per S-515) = 3.65 ft

- Distribution Factor – Shear

  Per AASHTO 3.23.1, the interior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the one-lane condition shown below, the reduction in load intensity factor is 1.0.




  $DF_V$ = P[3.208'/5.349'] x 1.0 = 0.600 wheels
  $DF_V$ = 0.600 wheels / (2 wheels/axle) = 0.300 axles

- Distribution Factor – Moment

  Per AASHTO 3.23.2.3.1.2, the exterior beam distribution factor for moment is based on the Lever Rule. The moment distribution factor is the same as calculated for shear.

  $DF_M$ = 0.300 axles

- Distribution Factor – Deflection
  Per AASHTO 8.13.2, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
    Roadway Width = 30.00 ft per Standard Drawing S-515
    $N_L$ = 30.00 ft / 12 ft = 2.50 ==> Use 2 lanes
  Reduction Factor = 1.0 for 2 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 2 lanes x 1.0 / 7 beams = 0.286 lanes/beam

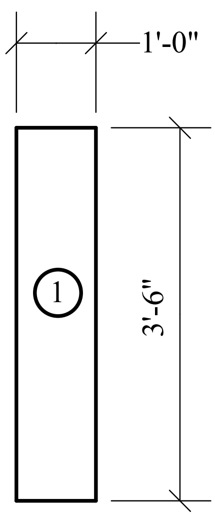
- Deck Thickness = 8.5 in

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight and deck slab weight which are calculated by BAR7. Weight due to SIP forms, haunch concrete, intermediate diaphragms, and stiffeners etc are equal to zero.

  Total Dead Load 1 = <u>0 kips/ft</u>

- Dead Loads – DL2
  The dead loads acting on the composite section include the barriers and asphalt wearing surface. Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  <u>Roadway Barriers</u>




  Area 1 = 12 in x 42 in = 504 in$^2$

  Barrier Weight = 504 in$^2$ / 144 in$^2$/ft$^2$ x 0.150 kcf = 0.525 kips/ft

  <u>Asphalt Wearing Surface</u>
  Per S-199 and S-515 Dimension T, the asphalt wearing surface is 2 ½” thick. However, a recent inspection indicates that the total wearing surface is 11 ½”.
  Overlay Weight = (11.5 in /12 in/ft) x 30 ft x 0.140 kcf = 4.025 kips/ft

  Total Dead Load 2 = (2 x 0.525 kips/ft + 4.025 kips/ft) / 7 beams = <u>0.725 kips/ft</u>

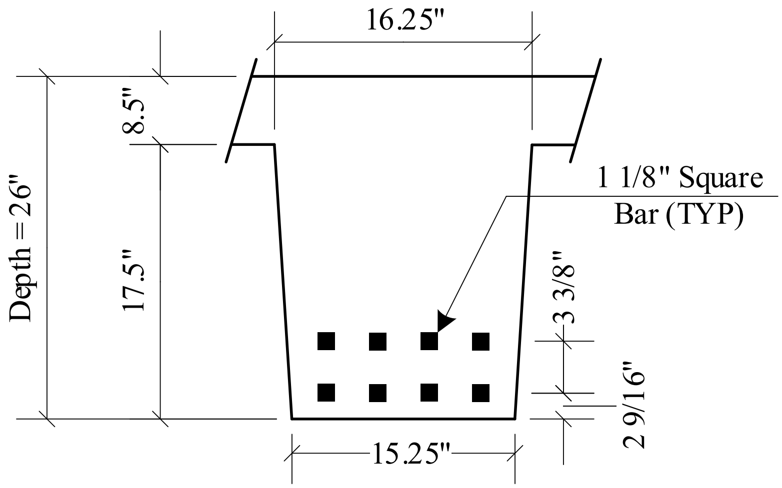
- Concrete Compressive Strength, $f'_c$ = 3 ksi for Class A Concrete per Table IP 3.7.2.2-1.
- Modular Ratio, n = 9 for Class A Concrete per DM-4 8.2

<u>Span Lengths</u>

- Continuity = S for Simple Span
- Span 1 Length = 28.0 ft + 2 x (9 in/12 in /ft) = 29.50 ft per Standard Drawings S-199

Concrete Member Properties

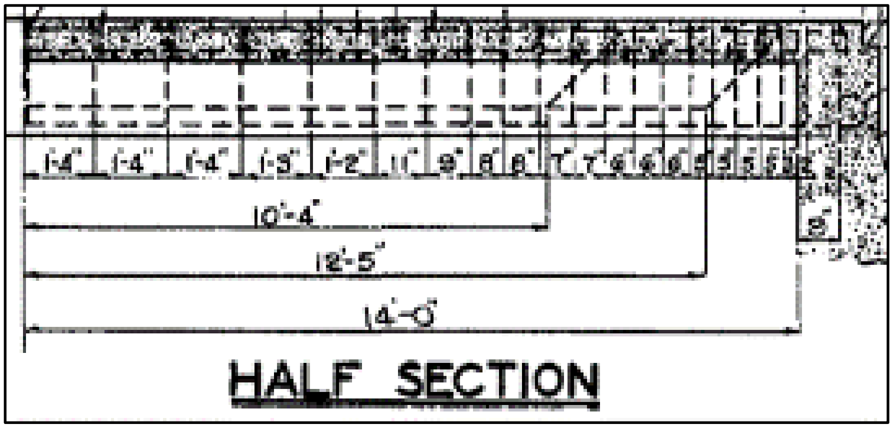
- Type = T for T-beam Bridge
- Depth from top of slab to bottom of web, DEPTH = Dimension B per S-515 + Slab Depth per S-199 = 17.5 in + 8.5 in = 26 in
- Width of the Web, B = 1'-8 5/8" for Dimension D per S-515 = 20.63 in
- Distance of the tension reinforcement (AS) from the top of section, D = 26 in – 4.25 in = 21.75 in
    - Distance to 1st Row (4 "a" bars) = 2.5625 in per S-199
    - Distance to 2nd Row (2 "b" & 2 "c" bars) = 2.5625 in + 3.375 in = 5.9375 in per S-199
    - All bars are 1 1/8" square bars, Bar Area = 1.125 in x 1.125 in = 1.266 in$^2$
    - Total Reinforcement Area = 8 x 1.266 in$^2$ = 10.125 in$^2$
    - Bar Group CG = [4 x 1.266 in$^2$ x (2.5625 in + 5.9375 in)] / 10.125 in$^2$ = 4.25 in




- Area of tension reinforcement, AS = 10.13 in$^2$
- Compression reinforcement not considered. D' = 0, A'S= 0
- Reinforcement yield strength, $f_y$ = 33 ksi per AASHTO MBE Table 6B.5.2.3-1
- Stirrup details = Y indicating stirrup reinforcement will be entered
- Stirrup area of bent up bars, AV = 2 "b" bars per S-199 = 2 x 1.125 in x 1.125 in = 2.53 in$^2$
- Specs = 3 for the program to use the 1973 or earlier AASHTO Specifications for shear ratings of a reinforced concrete member
- Alpha = Blank to default to 45 degrees.
- Integral wearing surface = 0 in.

Stirrup Details

The sketch below is taken from PennDOT's S-199 Standard Drawing.




- Stirrup Area = 0.20 in$^2$ for one leg of a #4 (½" diameter) stirrup
- Yield Strength of Reinforcement Stirrups, $f_y$ = 33 ksi per AASHTO MBE Table 6B.5.2.3-1
- Location = 0.00 ft
- Spacing = 3.0 in
- Location = 9 in + 3 in =1.00 ft per S-199
- Spacing = 5.0 in
- Location = 1 ft + 4 x 5 in/12 = 2.67 ft per S-199
- Spacing = 6.0 in
- Location = 2.67 ft + 3 x 6 in/12 = 4.17 ft per S-199
- Spacing = 7.0 in
- Location = 4.17 ft + 2 x 7 in/12 = 5.33 ft per S-199
- Spacing = 8.0 in
- Location = 5.33 ft + 2 x 8 in/12 = 6.67 ft per S-199
- Spacing = 16.0 in maximum spacing to centerline of span due to limitations of BAR7 locations

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 02-3070-0080-0205 | | |
| Description: | | | |
| Bridge Type: | CTB | | Concrete T-Beam |
| SLC Level: | | | Super = 5 & Sub = 6; ADTT = 23 therefore SLC = 1.0 |
| Lanes: | | | N/A for CTB |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 0 | | |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | | | |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | | | Leave blank and calculate DF |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | 1.084 | | See Hand Calcs |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for CTB |
| Overhang or Spacing: | 3.65 | ft | |
| CL of Girder: | -- | ft | Leave blank for CTB |
| Roadway Width: | -- | ft | Leave blank for CTB |
| Distr Factor -Shear: | 0.300 | | See Hand Calcs |
| Distr Factor -Moment: | 0.300 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.286 | | See Hand Calcs |
| Slab Thickness: | 8.50 | in | Per plans |
| Haunch: | 0.00 | in | |
| Bridge DL1: | 0.000 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.725 | kip/ft | See Hand Calcs |
| F'c: | 3.0 | ksi | Per plans |
| N: | 9 | | Class A per DM-4 8.2 |
| Symmetry: | | | |
| LL Location: | | | Only applies to truss |
| Number of Panels: | | | Only applies to truss |
| End Connections: | | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | | | Only applies to truss |
| Stringer DL: | | kip/ft | N/A for CTB |
| Floorbeam DL: | | kip/ft | N/A for CTB |
| Unit weight of Concrete: | | lb/cf | N/A for steel grid |
| Gusset Plate Analysis: | | | N/A for CTB |
| Patch Load Analysis: | | | N/A for CTB |
| Unsymmetrical Pier Support: | | | N/A for CTB |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 29.50 | ft | See Hand Calcs |

**Concrete Member Properties**

| | | | |
|---|---|---|---|
| Type: | T | | T for T-beam |
| Depth: | 26.00 | in | Per Contract Drawings |
| B: | 20.63 | in | See Hand Calcs |
| D: | 21.75 | in | See Hand Calcs |
| AS: | 10.13 | $in^2$ | See Hand Calcs |
| D': | 0 | in | |
| A'S: | 0 | $in^2$ | |
| Fy Reinf: | 33 | ksi | See Hand Calcs |
| Allowable Fs - IR: | | ksi | |
| Allowable Fs - OR: | | ksi | |
| St. Det: | Y | | Stirrup Details |
| Av: | 2.53 | $in^2$ | See Hand Calcs |
| Specs: | 3 | | For 1973 or earlier AASHTO Specs |
| Alpha: | | | |
| IWS: | 0 | in | |

**Stirrup Details**

| | | | |
|---|---|---|---|
| Area: | 0.200 | $in^2$ | Per Contract Drawings |
| fsy: | 33 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 3.000 | in | Per Contract Drawings |
| Location 2: | 1.00 | ft | Per Contract Drawings |
| Spacing 2: | 5.000 | in | Per Contract Drawings |
| Location 3: | 2.67 | ft | Per Contract Drawings |
| Spacing 3: | 6.000 | in | Per Contract Drawings |
| Location 4: | 4.17 | ft | Per Contract Drawings |
| Spacing 4: | 7.000 | in | Per Contract Drawings |
| Location 5: | 5.33 | ft | Per Contract Drawings |
| Spacing 5: | 8.000 | in | Per Contract Drawings |
| Location 6: | 6.67 | ft | Per Contract Drawings |
| Spacing 6: | 16.000 | in | Per Contract Drawings |

3.2.8.3.2 BAR7 Output

```
***************************************************************************
*                                                                         *
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*                                                                         *
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PROGRAM P4353000                                                06/05/2024 17:02
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: Exterior T-Beam BAR7 Analysis.dat




                             PROJECT IDENTIFICATION


STRUCTURE ID - 02307000800205 - EXTERIOR BEAM

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 CTB                  0   0.00  0.0  0.0                          0.00

    SKEW
    CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
    1.084        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    3.65       0.00        0.00   0.300  0.300  0.286

   SLAB              DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   8.50      0.00   0.000   0.275   3.000    9.

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00


                         SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     29.50


            CONCRETE MEMBER PROPERTIES

                                             FY
TYPE  DEPTH    B      D    AS     D'   A'S   REINF
 T    26.00  20.63 21.75 10.13  0.00  0.00   33.


     ALLOWABLE FS    ST                          INTEGRAL
       IR    OR      DET   AV   SPECS  ALPHA  WEARING SURFACE
       0.0   0.0      Y   2.53    3      0.          0.0


                               STIRRUP DETAILS
```

```
  STIRRUP
   AREA     FSY    LOCATION  SPACING    LOCATION  SPACING    LOCATION  SPACING
   0.200     33      0.00     3.000       1.00     5.000       2.67     6.000
                     4.17     7.000       5.33     8.000       6.67    16.000


                                            DEFAULT VALUES

                         UNIT                                   INTEGRAL
 SLC    GAGE   PASSING  WEIGHT   FY    ALLOWABLE FS             WEARING  STIRRUP      SKEW
CORR
LEVEL DISTANCE DISTANCE  DECK   REINF   IR     OR    ALPHA SURFACE   AREA   FSY
FACTOR
  I      6.0      4.0    150.0   ---   18.0   25.0   45.    ---      ---   ---    ----
-


          CONCRETE SECTION PROPERTIES

                                   SECTION     SECTION
DEPTH   MOMENT    SECTION    MODULUS     MODULUS
  OF      OF      MODULUS    TENSION   COMPRESSION
 N.A.   INERTIA   CONCRETE    STEEL       STEEL
 7.66   24661.9   3219.97    194.47        0.00


LIVE LOAD IMPACT FACTOR FOR REACTION, MOMENT AND DEFLECTION : 1.30


                        ********************************
                        *  T-BEAM  -  LIVE LOAD  H20   *
                        ********************************

                                   MAXIMUM REACTIONS
                         SUPPORT    D.L.      LL+I
                            1      15.3       14.1
                            2      15.3       14.1


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
      ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
      SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

          D.L.  LL+I   D.L.   LL+I      FLEX STRESS SHEAR STRESS DEFLECTION
   X    MOMENT MOMENT SHEAR  SHEAR I.F.  CONC   STEEL  D.L.  LL+I  D.L.  LL+I
  0.00    0.0    0.0   15.3  15.3  1.30 0.000  0.000 0.034 0.034 0.000 0.000
  0.91   13.5   12.4   14.4  14.8  1.30 0.096  1.593 0.032 0.033 0.021 0.014
  2.95   40.7   37.1   12.3  13.6  1.30 0.290  4.797 0.027 0.030 0.067 0.044
  5.90   72.3   64.9    9.2  11.9  1.30 0.511  8.468 0.020 0.027 0.127 0.084
  8.85   94.9   83.5    6.1  10.2  1.30 0.665 11.013 0.014 0.023 0.174 0.115
 11.80  108.5   93.0    3.1   8.5  1.30 0.751 12.432 0.007 0.019 0.204 0.134
 14.75  113.0   93.2   -0.0   6.9  1.30 0.769 12.725-0.000 0.015 0.214 0.139

                                                       RATING FACTORS
           MOMENT CAPACITY    SHEAR CAPACITY     ALLOW. STRESS   LOAD FACTOR
   X     IR      OR      ULT     IR     OR    ULT      MOMENT  SHEAR  MOMENT  SHEAR
  0.00   291.7   405.1   507.8 124.8 168.5 173.3 IR    0.00           0.00
                                                 OR    0.00           0.00
  0.91   291.7   405.1   507.8 124.8 168.5 173.3 IR   22.51    7.47   18.31   4.83
                                                 OR   31.69   10.42   30.51   8.04
  2.95   291.7   405.1   507.8  66.5  87.5  82.5 IR    6.78    3.98    5.67   2.25
                                                 OR    9.84    5.53    9.44   3.76
  5.90   291.7   405.1   507.8  60.0  78.5  72.3 IR    3.38    4.26    2.94   2.34
```

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | OR | 5.13 | 5.81 | 4.90 | 3.89 |
| 8.85 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 2.36 | 4.30 | 2.12 | 2.21 |
| | | | | | | | OR | 3.71 | 5.74 | 3.54 | 3.69 |
| 11.80 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.97 | 5.52 | 1.82 | 2.87 |
| | | | | | | | OR | 3.19 | 7.24 | 3.03 | 4.78 |
| 14.75 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.92 | 7.32 | 1.79 | 3.84 |
| | | | | | | | OR | 3.13 | 9.47 | 2.98 | 6.40 |

```
*******************************
*  T-BEAM  -  LIVE LOAD  HS20  *
*******************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 15.3 | 19.2 |
| 2 | 15.3 | 19.2 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 15.3 | 20.8 | 1.30 | 0.000 | 0.000 | 0.034 | 0.046 | 0.000 | 0.000 |
| 0.91 | 13.5 | 16.6 | 14.4 | 19.9 | 1.30 | 0.112 | 1.856 | 0.032 | 0.044 | 0.021 | 0.019 |
| 2.95 | 40.7 | 48.8 | 12.3 | 17.9 | 1.30 | 0.333 | 5.522 | 0.027 | 0.040 | 0.067 | 0.061 |
| 5.90 | 72.3 | 82.9 | 9.2 | 15.2 | 1.30 | 0.578 | 9.577 | 0.020 | 0.034 | 0.127 | 0.115 |
| 8.85 | 94.9 | 102.2 | 6.1 | 12.5 | 1.30 | 0.735 | 12.165 | 0.014 | 0.028 | 0.174 | 0.156 |
| 11.80 | 108.5 | 106.8 | 3.1 | 9.8 | 1.30 | 0.802 | 13.287 | 0.007 | 0.022 | 0.204 | 0.178 |
| 14.75 | 113.0 | 97.9 | -0.0 | 7.1 | 1.30 | 0.786 | 13.014 | -0.000 | 0.016 | 0.214 | 0.185 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 291.7 | 405.1 | 507.8 | 124.8 | 168.5 | 173.3 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.91 | 291.7 | 405.1 | 507.8 | 124.8 | 168.5 | 173.3 | IR | 16.75 | 5.56 | 13.62 | 3.59 |
| | | | | | | | OR | 23.57 | 7.76 | 22.70 | 5.98 |
| 2.95 | 291.7 | 405.1 | 507.8 | 66.5 | 87.5 | 82.5 | IR | 5.14 | 3.02 | 4.30 | 1.71 |
| | | | | | | | OR | 7.47 | 4.20 | 7.17 | 2.85 |
| 5.90 | 291.7 | 405.1 | 507.8 | 60.0 | 78.5 | 72.3 | IR | 2.65 | 3.33 | 2.30 | 1.83 |
| | | | | | | | OR | 4.02 | 4.55 | 3.84 | 3.05 |
| 8.85 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.93 | 3.52 | 1.74 | 1.81 |
| | | | | | | | OR | 3.03 | 4.69 | 2.89 | 3.01 |
| 11.80 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.71 | 4.80 | 1.58 | 2.49 |
| | | | | | | | OR | 2.78 | 6.30 | 2.64 | 4.16 |
| 14.75 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.83 | 7.06 | 1.70 | 3.70 |
| | | | | | | | OR | 2.98 | 9.13 | 2.84 | 6.17 |

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*******************************
*  T-BEAM  -  LIVE LOAD TK527  *
*******************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 15.3 | 21.7 |
| 2 | 15.3 | 21.7 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD

ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 15.3 | 23.6 | 1.30 | 0.000 | 0.000 | 0.034 | 0.053 | 0.000 | 0.000 |
| 0.91 | 13.5 | 18.9 | 14.4 | 22.6 | 1.30 | 0.121 | 1.996 | 0.032 | 0.050 | 0.021 | 0.023 |
| 2.95 | 40.7 | 55.6 | 12.3 | 20.4 | 1.30 | 0.359 | 5.940 | 0.027 | 0.046 | 0.067 | 0.073 |
| 5.90 | 72.3 | 94.1 | 9.2 | 17.3 | 1.30 | 0.620 | 10.269 | 0.020 | 0.039 | 0.127 | 0.139 |
| 8.85 | 94.9 | 118.1 | 6.1 | 14.2 | 1.30 | 0.794 | 13.146 | 0.014 | 0.032 | 0.174 | 0.190 |
| 11.80 | 108.5 | 134.1 | 3.1 | 11.3 | 1.30 | 0.904 | 14.968 | 0.007 | 0.025 | 0.204 | 0.220 |
| 14.75 | 113.0 | 135.0 | -0.0 | 8.6 | 1.30 | 0.924 | 15.302 | -0.000 | 0.019 | 0.214 | 0.228 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 291.7 | 405.1 | 507.8 | 124.8 | 168.5 | 173.3 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.91 | 291.7 | 405.1 | 507.8 | 124.8 | 168.5 | 173.3 | IR | 14.73 | 4.89 | 11.98 | 3.16 |
| | | | | | | | OR | 20.73 | 6.82 | 19.97 | 5.26 |
| 2.95 | 291.7 | 405.1 | 507.8 | 66.5 | 87.5 | 82.5 | IR | 4.52 | 2.66 | 3.78 | 1.50 |
| | | | | | | | OR | 6.56 | 3.69 | 6.30 | 2.51 |
| 5.90 | 291.7 | 405.1 | 507.8 | 60.0 | 78.5 | 72.3 | IR | 2.33 | 2.94 | 2.03 | 1.61 |
| | | | | | | | OR | 3.54 | 4.01 | 3.38 | 2.68 |
| 8.85 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.67 | 3.11 | 1.50 | 1.60 |
| | | | | | | | OR | 2.63 | 4.15 | 2.50 | 2.67 |
| 11.80 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.37 | 4.17 | 1.26 | 2.17 |
| | | | | | | | OR | 2.21 | 5.48 | 2.10 | 3.61 |
| 14.75 | 291.7 | 405.1 | 507.8 | 50.2 | 64.9 | 57.0 | IR | 1.32 | 5.80 | 1.23 | 3.04 |
| | | | | | | | OR | 2.16 | 7.50 | 2.06 | 5.07 |

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********************************
*  T-BEAM  -  LIVE LOAD  ML80  *
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MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 15.3 | 22.9 |
| 2 | 15.3 | 22.9 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 15.3 | 24.8 | 1.30 | 0.000 | 0.000 | 0.034 | 0.055 | 0.000 | 0.000 |
| 0.91 | 13.5 | 19.9 | 14.4 | 23.9 | 1.30 | 0.124 | 2.061 | 0.032 | 0.053 | 0.021 | 0.025 |
| 2.95 | 40.7 | 58.9 | 12.3 | 21.6 | 1.30 | 0.371 | 6.146 | 0.027 | 0.048 | 0.067 | 0.079 |
| 5.90 | 72.3 | 100.5 | 9.2 | 18.5 | 1.30 | 0.644 | 10.662 | 0.020 | 0.041 | 0.127 | 0.150 |
| 8.85 | 94.9 | 127.8 | 6.1 | 15.3 | 1.30 | 0.830 | 13.745 | 0.014 | 0.034 | 0.174 | 0.206 |
| 11.80 | 108.5 | 146.4 | 3.1 | 12.1 | 1.30 | 0.950 | 15.729 | 0.007 | 0.027 | 0.204 | 0.240 |
| 14.75 | 113.0 | 147.6 | -0.0 | 9.5 | 1.30 | 0.971 | 16.083 | -0.000 | 0.021 | 0.214 | 0.249 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 291.7 | 405.1 | 507.8 | 124.8 | 168.5 | 173.3 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.91 | 291.7 | 405.1 | 507.8 | 124.8 | 168.5 | 173.3 | IR | 13.95 | 4.63 | 11.35 | 2.99 |
| | | | | | | | OR | 19.64 | 6.46 | 18.91 | 4.98 |

```
  2.95  291.7  405.1  507.8  66.5  87.5  82.5 IR   4.26   2.50    3.56   1.42
                                              OR   6.19   3.48    5.94   2.36
  5.90  291.7  405.1  507.8  60.0  78.5  72.3 IR   2.18   2.75    1.90   1.51
                                              OR   3.31   3.75    3.17   2.51
  8.85  291.7  405.1  507.8  50.2  64.9  57.0 IR   1.54   2.89    1.39   1.48
                                              OR   2.43   3.85    2.31   2.47
 11.80  291.7  405.1  507.8  50.2  64.9  57.0 IR   1.25   3.88    1.16   2.02
                                              OR   2.03   5.09    1.93   3.36
 14.75  291.7  405.1  507.8  50.2  64.9  57.0 IR   1.21   5.27    1.13   2.76
                                              OR   1.98   6.81    1.88   4.61


                ++++++++++++++++++++++++++++++++++++++++++++++
                +                                            +
                +        R A T I N G   S U M M A R Y         +
                +                                            +
                ++++++++++++++++++++++++++++++++++++++++++++++

                        ALLOWABLE STRESS RATING         LOAD FACTOR RATING
  LOAD                    FACTOR    TONS      X          FACTOR    TONS      X
  H20     IR (DESIGN)      1.92 M   38.3   14.75          1.79 M   35.7   14.75
          OR (DESIGN)      3.13 M   62.7   14.75          2.98 M   59.6   14.75
  HS20    IR (DESIGN)      1.71 M   61.7   11.80          1.58 M   57.0   11.80
          OR (DESIGN)      2.78 M  100.0   11.80          2.64 M   95.1   11.80
 TK527    IR (DESIGN)      1.32 M   53.0   14.75          1.23 M   49.4   14.75
          OR (DESIGN)      2.16 M   86.6   14.75          2.06 M   82.3   14.75
  ML80    IR (DESIGN)      1.21 M   44.4   14.75          1.13 M   41.3   14.75
          OR (DESIGN)      1.98 M   72.5   14.75          1.88 M   68.9   14.75

      RATING FACTOR CODES:
      M - MAXIMUM MOMENT STRENGTH GOVERNS
      V - MAXIMUM SHEAR STRENGTH GOVERNS

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

## 3.3 PRECAST CONCRETE CHANNEL BEAMS

This Section covers the rating of precast reinforced concrete channel beam bridges.

### 3.3.1 Policies and Guidelines

Most channel beam bridges can be rated by analytical methods based on design plans and field measurements. If design plans are not available, PennDOT standard plans can be utilized if field measured values are in reasonable agreement with one of the standard plans.

If plans are not available and field measurements are not in reasonable agreement with PennDOT standard plans, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.3.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating.

#### 3.3.2.1 LFR or ASR Method

Based on the age of these bridges, the typical load rating method will be LFR. PennDOT's BAR7 program does not have a specific Channel Beam bridge type. Therefore, the T-Beam (Bridge Type CTB) should be utilized to analyze and rate these bridges when the necessary information is available to complete an analytical rating.

#### 3.3.2.2 LRFR Method

Most channel beam bridges that were built prior to 2003 were designed by load factor or allowable stress methods; starting in 2003, however, these bridges were designed using LRFD and could be rated using LRFR. AASHTOWare Bridge Rating (BrR) or other approved software could be utilized. In addition, PennDOT's PSLRFD program can be utilized to rate the bridges built after 2011, which require the PHL-93 rating vehicle as the NBI vehicle. PennDOT is finalizing a procedure to develop the PHL-93 ratings and will provide in a future update to this manual.

### 3.3.3 Live Load and Dead Load Distribution

#### 3.3.3.1 LFR or ASR Method

Pub. 238 Section 3.3 provides guidance on the distribution of live load to longitudinal girders. Typically, the live load distribution factor for exterior beams is in accordance with the AASHTO Standard Specification.

The typical distribution factors are as follows when AASHTO distribution factors are applicable:

| Exterior | | Interior | |
|---|---|---|---|
| Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.4.3) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.4.3) |
| Lever Rule* | S/D | Lever Rule* | S/D |

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2.2.3 and Pub. 238 IP 3.3.3.1 for additional discussion of skew.

Dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all girders (AASHTO Standard Specification 3.23.2.3.2).

3.3.3.2 LRFR Method

For channel beam bridges built after 2011, the LRFR rating method would be utilized. See Section 3.3.9 for a procedure and sample that used PSLRFD to provide LRFD moment and shear ratings for the PHL-93 vehicle.

For channel beam bridges designed with LRFD, distribution factors may be computed by PSLRFD. For analysis outside of the program, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.2.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO distribution factors.

### 3.3.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| BC-700 Series, Jan. 1989 Edition | Precast Channel Beam Bridges | 1/20/1989 | BC-793 |
| BC-700 Series, Jan. 1993 Edition | Precast Channel Beam Bridges | 7/1/1993 | BC-793 |
| BC-700 Series, Jan. 1994 Edition | Precast Channel Beam Bridges | 9/30/1994 | BC-793 |
| "BC-793 PRECAST CHANNEL BEAM BRIDGES" DELETED 12/24/1999 | | | |
| Online document not available | Standard Precast Channel Beam Bridges | 1/21/2003 | BD-668M |
| Online document not available | Standard Precast Channel Beam Bridges | 9/20/2010 (Latest Revision 8/31/2012) | BD-668M |
| Standards for Bridge Design April 2016 Edition Change 7 | Standard Precast Channel Beam Bridges | 4/29/2016 (Latest Revision 10/7/2024) | BD-668M |

### 3.3.5 Modeling Section Properties and Deterioration

See Section 2.5.1.2 for a discussion of modeling section loss.

### 3.3.6 Standard Practices

For LFR and ASR ratings using BAR7, special care should be taken to modify the T-Beam input values to best emulate the actual geometry of the Channel Beams. For the vertical shear reinforcement of the interior beams, the T-Beam inputs only account for one leg of the stirrup. Since there will be two webs in this location, the vertical stirrup area should be doubled to properly account for both webs.

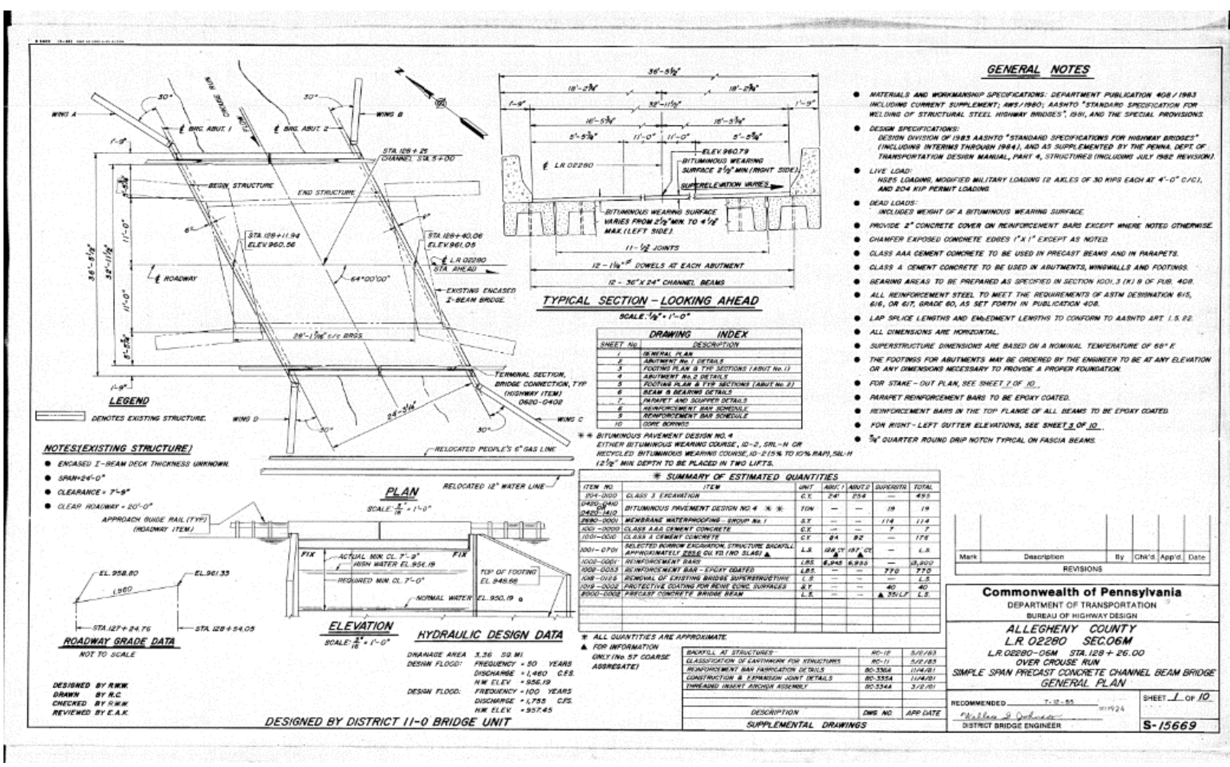
### 3.3.7 Common QA Findings

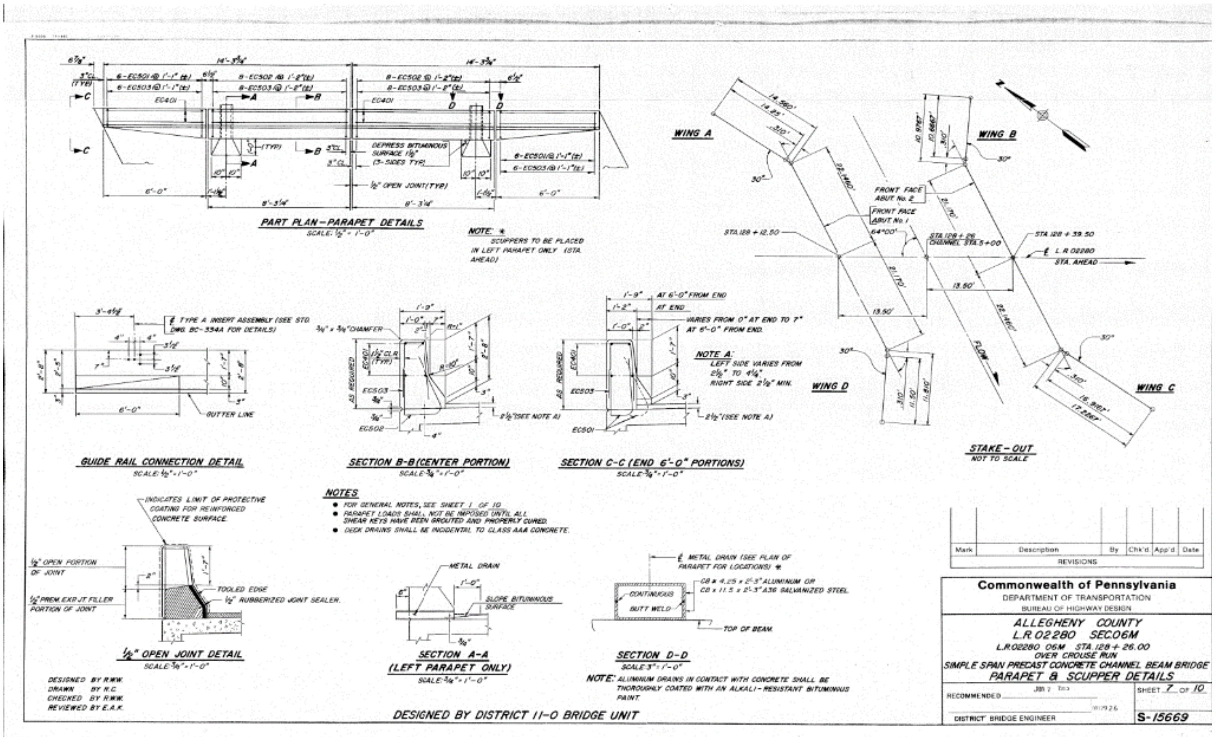
This Section is in development and will be provided in future editions of this manual.

### 3.3.8 Sample Load Rating

This Section contains sample load rating calculations for an interior and exterior beam for a single span non-composite channel beam structure shown below. The analysis will be performed using PennDOT's Bridge Analysis and Rating (BAR7) Program, Version 7.15.0.0 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part IV (DM-4).

The single span superstructure consists of a bituminous wearing surface supported on twelve (12) 36" x 24" reinforced concrete channel beams with ½" joint between the beams.




PART PLAN–PARAPET DETAILS
GUIDE RAIL CONNECTION DETAIL
SECTION B-B (CENTER PORTION)
SECTION C-C (END 6'-0" PORTIONS)
STAKE-OUT
NOT TO SCALE
WING A
WING B
WING C
WING D
1/2" OPEN JOINT DETAIL
SECTION A-A
(LEFT PARAPET ONLY)
SECTION D-D
DESIGNED BY DISTRICT 11-0 BRIDGE UNIT
Commonwealth of Pennsylvania
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAY DESIGN
ALLEGHENY COUNTY
L.R. 02280 SEC. 06M
OVER CROUSE RUN
SIMPLE SPAN PRECAST CONCRETE CHANNEL BEAM BRIDGE
PARAPET & SCUPPER DETAILS
S-15669

### 3.3.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

| | | | |
|---|---|---|---|
| Done By: | | Date: | |
| Checked By: | | Date: | |
| Structure ID (5A01): | 02-4070-0130-0537 | Inspection Date (7A01): | 6/15/2022 |
| Facility Carried (5A08): | Wildwood Road | | |
| Feature Intersected (5A07): | Wildwood Road Over Crouse Run | | |
| Structure Type (6A26 - 6A29): | 31108 - PC Channel Beam | | |
| Spans / Members Analyzed: | Span 1, Exterior Beam and 1st Interior Beam | | |
| Analysis Method: | LFD | | |
| PennDOT Program / Version: | BAR7 Version 7.15.0.0 | | |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 1.66 | 33.1 | 2.76 | 55.2 | 2.76 | 55.2 | * | * | M | M |
| HS20 | 1.49 | 53.7 | 2.48 | 89.4 | 2.48 | 89.4 | * | * | M | M |
| ML80 | 1.06 | 38.8 | 1.76 | 64.6 | 1.76 | 64.6 | * | * | M | M |
| TK527 | 1.15 | 46.2 | 1.92 | 76.9 | 1.92 | 76.9 | * | * | M | M |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 1.54 | 44.4 | 2.57 | 73.9 | 2.57 | 73.9 | * | * | M | M |
| EV3 | 1.00 | 42.9 | 1.66 | 71.5 | 1.66 | 71.5 | * | * | M | M |

Comments/Assumptions*:

Superstructure and substructure condition ratings are both 7. Therefore, for an ADTT < 500, the SLC Factor equals 1.0 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is the 1st Interior Beam 1 in Span 1.

BAR7 analysis does not account for any section loss but does include a 3" asphalt wearing surface.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

3.3.8.2 Interior Beam Load Rating Analysis

3.3.8.2.1 BAR7 Input Parameters

A typical interior channel beam was selected and will be rated using BAR7. Many of the BAR7 input parameters can be left blank. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02407001300537
- Description = INTERIOR BEAM
- Bridge Type = CTB (channel beam will be modeled as T-beam)
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 0 for Normal Output
- Concrete Deck = "Y" for top flange of channel modeled as a T-beam.
- Skew Correction Factor = 1.109 (See below). Per DM-4 Table 3.23.2(A), in determining end shear on multi-beam bridges, all beams shall be treated like the beam at the obtuse corner, i.e., the adjustment is appliable to all beams and shall be applied to the distribution factor for interior beams.

  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

$$\text{SCF} = 1.0 + \left(\frac{12L}{90d}\right)\sqrt{\tan\,\theta}$$

where,

Span Length, L = 28'-1 7/16" = 28.12 ft

Beam Depth, d = 24 in

AASHTO Skew Angle, θ = (90° – 64° 00' 00") = 26°

The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

| | |
|---|---|
| $0° \leq \theta \leq 60°$ | θ = 26.0°, OK |
| 20 ft ≤ L ≤ 120 ft | L = 28.12 ft, OK |
| 17 in ≤ d ≤ 60 in | d = 24 in, OK |
| 3.5 ft ≤ b ≤ 6 ft | b = 3 ft, NG…Call OK. |
| $5 \leq N_b \leq 20$ | $N_b$ = 12, OK |

Although the range of applicability limits are exceeded for several variables, apply the calculated skew correction factor as calculated. Code guidance when the limits are exceeded is not provided.

Bridge Cross Section and Loading

- Spacing = 3 ft + ½" Joint / 12 = 3.04 ft
- Roadway Width = 32' – 11 ½" = 32.96 ft

- Distribution Factor – Shear
  Per Pub 238 IE 6B.6.3, the interior beam distribution factor for moment and shear shall be equal to 1.0 where there is a loss of grout in the shear key and/or tie rod. This example assumes neither a tie rod or a shear key is present to allow sharing of load between the beams.

  $DF_V$ = 1.0 wheels x (1 axle/2 wheels) = <u>0.500 axles</u>

- Distribution Factor – Moment
  Per Pub 238 IE 6B.6.3, the interior beam distribution factor for moment and shear shall be equal to 1.0 where there is a loss of grout in the shear key and/or tie rod. This example assumes a tie rod is not present and there is not a shear key to allow sharing of load between the beams.

  $DF_M$ = 1.0 wheels x (1 axle/2 wheels) = <u>0.500 axles</u>.

- Distribution Factor – Deflection
  Per AASHTO 8.13.2, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
  Roadway Width = 32.96 ft per Contract Drawing
  $N_L$ = 32.96 ft / 12 ft = 2.75 ==> Use 2 lanes
  Reduction Factor = 1.0 for 2 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 2 lanes x 1.0 / 12 beams = 0.167 lanes/beam

- Slab Thickness = 6.5 in (The top flange of the channel is modeled as the deck slab in BAR7.)

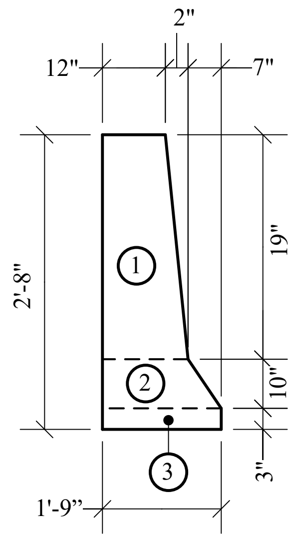
- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight and deck slab weight (i.e., top flange) which are calculated by BAR7. Weight due to SIP forms, haunch concrete, intermediate diaphragms, and stiffeners etc are equal to zero.

  Total Dead Load 1 = <u>0 kips/ft</u>

- Dead Loads – DL2

  The dead loads acting on the entire channel beam section include the barriers and asphalt wearing surface. Per IE 6B.6.1 for adjacent non-composite beams, assume the first interior beam supports 50% of the barrier dead load. This also applies to channel beams. Since the structure is non-composite, the wearing surface will be calculated based on a tributary width.

  Roadway Barriers




  Area 1 = (12 in + 14 in)/2 x 19 in = 247 in$^2$
  Area 2 = (14 in + 21 in)/2 x 10 in = 175 in$^2$
  Area 3 = 21 in x 3 in = 63 in$^2$
  Total Area = 485 in$^2$

  Barrier Weight = 485 in$^2$ / 144 in$^2$/ft$^2$ x 0.150 kcf = 0.505 kips/ft

  Asphalt Wearing Surface

  Based on field measurements, the asphalt wearing surface is 3” thick.
  Overlay Weight = (3 in /12 in/ft) x 3.042 ft x 0.140 kcf = 0.107 kips/ft

  Total Dead Load 2 = (0.50 x 0.505 kips/ft + 0.107 kips/ft) = 0.360 kips/ft

- Concrete Compressive Strength, $f'_c$ = 4.5 ksi for Class AAA Concrete per Table IP 3.7.2.2-1. Per DM-4 8.2, use $f'_c$ = 4.0 ksi for Class AAA Concrete unless otherwise approved.
- Modular Ratio, n = 8 for Class A Concrete per DM-4 8.2

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = 28’-1 7/16” = 28.12 ft

Concrete Member Properties

- Type = T for T-beam Bridge
- Depth from top of slab to bottom of web, Depth = 24 in
- Width of the Web, B = 2 x 9.77 in = 19.54 in

  Top of Web Width = 10.5 in
  Bottom of Web Width = 10.5 in – (24 in – 6.5 in) / 12 = 9.04 in
  Average Web Width = (10.5 in + 9.04 in) / 2 = 9.77 in

- Distance of the tension reinforcement (AS) from the top of section, D = 24 in – 4.03 in = 19.97 in
    - Distance to 1st Row (4 - #9 bars) = 1.5 in + 0.5 in + 1.128 in/2 = 2.56 in
    - Distance to 2nd Row (4 - #9 bars) = 5.50 in
    - Total Reinforcement Area = 8 x 1.00 $in^2$ = 8.0 $in^2$
    - Bar Group CG = [4.00 $in^2$ x (2.56 in + 5.50 in)] / 8.00 $in^2$ = 4.03 in

- Area of tension reinforcement, AS = 8.00 $in^2$
- Compression reinforcement not typically considered. D' = 0, A'S= 0
- Reinforcement yield strength, $f_y$ = 60 ksi per General Notes
- Stirrup details = Y indicating stirrup reinforcement will be entered
- Integral wearing surface = 0 in due to asphalt wearing surface.

Stirrup Details

- Stirrup Area = 0.40 $in^2$ for two-#4 (½" diameter) stirrup legs (BAR7 assumes 2 legs and will correctly double to equal 4 legs)
- Yield Strength of Reinforcement Stirrups, $f_y$ = 60 ksi per General Notes
- Location = 0.00 ft
- Spacing = 4.5 in
- Location = 2.53 ft
- Spacing = 10.0 in

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 02-4070-0130-0537 | | |
| Description: | INTERIOR BEAM | | |
| Bridge Type: | CTB | | Channel Beam |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 0 | | Normal Output |
| Concrete Deck: | Y | | See Hand Calcs |
| Skew Correction Factor: | 1.109 | | See Hand Calcs |

**Bridge Cross Section and Loading**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for CTB |
| Overhang or Spacing: | 3.04 | ft | See Hand Calcs |
| CL of Girder: | -- | ft | Leave blank for CTB |
| Roadway Width: | 32.96 | ft | See Hand Calcs |
| Distr Factor -Shear: | 0.500 | | See Hand Calcs |
| Distr Factor -Moment: | 0.500 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.167 | | See Hand Calcs |
| Slab Thickness: | 6.50 | in | See Hand Calcs |
| Haunch: | | in | |
| Bridge DL1: | 0.000 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.360 | kip/ft | See Hand Calcs |
| F'c: | 4.0 | ksi | Per plans |
| N: | 8 | | Class A per DM-4 8.2 |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple span |
| Span Length 1: | 28.12 | ft | Per plans |

**Concrete Member Properties**

| | | | |
|---|---|---|---|
| Type: | T | | T for T-Beam Bridge |
| Depth: | 24.00 | in | See Hand Calcs |
| B: | 19.54 | in | See Hand Calcs |
| D: | 19.97 | in | See Hand Calcs |
| AS: | 8.00 | | See Hand Calcs |
| $f_y$: | 60 | | Per Contract Drawings |
| Stirrup Details: | Y | | |

**Stirrup Details**

| | | | |
|---|---|---|---|
| Stirrup Area: | 0.400 | $in^2$ | Per Contract Drawings |
| $f_{sy}$: | 60 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 4.500 | in | Per Contract Drawings |
| Location 2: | 2.53 | ft | Per Contract Drawings |
| Spacing 2: | 10.000 | in | Per Contract Drawings |

3.3.8.2.2 BAR7 Output

```
******************************************************************************
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                     BRIDGE ANALYSIS AND RATING (BAR7)                330427

PROGRAM P4353000                                               06/14/2024 16:26
VERSION 7.15.0.0          LAST UPDATED 02/15/2018       DOCUMENTATION 02/2018

INPUT: Interior Channel Beam BAR7 Analysis.dat




                              PROJECT IDENTIFICATION


STRUCTURE ID - 02407001300537 - 1ST INT BEAM

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 CTB                 0   0.00  0.0  0.0        Y                 0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     1.109        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    3.04       0.00       32.96   0.500  0.500  0.167

   SLAB              DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   6.50      0.00   0.000   0.360   4.000    8.     Y

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00


                        SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     28.12


            CONCRETE MEMBER PROPERTIES

                                             FY
TYPE  DEPTH    B      D    AS     D'   A'S   REINF
 T    24.00  19.54 19.97  8.00  0.00  0.00   60.


     ALLOWABLE FS    ST                          INTEGRAL
       IR    OR      DET   AV   SPECS  ALPHA  WEARING SURFACE
       0.0   0.0      Y   0.00    0      0.         0.0


                              STIRRUP DETAILS
```

| STIRRUP AREA | FSY | LOCATION | SPACING | LOCATION | SPACING | LOCATION | SPACING |
|---|---|---|---|---|---|---|---|
| 0.400 | 60 | 0.00 | 4.500 | 2.53 | 10.000 | 0.00 | 0.000 |
| | | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 |

DEFAULT VALUES

| SLC LEVEL | GAGE DISTANCE | PASSING DISTANCE | UNIT WEIGHT DECK | FY REINF | ALLOWABLE FS IR | OR | ALPHA | INTEGRAL WEARING SURFACE | STIRRUP AREA | FSY | SKEW CORR FACTOR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| I | 6.0 | 4.0 | 150.0 | --- | 24.0 | 36.0 | --- | --- | --- | --- | ----- |

CONCRETE SECTION PROPERTIES

| DEPTH OF N.A. | MOMENT OF INERTIA | SECTION MODULUS CONCRETE | SECTION MODULUS TENSION STEEL | SECTION MODULUS COMPRESSION STEEL |
|---|---|---|---|---|
| 6.80 | 14924.1 | 2194.48 | 141.66 | 0.00 |

LIVE LOAD IMPACT FACTOR FOR REACTION, MOMENT AND DEFLECTION : 1.30

```
********************************
*  T-BEAM  -  LIVE LOAD  H20   *
********************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 13.5 | 23.4 |
| 2 | 13.5 | 23.4 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 13.5 | 26.0 | 1.30 | 0.000 | 0.000 | 0.035 | 0.067 | 0.000 | 0.000 |
| 0.83 | 10.9 | 18.8 | 12.7 | 25.1 | 1.30 | 0.163 | 2.522 | 0.033 | 0.064 | 0.023 | 0.010 |
| 2.81 | 34.3 | 58.5 | 10.8 | 23.1 | 1.30 | 0.507 | 7.861 | 0.028 | 0.059 | 0.074 | 0.031 |
| 5.62 | 60.9 | 102.4 | 8.1 | 20.2 | 1.30 | 0.893 | 13.838 | 0.021 | 0.052 | 0.139 | 0.060 |
| 8.44 | 80.0 | 131.7 | 5.4 | 17.3 | 1.30 | 1.157 | 17.931 | 0.014 | 0.044 | 0.191 | 0.082 |
| 11.25 | 91.4 | 146.3 | 2.7 | 14.4 | 1.30 | 1.300 | 20.140 | 0.007 | 0.037 | 0.224 | 0.096 |
| 14.06 | 95.2 | 146.4 | 0.0 | 11.5 | 1.30 | 1.321 | 20.465 | 0.000 | 0.030 | 0.235 | 0.099 |

| X | MOMENT CAPACITY IR | OR | ULT | SHEAR CAPACITY IR | OR | ULT | | RATING FACTORS ALLOW. STRESS MOMENT | ALLOW. STRESS SHEAR | LOAD FACTOR MOMENT | LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 14.46 | 3.82 | 15.56 | 3.55 |
| | | | | | | | OR | 20.77 | 5.67 | 25.93 | 5.92 |
| 2.81 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 4.26 | 2.21 | 4.77 | 2.19 |
| | | | | | | | OR | 6.29 | 3.31 | 7.95 | 3.64 |
| 5.62 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 2.17 | 2.66 | 2.57 | 2.58 |
| | | | | | | | OR | 3.33 | 3.92 | 4.28 | 4.30 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 8.44 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.54 | 3.26 | 1.91 | 3.10 |
| | | | | | | | OR | 2.45 | 4.73 | 3.19 | 5.17 |
| 11.25 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.31 | 4.09 | 1.67 | 3.84 |
| | | | | | | | OR | 2.12 | 5.86 | 2.79 | 6.39 |
| 14.06 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.29 | 5.35 | 1.66 | 4.93 |
| | | | | | | | OR | 2.10 | 7.56 | 2.76 | 8.22 |

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*******************************
*  T-BEAM  -  LIVE LOAD  HS20  *
*******************************
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MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 13.5 | 31.3 |
| 2 | 13.5 | 31.3 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 13.5 | 34.7 | 1.30 | 0.000 | 0.000 | 0.035 | 0.089 | 0.000 | 0.000 |
| 0.83 | 10.9 | 25.0 | 12.7 | 33.3 | 1.30 | 0.196 | 3.042 | 0.033 | 0.085 | 0.023 | 0.013 |
| 2.81 | 34.3 | 76.2 | 10.8 | 30.0 | 1.30 | 0.604 | 9.355 | 0.028 | 0.077 | 0.074 | 0.043 |
| 5.62 | 60.9 | 128.9 | 8.1 | 25.4 | 1.30 | 1.038 | 16.083 | 0.021 | 0.065 | 0.139 | 0.080 |
| 8.44 | 80.0 | 158.3 | 5.4 | 20.8 | 1.30 | 1.303 | 20.184 | 0.014 | 0.053 | 0.191 | 0.109 |
| 11.25 | 91.4 | 164.3 | 2.7 | 16.2 | 1.30 | 1.398 | 21.658 | 0.007 | 0.042 | 0.224 | 0.124 |
| 14.06 | 95.2 | 147.0 | 0.0 | 11.6 | 1.30 | 1.324 | 20.518 | 0.000 | 0.030 | 0.235 | 0.129 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 10.91 | 2.88 | 11.74 | 2.68 |
| | | | | | | | OR | 15.67 | 4.28 | 19.56 | 4.47 |
| 2.81 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 3.27 | 1.70 | 3.66 | 1.68 |
| | | | | | | | OR | 4.83 | 2.55 | 6.11 | 2.80 |
| 5.62 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.72 | 2.11 | 2.04 | 2.05 |
| | | | | | | | OR | 2.65 | 3.11 | 3.40 | 3.42 |
| 8.44 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.28 | 2.71 | 1.59 | 2.58 |
| | | | | | | | OR | 2.04 | 3.93 | 2.65 | 4.30 |
| 11.25 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.17 | 3.65 | 1.49 | 3.42 |
| | | | | | | | OR | 1.89 | 5.22 | 2.48 | 5.70 |
| 14.06 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.28 | 5.33 | 1.65 | 4.92 |
| | | | | | | | OR | 2.09 | 7.54 | 2.75 | 8.20 |

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*******************************
*  T-BEAM  -  LIVE LOAD TK527  *
*******************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 13.5 | 35.6 |
| 2 | 13.5 | 35.6 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER

SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| | D.L. | LL+I | D.L. | LL+I | | FLEX STRESS | | SHEAR STRESS | | DEFLECTION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | MOMENT | MOMENT | SHEAR | SHEAR | I.F. | CONC | STEEL | D.L. | LL+I | D.L. | LL+I |
| 0.00 | 0.0 | 0.0 | 13.5 | 39.5 | 1.30 | 0.000 | 0.000 | 0.035 | 0.101 | 0.000 | 0.000 |
| 0.83 | 10.9 | 28.5 | 12.7 | 37.9 | 1.30 | 0.215 | 3.337 | 0.033 | 0.097 | 0.023 | 0.016 |
| 2.81 | 34.3 | 86.6 | 10.8 | 34.2 | 1.30 | 0.661 | 10.243 | 0.028 | 0.088 | 0.074 | 0.052 |
| 5.62 | 60.9 | 146.2 | 8.1 | 28.8 | 1.30 | 1.133 | 17.545 | 0.021 | 0.074 | 0.139 | 0.098 |
| 8.44 | 80.0 | 182.9 | 5.4 | 23.5 | 1.30 | 1.437 | 22.268 | 0.014 | 0.060 | 0.191 | 0.134 |
| 11.25 | 91.4 | 207.5 | 2.7 | 18.8 | 1.30 | 1.634 | 25.319 | 0.007 | 0.048 | 0.224 | 0.155 |
| 14.06 | 95.2 | 210.2 | 0.0 | 14.5 | 1.30 | 1.670 | 25.869 | 0.000 | 0.037 | 0.235 | 0.160 |

| | | | | | | | | RATING FACTORS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | ALLOW. STRESS | | LOAD FACTOR | |
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 9.57 | 2.53 | 10.30 | 2.35 |
| | | | | | | | OR | 13.75 | 3.75 | 17.16 | 3.92 |
| 2.81 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 2.87 | 1.49 | 3.22 | 1.48 |
| | | | | | | | OR | 4.25 | 2.24 | 5.37 | 2.46 |
| 5.62 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.52 | 1.86 | 1.80 | 1.81 |
| | | | | | | | OR | 2.34 | 2.75 | 3.00 | 3.01 |
| 8.44 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 1.11 | 2.39 | 1.38 | 2.28 |
| | | | | | | | OR | 1.76 | 3.48 | 2.29 | 3.80 |
| 11.25 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 0.92 | 3.14 | 1.18 | 2.94 |
| | | | | | | | OR | 1.50 | 4.50 | 1.97 | 4.91 |
| 14.06 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 0.90 | 4.28 | 1.15 | 3.94 |
| | | | | | | | OR | 1.46 | 6.04 | 1.92 | 6.57 |

```
********************************
*  T-BEAM  -  LIVE LOAD  ML80  *
********************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 13.5 | 37.7 |
| 2 | 13.5 | 37.7 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| | D.L. | LL+I | D.L. | LL+I | | FLEX STRESS | | SHEAR STRESS | | DEFLECTION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | MOMENT | MOMENT | SHEAR | SHEAR | I.F. | CONC | STEEL | D.L. | LL+I | D.L. | LL+I |
| 0.00 | 0.0 | 0.0 | 13.5 | 41.8 | 1.30 | 0.000 | 0.000 | 0.035 | 0.107 | 0.000 | 0.000 |
| 0.83 | 10.9 | 30.1 | 12.7 | 40.2 | 1.30 | 0.225 | 3.478 | 0.033 | 0.103 | 0.023 | 0.017 |
| 2.81 | 34.3 | 92.1 | 10.8 | 36.3 | 1.30 | 0.691 | 10.705 | 0.028 | 0.093 | 0.074 | 0.056 |
| 5.62 | 60.9 | 156.6 | 8.1 | 30.9 | 1.30 | 1.189 | 18.427 | 0.021 | 0.079 | 0.139 | 0.106 |
| 8.44 | 80.0 | 198.8 | 5.4 | 25.4 | 1.30 | 1.524 | 23.617 | 0.014 | 0.065 | 0.191 | 0.146 |
| 11.25 | 91.4 | 227.8 | 2.7 | 20.4 | 1.30 | 1.745 | 27.036 | 0.007 | 0.052 | 0.224 | 0.170 |
| 14.06 | 95.2 | 229.1 | 0.0 | 15.9 | 1.30 | 1.773 | 27.473 | 0.000 | 0.041 | 0.235 | 0.176 |

| | | | | | | | | RATING FACTORS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | ALLOW. STRESS | | LOAD FACTOR | |
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 283.3 | 402.3 | 649.3 | 108.7 | 155.1 | 209.8 | IR | 9.04 | 2.39 | 9.73 | 2.22 |
| | | | | | | | OR | 12.99 | 3.55 | 16.22 | 3.70 |
| 2.81 | 283.3 | 402.3 | 649.3 | 61.8 | 87.3 | 123.4 | IR | 2.70 | 1.40 | 3.03 | 1.39 |

```
                                                                  OR    4.00    2.11     5.05    2.32
  5.62   283.3   402.3   649.3   61.8    87.3 123.4 IR    1.42    1.74     1.68    1.69
                                                                  OR    2.18    2.56     2.80    2.81
  8.44   283.3   402.3   649.3   61.8    87.3 123.4 IR    1.02    2.22     1.27    2.11
                                                                  OR    1.62    3.22     2.11    3.52
 11.25   283.3   402.3   649.3   61.8    87.3 123.4 IR    0.84    2.90     1.07    2.71
                                                                  OR    1.37    4.15     1.79    4.52
 14.06   283.3   402.3   649.3   61.8    87.3 123.4 IR    0.82    3.88     1.06    3.57
                                                                  OR    1.34    5.48     1.76    5.96


                ++++++++++++++++++++++++++++++++++++++++++++
                +                                          +
                +        R A T I N G   S U M M A R Y       +
                +                                          +
                ++++++++++++++++++++++++++++++++++++++++++++

                        ALLOWABLE STRESS RATING        LOAD FACTOR RATING
  LOAD                   FACTOR    TONS      X          FACTOR    TONS      X
  H20     IR (DESIGN)     1.29 M   25.7   14.06          1.66 M   33.1   14.06
          OR (DESIGN)     2.10 M   42.0   14.06          2.76 M   55.2   14.06
  HS20    IR (DESIGN)     1.17 M   42.1   11.25          1.49 M   53.7   11.25
          OR (DESIGN)     1.89 M   68.1   11.25          2.48 M   89.4   11.25
 TK527    IR (DESIGN)     0.90 M   35.8   14.06          1.15 M   46.2   14.06
          OR (DESIGN)     1.46 M   58.5   14.06          1.92 M   76.9   14.06
  ML80    IR (DESIGN)     0.82 M   30.1   14.06          1.06 M   38.8   14.06
          OR (DESIGN)     1.34 M   49.1   14.06          1.76 M   64.6   14.06

      RATING FACTOR CODES:
      M - MAXIMUM MOMENT STRENGTH GOVERNS
      V - MAXIMUM SHEAR STRENGTH GOVERNS

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

### 3.3.8.3 Exterior Beam Load Rating Analysis

#### 3.3.8.3.1 BAR7 Input Parameters

The exterior channel beam was selected and will be rated using BAR7. Many of the BAR7 input parameters can be left blank if the default values are correct. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02407001300537
- Description = EXTERIOR BEAM
- Bridge Type = CTB (channel beam will be modeled as T-beam)
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 0 for Normal Output
- Concrete Deck = "Y" for top flange of channel modeled as a T-beam.
- Skew Correction Factor = 1.109 (See below). Per DM-4 Table 3.23.2(A), in determining end shear on multi-beam bridges, all beams shall be treated like the beam at the obtuse corner, i.e., the adjustment is appliable to all beams and shall be applied to the distribution factor for exterior beams.

  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

$$\text{SCF} = 1.0 + \left(\frac{12L}{90d}\right)\sqrt{\tan\ \theta}$$

  where,

  Span Length, L = 28'-1 7/16" = 28.12 ft

  Beam Depth, d = 24 in

  AASHTO Skew Angle, θ = (90° – 64° 00' 00") = 26°

  The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

  $0° \le \theta \le 60°$ θ = 26.0°, OK
  20 ft ≤ L ≤ 120 ft L = 28.12 ft, OK
  17 in ≤ d ≤ 60 in d = 24 in, OK
  3.5 ft ≤ b ≤ 6 ft b = 3 ft, NG…Call OK.
  $5 \le N_b \le 20$ $N_b = 12$, OK

  Although the range of applicability limits are exceeded for several variables, apply the calculated skew correction factor as calculated. Code guidance when the limits are exceeded is not provided.

Bridge Cross Section and Loading

- Overhang/Spacing = 3 ft / 2 = 1.5 ft
- Roadway Width = 32' – 11 ½" = 32.96 ft

- Distribution Factor – Shear

Per Pub 238 IE 6B.6.3, the exterior beam distribution factor for moment and shear shall use the larger of the LFD Distribution Factor per IP 3.3.2.2 or Lever Rule per AASHTO 3.23.2.3.

<u>LFD Distribution Factor</u>
Per AASHTO Equation 3-11, the moment distribution factor is equal to

$$DF_M = \frac{S}{D}$$

where,

Width of Precast Member, S = 3'-0" + ½" joint = 3.042 ft

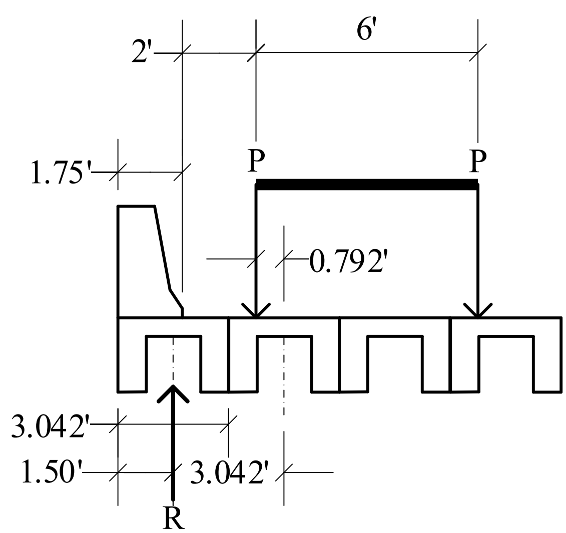
Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
Roadway Width, W = 32.96 ft
$N_L$ = 32.96 ft / 12 ft = 2.75 ==> Use 2 lanes

Span Length, L = 28.12 ft
K = 2.2 per AASHTO 3.23.4.3
W/L = 32.96 ft / 28.12 ft = 1.17 > 1, Therefore, C = K = 2.2 per AASHTO Equation 3-13

$D = (5.75 – 0.5N_L) + 0.7N_L(1 – 0.2C)^2$ per AASHTO Equation 3-12
$D = (5.75 – 0.5 \times 2) + 0.7 \times 2 \times (1 – 0.2 \times 2.2)^2 = 5.064$

$DF_V = DF_M$ = S / D = 3.042 / 5.064 = 0.601 wheels x (1 axle/2 wheels) = <u>0.300 axles</u>

<u>Lever Rule</u>
In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously when the Lever Rule is used. For the one lane condition shown below, the reduction in load intensity factor is 1.00.




$DF_V$ = P[0.792'/3.042'] x 1.0 = 0.260 wheels

$DF_V$ = 0.260 wheels / (2 wheels/axle) = 0.130 axles

- Distribution Factor – Moment
  Per Pub 238 IE 6B.6.3, the exterior beam distribution factor for moment and shear shall use the larger of the LFD Distribution Factor per IP 3.3.2.2 or Lever Rule per AASHTO 3.23.2.3. Based on the previous calculations, the exterior beam moment distribution factor for multi-beam bridges is $DF_M$ = 0.300 axles.

- Distribution Factor – Deflection
  Per AASHTO 8.13.2, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
  Roadway Width = 32.96 ft per Contract Drawing
  $N_L$ = 32.96 ft / 12 ft = 2.75 ==> Use 2 lanes
  Reduction Factor = 1.0 for 2 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 2 lanes x 1.0 / 12 beams = 0.167 lanes/beam

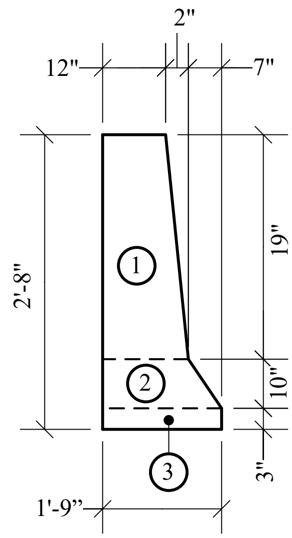
- Slab Thickness = 6.5 in (The top flange of the channel is modeled as the deck slab in BAR7.)

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight and deck slab weight (i.e., top flange) which are calculated by BAR7. Weight due to SIP forms, haunch concrete, intermediate diaphragms, and stiffeners etc are equal to zero.

  Total Dead Load 1 = 0 kips/ft

- Dead Loads – DL2
  The dead loads acting on the entire channel beam section include the barriers and asphalt wearing surface. Per IE 6B.6.1 for adjacent non-composite beams, assume the exterior beam supports 100% of the barrier dead load. Since the structure is non-composite, the wearing surface will be calculated based on a tributary width.

Roadway Barriers




Area 1 = (12 in + 14 in)/2 x 19 in = 247 in$^2$
Area 2 = (14 in + 21 in)/2 x 10 in = 175 in$^2$
Area 3 = 21 in x 3 in = 63 in$^2$
Total Area = 485 in$^2$

Barrier Weight = 485 in$^2$ / 144 in$^2$/ft$^2$ x 0.150 kcf = 0.505 kips/ft

Asphalt Wearing Surface
Based on field measurements, the asphalt wearing surface is 3” thick.
Overlay Weight = (3 in /12 in/ft) x (3.042 ft – 1.75 ft) x 0.140 kcf = 0.045 kips/ft

Total Dead Load 2 = (0.505 kips/ft + 0.045 kips/ft) = 0.550 kips/ft

- Concrete Compressive Strength, $f'_c$ = 4.5 ksi for Class AAA Concrete per Table IP 3.7.2.2-1. Per DM-4 8.2, use $f'_c$ = 4.0 ksi for Class AAA Concrete unless otherwise approved.
- Modular Ratio, n = 8 for Class A Concrete per DM-4 8.2

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = 28’-1 7/16” = 28.12 ft

Concrete Member Properties

- Type = T for T-beam Bridge
- Depth from top of slab to bottom of web, Depth = 24 in
- Width of the Web, B = 2 x 9.77 in = 19.54 in
  Top of Web Width = 10.5 in
  Bottom of Web Width = 10.5 in – (24 in – 6.5 in) / 12 = 9.04 in
  Average Web Width = (10.5 in + 9.04 in) / 2 = 9.77 in

- Distance of the tension reinforcement (AS) from the top of section, D = 24 in – 4.03 in = 19.97 in
  - Distance to 1$^{st}$ Row (4 - #9 bars) = 1.5 in + 0.5 in + 1.128 in/2 = 2.56 in
  - Distance to 2$^{nd}$ Row (4 - #9 bars) = 5.50 in
  - Total Reinforcement Area = 8 x 1.00 in$^2$ = 8.0 in$^2$
  - Bar Group CG = [4.00 in$^2$ x (2.56 in + 5.50 in)] / 8.00 in$^2$ = 4.03 in

- Area of tension reinforcement, AS = 8.00 in$^2$

- Compression reinforcement not considered. D' = 0, A'S= 0
- Reinforcement yield strength, $f_y$ = 60 ksi per AASHTO MBE Table 6B.5.2.3-1
- Stirrup details = Y indicating stirrup reinforcement will be entered
- Integral wearing surface = 0 in due to asphalt wearing surface.

Stirrup Details

- Stirrup Area = 0.40 $in^2$ for #4 (½"  diameter) stirrup leg (BAR7 assumes 2 legs)
- Yield Strength of Reinforcement Stirrups, $f_y$ = 60 ksi per AASHTO MBE Table 6B.5.2.3-1
- Location = 0.00 ft
- Spacing = 4.5 in
- Location = 2.53 ft
- Spacing = 10.0 in

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 02-4070-0130-0537 | | |
| Description: | EXTERIOR BEAM | | |
| Bridge Type: | CTB | | Channel Beam |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 0 | | Normal Output |
| Concrete Deck: | Y | | See Hand Calcs |
| Skew Correction Factor: | 1.109 | | See Hand Calcs |

**Bridge Cross Section and Loading**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for CTB |
| Overhang or Spacing: | 3.04 | ft | See Hand Calcs |
| CL of Girder: | -- | ft | Leave blank for CTB |
| Roadway Width: | 32.96 | ft | See Hand Calcs |
| Distr Factor -Shear: | 0.300 | | See Hand Calcs |
| Distr Factor -Moment: | 0.300 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.167 | | See Hand Calcs |
| Slab Thickness: | 6.50 | in | See Hand Calcs |
| Haunch: | | in | |
| Bridge DL1: | 0.000 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.550 | kip/ft | See Hand Calcs |
| F'c: | 4.0 | ksi | Per plans |
| N: | 8 | | Class A per DM-4 8.2 |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple span |
| Span Length 1: | 28.12 | ft | Per plans |

**Concrete Member Properties**

| | | | |
|---|---|---|---|
| Type: | T | | T for T-Beam Bridge |
| Depth: | 24.00 | in | See Hand Calcs |
| B: | 19.54 | in | See Hand Calcs |
| D: | 19.97 | in | See Hand Calcs |
| AS: | 8.00 | | See Hand Calcs |
| $f_y$: | 60 | | Per Contract Drawings |
| Stirrup Details: | Y | | |

**Stirrup Details**

| | | | |
|---|---|---|---|
| Stirrup Area: | 0.400 | $in^2$ | Per Contract Drawings |
| $f_{sy}$: | 60 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 4.500 | in | Per Contract Drawings |
| Location 2: | 2.53 | ft | Per Contract Drawings |
| Spacing 2: | 10.000 | in | Per Contract Drawings |

### 3.3.8.3.2 BAR7 Output

```
******************************************************************************
*                                                                            *
*                 BRIDGE ANALYSIS AND RATING (BAR7)                   330427 *
*                                                                            *
*                         COPYRIGHT (C) 1990-2018                            *
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*                                                                            *
******************************************************************************
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                     BRIDGE ANALYSIS AND RATING (BAR7)                330427

PROGRAM P4353000                                                06/14/2024 16:30
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: Exterior Channel Beam BAR7 Analysis.dat




                           PROJECT IDENTIFICATION


STRUCTURE ID - 02407001300537 - EXTERIOR BEAM

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 CTB                 0   0.00  0.0  0.0        Y                 0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     1.109        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    1.50       0.00       32.96   0.300  0.300  0.167

   SLAB                DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   6.50      0.00   0.000   0.550   4.000    8.     Y

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00


                       SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     28.12


            CONCRETE MEMBER PROPERTIES

                                             FY
TYPE  DEPTH    B      D    AS     D'   A'S   REINF
 T    24.00  19.54 19.97  8.00  0.00  0.00   60.


     ALLOWABLE FS    ST                               INTEGRAL
       IR    OR      DET   AV   SPECS  ALPHA  WEARING SURFACE
       0.0   0.0      Y   0.00    0      0.         0.0


                               STIRRUP DETAILS
```

| STIRRUP AREA | FSY | LOCATION | SPACING | LOCATION | SPACING | LOCATION | SPACING |
|---|---|---|---|---|---|---|---|
| 0.400 | 60 | 0.00 | 4.500 | 2.53 | 10.000 | 0.00 | 0.000 |
| | | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 |

DEFAULT VALUES

| SLC LEVEL | GAGE DISTANCE | PASSING DISTANCE | UNIT WEIGHT DECK | FY REINF | ALLOWABLE FS IR | ALLOWABLE FS OR | ALPHA | INTEGRAL WEARING SURFACE | STIRRUP AREA | STIRRUP FSY | SKEW CORR FACTOR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| I | 6.0 | 4.0 | 150.0 | --- | 24.0 | 36.0 | --- | --- | --- | --- | ----- |

CONCRETE SECTION PROPERTIES

| DEPTH OF N.A. | MOMENT OF INERTIA | SECTION MODULUS CONCRETE | SECTION MODULUS TENSION STEEL | SECTION MODULUS COMPRESSION STEEL |
|---|---|---|---|---|
| 8.86 | 12079.4 | 1363.20 | 135.92 | 0.00 |

LIVE LOAD IMPACT FACTOR FOR REACTION, MOMENT AND DEFLECTION : 1.30

```
********************************
*  T-BEAM  -  LIVE LOAD  H20   *
********************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 14.5 | 14.0 |
| 2 | 14.5 | 14.0 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 14.5 | 15.6 | 1.30 | 0.000 | 0.000 | 0.037 | 0.040 | 0.000 | 0.000 |
| 0.83 | 11.7 | 11.3 | 13.6 | 15.1 | 1.30 | 0.202 | 2.028 | 0.035 | 0.039 | 0.030 | 0.012 |
| 2.81 | 36.6 | 35.1 | 11.6 | 13.8 | 1.30 | 0.631 | 6.330 | 0.030 | 0.035 | 0.097 | 0.039 |
| 5.62 | 65.0 | 61.5 | 8.7 | 12.1 | 1.30 | 1.113 | 11.167 | 0.022 | 0.031 | 0.184 | 0.074 |
| 8.44 | 85.4 | 79.0 | 5.8 | 10.4 | 1.30 | 1.447 | 14.512 | 0.015 | 0.027 | 0.252 | 0.102 |
| 11.25 | 97.6 | 87.8 | 2.9 | 8.7 | 1.30 | 1.632 | 16.365 | 0.007 | 0.022 | 0.295 | 0.119 |
| 14.06 | 101.6 | 87.8 | 0.0 | 6.9 | 1.30 | 1.668 | 16.725 | 0.000 | 0.018 | 0.310 | 0.122 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 15.05 | 6.31 | 22.98 | 5.88 |
| | | | | | | | OR | 21.08 | 9.40 | 38.31 | 9.81 |
| 2.81 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 4.13 | 3.63 | 6.97 | 3.61 |
| | | | | | | | OR | 6.08 | 5.47 | 11.62 | 6.02 |
| 5.62 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 1.90 | 4.38 | 3.71 | 4.27 |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | OR | 3.01 | 6.49 | 6.18 | 7.12 |
| 8.44 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 1.22 | 5.39 | 2.73 | 5.15 |
| | | | | | | | OR | 2.08 | 7.85 | 4.55 | 8.58 |
| 11.25 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 0.96 | 6.80 | 2.37 | 6.38 |
| | | | | | | | OR | 1.74 | 9.75 | 3.95 | 10.63 |
| 14.06 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 0.91 | 8.92 | 2.34 | 8.22 |
| | | | | | | | OR | 1.69 | 12.60 | 3.91 | 13.71 |

```
*******************************
*  T-BEAM  -  LIVE LOAD  HS20  *
*******************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 14.5 | 18.8 |
| 2 | 14.5 | 18.8 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| | D.L. | LL+I | D.L. | LL+I | | FLEX STRESS | | SHEAR STRESS | | DEFLECTION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | MOMENT | MOMENT | SHEAR | SHEAR | I.F. | CONC | STEEL | D.L. | LL+I | D.L. | LL+I |
| 0.00 | 0.0 | 0.0 | 14.5 | 20.8 | 1.30 | 0.000 | 0.000 | 0.037 | 0.053 | 0.000 | 0.000 |
| 0.83 | 11.7 | 15.0 | 13.6 | 20.0 | 1.30 | 0.235 | 2.353 | 0.035 | 0.051 | 0.030 | 0.016 |
| 2.81 | 36.6 | 45.7 | 11.6 | 18.0 | 1.30 | 0.724 | 7.264 | 0.030 | 0.046 | 0.097 | 0.053 |
| 5.62 | 65.0 | 77.4 | 8.7 | 15.3 | 1.30 | 1.253 | 12.571 | 0.022 | 0.039 | 0.184 | 0.099 |
| 8.44 | 85.4 | 95.0 | 5.8 | 12.5 | 1.30 | 1.587 | 15.921 | 0.015 | 0.032 | 0.252 | 0.135 |
| 11.25 | 97.6 | 98.6 | 2.9 | 9.7 | 1.30 | 1.726 | 17.314 | 0.007 | 0.025 | 0.295 | 0.154 |
| 14.06 | 101.6 | 88.2 | 0.0 | 6.9 | 1.30 | 1.671 | 16.758 | 0.000 | 0.018 | 0.310 | 0.159 |

| | | | | | | | | RATING FACTORS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | ALLOW. | STRESS | LOAD FACTOR | |
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 11.35 | 4.76 | 17.34 | 4.44 |
| | | | | | | | OR | 15.90 | 7.09 | 28.90 | 7.40 |
| 2.81 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 3.18 | 2.79 | 5.36 | 2.78 |
| | | | | | | | OR | 4.67 | 4.20 | 8.93 | 4.63 |
| 5.62 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 1.51 | 3.48 | 2.94 | 3.39 |
| | | | | | | | OR | 2.39 | 5.15 | 4.91 | 5.66 |
| 8.44 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 1.01 | 4.49 | 2.27 | 4.28 |
| | | | | | | | OR | 1.73 | 6.53 | 3.78 | 7.14 |
| 11.25 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 0.85 | 6.06 | 2.11 | 5.68 |
| | | | | | | | OR | 1.55 | 8.68 | 3.52 | 9.47 |
| 14.06 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 0.91 | 8.89 | 2.33 | 8.20 |
| | | | | | | | OR | 1.68 | 12.56 | 3.89 | 13.66 |

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*******************************
*  T-BEAM  -  LIVE LOAD TK527  *
*******************************
```

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 14.5 | 21.4 |
| 2 | 14.5 | 21.4 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD

ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| | D.L. | LL+I | D.L. | LL+I | | FLEX STRESS | | SHEAR STRESS | | DEFLECTION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | MOMENT | MOMENT | SHEAR | SHEAR | I.F. | CONC | STEEL | D.L. | LL+I | D.L. | LL+I |
| 0.00 | 0.0 | 0.0 | 14.5 | 23.7 | 1.30 | 0.000 | 0.000 | 0.037 | 0.061 | 0.000 | 0.000 |
| 0.83 | 11.7 | 17.1 | 13.6 | 22.8 | 1.30 | 0.253 | 2.538 | 0.035 | 0.058 | 0.030 | 0.020 |
| 2.81 | 36.6 | 52.0 | 11.6 | 20.5 | 1.30 | 0.780 | 7.819 | 0.030 | 0.053 | 0.097 | 0.064 |
| 5.62 | 65.0 | 87.7 | 8.7 | 17.3 | 1.30 | 1.345 | 13.485 | 0.022 | 0.044 | 0.184 | 0.121 |
| 8.44 | 85.4 | 109.7 | 5.8 | 14.1 | 1.30 | 1.717 | 17.224 | 0.015 | 0.036 | 0.252 | 0.166 |
| 11.25 | 97.6 | 124.5 | 2.9 | 11.3 | 1.30 | 1.955 | 19.604 | 0.007 | 0.029 | 0.295 | 0.191 |
| 14.06 | 101.6 | 126.1 | 0.0 | 8.7 | 1.30 | 2.005 | 20.104 | 0.000 | 0.022 | 0.310 | 0.198 |

| | | | | | | | | RATING FACTORS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | ALLOW. STRESS | | LOAD FACTOR | |
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 9.96 | 4.18 | 15.21 | 3.90 |
| | | | | | | | OR | 13.95 | 6.22 | 25.35 | 6.49 |
| 2.81 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 2.79 | 2.45 | 4.71 | 2.44 |
| | | | | | | | OR | 4.10 | 3.69 | 7.85 | 4.07 |
| 5.62 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 1.33 | 3.07 | 2.60 | 2.99 |
| | | | | | | | OR | 2.11 | 4.55 | 4.33 | 4.99 |
| 8.44 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 0.88 | 3.96 | 1.96 | 3.79 |
| | | | | | | | OR | 1.50 | 5.77 | 3.27 | 6.31 |
| 11.25 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 0.68 | 5.22 | 1.67 | 4.90 |
| | | | | | | | OR | 1.22 | 7.48 | 2.79 | 8.16 |
| 14.06 | 181.8 | 249.9 | 578.1 | 61.8 | 87.3 | 123.4 | IR | 0.64 | 7.13 | 1.63 | 6.57 |
| | | | | | | | OR | 1.18 | 10.07 | 2.72 | 10.95 |

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********************************
*  T-BEAM  -  LIVE LOAD  ML80  *
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MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 14.5 | 22.6 |
| 2 | 14.5 | 22.6 |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

| | D.L. | LL+I | D.L. | LL+I | | FLEX STRESS | | SHEAR STRESS | | DEFLECTION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | MOMENT | MOMENT | SHEAR | SHEAR | I.F. | CONC | STEEL | D.L. | LL+I | D.L. | LL+I |
| 0.00 | 0.0 | 0.0 | 14.5 | 25.1 | 1.30 | 0.000 | 0.000 | 0.037 | 0.064 | 0.000 | 0.000 |
| 0.83 | 11.7 | 18.1 | 13.6 | 24.1 | 1.30 | 0.262 | 2.626 | 0.035 | 0.062 | 0.030 | 0.021 |
| 2.81 | 36.6 | 55.3 | 11.6 | 21.8 | 1.30 | 0.808 | 8.108 | 0.030 | 0.056 | 0.097 | 0.069 |
| 5.62 | 65.0 | 94.0 | 8.7 | 18.5 | 1.30 | 1.400 | 14.037 | 0.022 | 0.047 | 0.184 | 0.131 |
| 8.44 | 85.4 | 119.3 | 5.8 | 15.3 | 1.30 | 1.801 | 18.068 | 0.015 | 0.039 | 0.252 | 0.180 |
| 11.25 | 97.6 | 136.7 | 2.9 | 12.2 | 1.30 | 2.062 | 20.677 | 0.007 | 0.031 | 0.295 | 0.210 |
| 14.06 | 101.6 | 137.5 | 0.0 | 9.6 | 1.30 | 2.105 | 21.107 | 0.000 | 0.025 | 0.310 | 0.218 |

| | | | | | | | | RATING FACTORS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | ALLOW. STRESS | | LOAD FACTOR | |
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.83 | 181.8 | 249.9 | 578.1 | 108.7 | 155.1 | 209.8 | IR | 9.41 | 3.95 | 14.37 | 3.68 |
| | | | | | | | OR | 13.18 | 5.88 | 23.96 | 6.13 |

```
  2.81  181.8  249.9  578.1  61.8  87.3 123.4 IR   2.63   2.30    4.43   2.30
                                              OR   3.86   3.47    7.39   3.83
  5.62  181.8  249.9  578.1  61.8  87.3 123.4 IR   1.24   2.87    2.42   2.79
                                              OR   1.97   4.24    4.04   4.66
  8.44  181.8  249.9  578.1  61.8  87.3 123.4 IR   0.81   3.67    1.81   3.51
                                              OR   1.38   5.34    3.01   5.84
 11.25  181.8  249.9  578.1  61.8  87.3 123.4 IR   0.62   4.81    1.52   4.51
                                              OR   1.11   6.90    2.54   7.52
 14.06  181.8  249.9  578.1  61.8  87.3 123.4 IR   0.58   6.46    1.50   5.96
                                              OR   1.08   9.13    2.50   9.93


                ++++++++++++++++++++++++++++++++++++++++++++
                +                                          +
                +        R A T I N G   S U M M A R Y       +
                +                                          +
                ++++++++++++++++++++++++++++++++++++++++++++

                        ALLOWABLE STRESS RATING        LOAD FACTOR RATING
 LOAD                    FACTOR    TONS     X         FACTOR    TONS      X
 H20     IR (DESIGN)      0.91 M   18.2   14.06        2.34 M   46.9   14.06
         OR (DESIGN)      1.69 M   33.8   14.06        3.91 M   78.1   14.06
 HS20    IR (DESIGN)      0.85 M   30.8   11.25        2.11 M   76.1   11.25
         OR (DESIGN)      1.55 M   55.7   11.25        3.52 M  126.8   11.25
TK527    IR (DESIGN)      0.64 M   25.4   14.06        1.63 M   65.3   14.06
         OR (DESIGN)      1.18 M   47.0   14.06        2.72 M  108.8   14.06
 ML80    IR (DESIGN)      0.58 M   21.4   14.06        1.50 M   54.9   14.06
         OR (DESIGN)      1.08 M   39.5   14.06        2.50 M   91.4   14.06

      RATING FACTOR CODES:
      M - MAXIMUM MOMENT STRENGTH GOVERNS
      V - MAXIMUM SHEAR STRENGTH GOVERNS

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

## 3.4 CONCRETE SLAB/PRECAST SLAB

This Section covers the rating of cast-in-place and precast reinforced concrete slab bridges.

### 3.4.1 Policies and Guidelines

Most slab bridges can be rated by analytical methods based on design plans and field measurements. If design plans are not available, PennDOT standard plans can be utilized if field measured values are in reasonable agreement with one of the standard plans.

If plans are not available and field measurements are not in reasonable agreement with PennDOT standard plans, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.4.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating.

#### 3.4.2.1 LFR or ASR Method

Based on the age of these bridges, the typical load rating method will be LFR. PennDOT's BAR7 program (Bridge Type CSL (Cast-in-place slab) and CPL (precast slab)) should be utilized to analyze and rate these bridges when the necessary information is available to complete an analytical rating.

Multi-span continuous slab bridges cannot be modeled directly in BAR7. A potential solution would be to determine the forces with PennDOT's CBA program and calculate capacity by hand. Reach out to the BIS for guidance if you have this type of bridge.

#### 3.4.2.2 LRFR Method

Most slab bridges were built prior to 2011 and designed by load factor or allowable stress methods; therefore, the LRFR rating method would not be utilized. In the cases which an LRFR rating is required, BXLRFD can analyze a simply supported concrete top slab, or AASHTOWare BrR can be used for this analysis. See Section 2.5.3 for more information.

### 3.4.3 Live Load and Dead Load Distribution

#### 3.4.3.1 LFR or ASR Method

All distribution factors may be calculated within the BAR7 run. If analysis outside of the program is needed, compute the live load distribution factors per AASHTO Std. Spec. 3.24.3. Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted if the Lever Rule is used for computation of distribution factors.

#### 3.4.3.2 LRFR Method

All live load distribution factors may be calculated within the BXLRFD or AASHTOWare run if all applicable input values are entered. For analysis outside of the program, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.3.

### 3.4.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1918-1930 Volume 1 | Reinforced Concrete Bridge | 2/27/1922 | 608-B Series, 610-B Series, 612-B Series, 614-B Series, 616-B Series, 618-B Series |
| Standards for Old Bridges 1931-1940 Volume 2 | Reinforced Concrete Culvert | 12/15/1939 | S-184 through S-190 |
| Standards for Old Bridges 1961-1965 Volume 4 | Standard Reinforced Concrete Slab Bridge | 3/1/1961 | S-2711, S-2712 and S-2713 |
| Standards for Old Bridges 1961-1965 Volume 4 | Standard Reinforced Concrete Slab Bridges | 2/25/1965 | ST-100 |

### 3.4.5 Modeling Section Properties and Deterioration

See Section 2.5.1.2 for a discussion of modeling section loss.

For rating with LFR method when longitudinal reinforcement is rendered ineffective, adjust the reinforcement area to be analyzed in BAR7 accordingly for the worst section over a 12’-0” lane width on the bridge. For example, a slab bridge built according to Old Standard 612-B with a 12’-0” span and 18’-0” roadway width, the main longitudinal reinforcement was designed to be 7/8” diameter bars spaced at 6” making the bottom area of steel per 12” width input for BAR7:

$$A_b = \pi \text{ x } (0.875\text{in}/2)^2 = 0.6\text{in}^2$$

$$A_s|_{Design} = (12\text{in}/6\text{in}) \text{ x } 0.6\text{in}^2 = 1.2\text{in}^2$$

Inspection found intermittent spalls with exposed reinforcement that exhibited corrosion and section loss such that 6 of the bars were determined to be ineffective within a 12’-0” width near the centerline of the road. Based on the standard, there are typically 24 longitudinal bars for this size of structure spaced over a 12’-0” width. Removing the ineffective bars, the average area provided for the BAR7 input becomes:

$$A_s = (1.2\text{in}^2) \text{ x } (18/24) = \underline{0.90\text{in}^2}$$

As seen in the standard drawings, there is shear reinforcement provided in slab bridges and the area shall be averaged over the width of the bridge.  For example, a slab bridge with an 18’-0” span and constructed as per S-194 (from Standards for Old Bridges 1913-1940 Volume 2) has 7/8” diameter bent up bars are provided ‘every fourth bar’ or spaced at 1’-3”.  For the shear reinforcement area per one foot width input into BAR7 becomes:

$$A_v = (0.60\text{in}^2)/(1.25\text{ft}) = \underline{0.48\text{in}^2}$$

For rating with LRFR method when longitudinal reinforcement is rendered ineffective, adjust the reinforcement area to be analyzed (similar to above) over the equivalent strip width for lateral distribution ($E_{lat}$) in BXLRFD. BXLRFD also allows for more specific inputs of shear reinforcement, therefore the average area method is not needed.

### 3.4.6 Standard Practices

As shown in the standards, a concrete curb was typically cast with the CIP slab, however, its weight should be included with the parapet above the curb when computing DL2 and divided equally over the width of the bridge. The centerline of bearing to centerline of bearing span length is typically taken as the clear span plus 9” at each end.

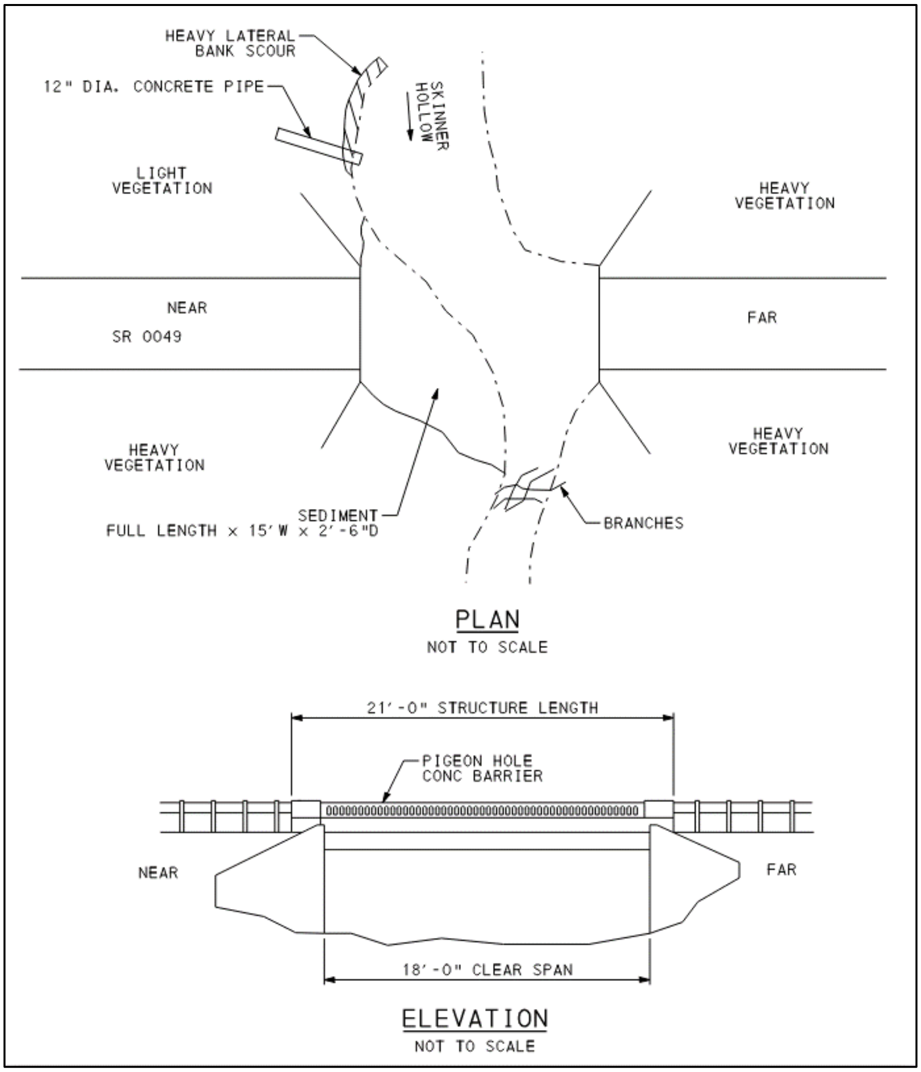
### 3.4.7 Common QA Findings

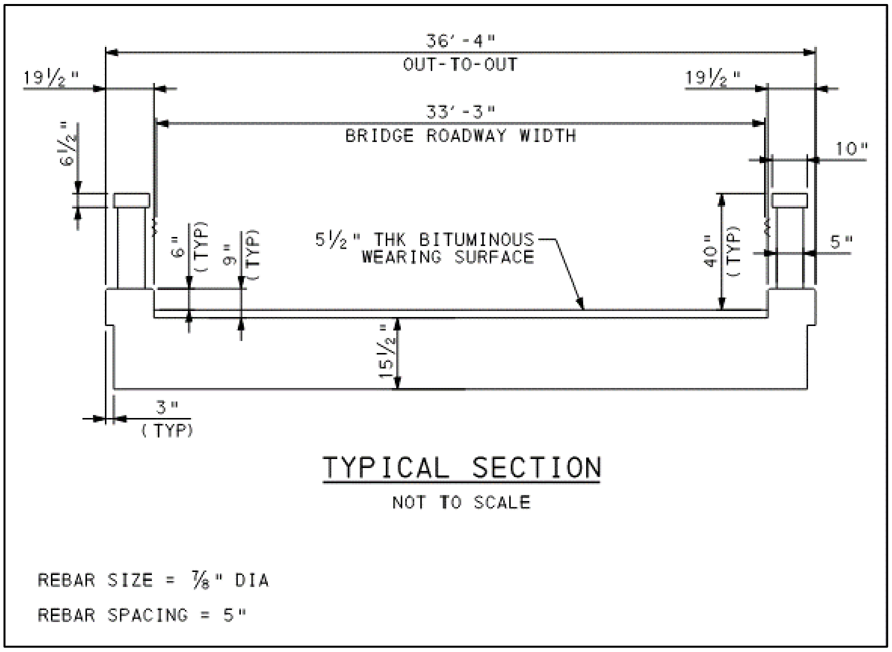
This Section is in development and will be provided in future editions of this manual.

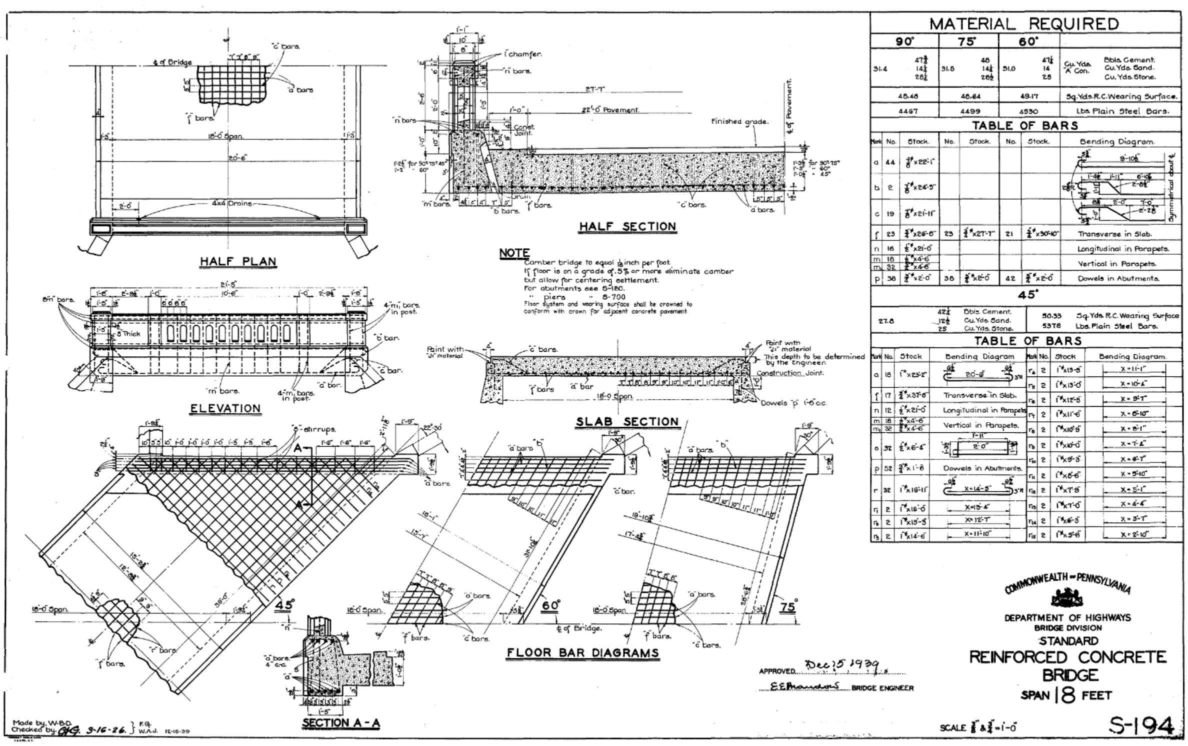
### 3.4.8 Sample Load Rating

This Section contains sample load rating calculations for a single span slab bridge shown below. The analysis will be performed using PennDOT’s Bridge Analysis and Rating (BAR7) Program, Version 7.15.0.0 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part IV (DM-4).

The 15 ½” thick reinforced concrete slab supports two lanes of traffic with a span length of 19’-3”. The bridge information is based on teh Pennsylvania Department of Highways Reinforced Concrete Bridge Standard S-194.


HEAVY LATERAL
BANK SCOUR
12" DIA. CONCRETE PIPE
SKINNER
HOLLOW
LIGHT
VEGETATION
HEAVY
VEGETATION
NEAR
SR 0049
FAR
HEAVY
VEGETATION
HEAVY
VEGETATION
SEDIMENT
FULL LENGTH x 15'W x 2'-6"D
BRANCHES
PLAN
NOT TO SCALE
21'-0" STRUCTURE LENGTH
PIGEON HOLE
CONC BARRIER
NEAR
FAR
18'-0" CLEAR SPAN
ELEVATION
NOT TO SCALE


36′-4"
OUT-TO-OUT
19½"
19½"
33′-3"
BRIDGE ROADWAY WIDTH
10"
6½"
6" (TYP)
9" (TYP)
5½" THK BITUMINOUS WEARING SURFACE
40" (TYP)
5"
15½"
3" (TYP)
TYPICAL SECTION
NOT TO SCALE
REBAR SIZE = ⅞" DIA
REBAR SPACING = 5"


MATERIAL REQUIRED
TABLE OF BARS
HALF PLAN
ELEVATION
HALF SECTION
SLAB SECTION
FLOOR BAR DIAGRAMS
SECTION A-A
NOTE
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF HIGHWAYS
BRIDGE DIVISION
STANDARD
REINFORCED CONCRETE BRIDGE
SPAN 18 FEET
BRIDGE ENGINEER
S-194

### 3.4.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

Done By: Date:

Checked By: Date:

| | | | |
|---|---|---|---|
| Structure ID (5A01): | 58-0049-0180-2680 | Inspection Date (7A01): | 11/03/2022 |
| Facility Carried (5A08): | PA 0049 over Skinner Hollow | | |
| Feature Intersected (5A07): | Skinner Hollow | | |
| Structure Type (6A26 - 6A29): | RC Slab | | |
| Spans / Members Analyzed: | Slab | | |
| Analysis Method: | LFD | | |
| PennDOT Program / Version: | BAR7 Version 7.15.0.0 | | |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 0.41 | 8.2 | 0.68 | 13.7 | 0.61 | 12.3 | * | * | M | M |
| HS20 | 0.41 | 14.7 | 0.68 | 24.6 | 0.61 | 22.1 | * | * | M | M |
| ML80 | 0.29 | 10.7 | 0.49 | 17.9 | 0.44 | 16.1 | * | * | M | M |
| TK527 | 0.34 | 13.5 | 0.56 | 22.5 | 0.50 | 20.2 | * | * | M | M |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 0.39 | 11.2 | 0.65 | 18.7 | 0.58 | 16.8 | * | * | M | M |
| EV3 | 0.27 | 11.5 | 0.44 | 19.1 | 0.39 | 17.1 | * | * | M | M |

Comments/Assumptions*:

Superstructure and substructure condition ratings are 4 and 6, respectively. Therefore, the SLC Factor equals 0.9 per PennDOT Publication 238, Table IP 4.3.2-1.
Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.
BAR7 analysis includes section loss to the main longitudinal reinforcement.
* Controlling Member is Slab.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

3.4.8.2 Reinforced Concrete Slab Bridge Load Rating Analysis

3.4.8.2.1 BAR7 Input Parameters

The reinforced concrete slab will be rated using BAR7. Many of the BAR7 input parameters can be left blank if the default values are correct. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

<u>Project Identification</u>

- Project Identification = "=BRRAT"
- Structure ID = 58004901802680
- Description = REINFORCED CONCRETE SLAB
- Bridge Type = CSL
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 0 for Normal Output

<u>Bridge Cross Section and Loading</u>

- Slab Thickness = 15 ½" per S-194 = 15.50 in

- Distribution Factors are internally calculated by BAR7 in accordance with the AASHTO Standard Specification for Highway Bridges, 17th Edition, Section 3.24.3.2.

- Dead Loads – DL1
  The only dead load acting on the non-composite section is the slab self-weight.
  Total Dead Load 1 = <u>0 kips/ft</u>

- Dead Loads – DL2
  Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  <u>Concrete Curb/Barrier</u>
  Barrier Weight = 2 curbs x 9 in x 19.5 in / 144 in$^2$/ft$^2$ x 0.150 kcf / 35.833 ft = 0.010 ksf
  Parapet Stem = 2 parapets x 5 in x 27.5 in / 144 in$^2$/ft$^2$ x 0.150 kcf / 35.833 ft = 0.008 ksf
  <u>Parapet Cap = 2 parapets x 10 in x 6.5 in / 144 in$^2$/ft$^2$ x 0.150 kcf / 35.833 ft = 0.004 ksf</u>
  Total = 0.022 ksf

  <u>Asphalt Wearing Surface</u>
  Based on field measurements, the asphalt wearing surface is 5 ½" thick.
  Overlay Weight = (5.5 in /12 in/ft) x 1 ft x 0.140 kcf = 0.064 ksf

  Total Dead Load 2 = 0.022 ksf + 0.064 ksf = <u>0.086 ksf</u>

- Concrete Compressive Strength, $f'_c$ = 3 ksi for Class A Concrete per Table IP 3.7.2.2-1.

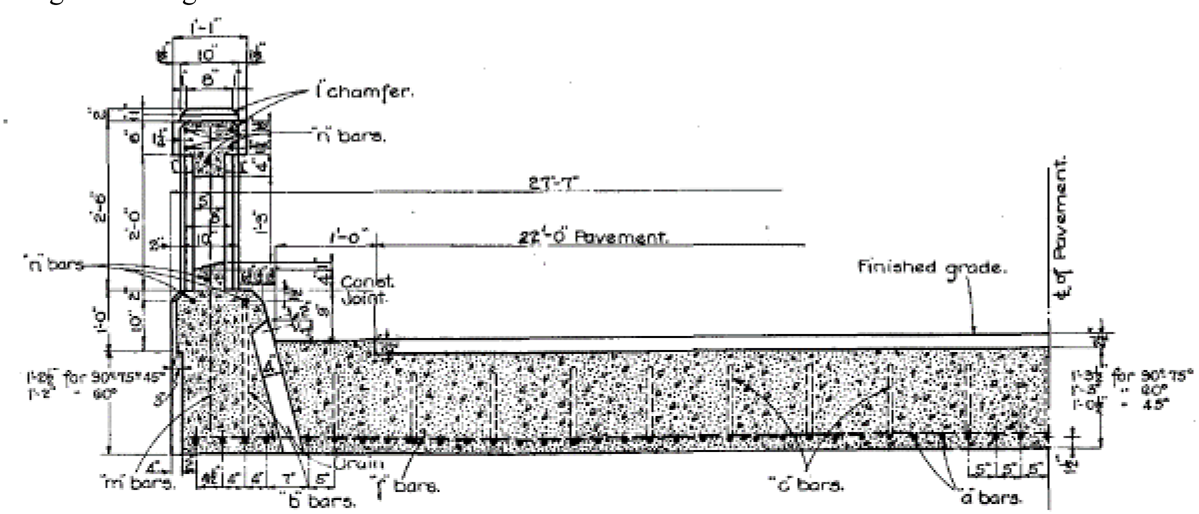
- Modular Ratio, n = 9 for Class A Concrete per DM-4 8.2

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = 18.0 ft + 2 x (7.5 in/12 in /ft) = 19.25 ft per Standard Drawings S-194

Concrete Member Properties

- Type = S for Slab Bridge
- Slab Depth, Depth = 15.50 in per S-194
- Distance of the tension reinforcement (AS) from the top of concrete section, D = 15.50 in – 1.50 in cover per S-194 – 7/8 in/2 half bar diameter = 13.56 in
- Area of tension reinforcement, AS = [$\pi$(0.75 in)$^2$/4] x 12 in / 5 in = 1.06 in$^2$
  (Standard Drawing S-194 indicates 7/8 inch diameter "a" reinforcing bars spaced at 5 inches. This reinforcement was verified in the field. At the location of the spall at the centerline of the roadway, the reinforcement bars have 3/4" in section remaining. The 1926 Edition of S-194 shows #8 bars at 6" spacing with only 1.5" of cover. These field measurements confirm the correct edition of S-194 was selected.)
- Compression reinforcement not considered. D' = 0, A'S= 0
- Reinforcement yield strength, $f_y$ = 33 ksi per AASHTO MBE Table 6B.5.2.3-1
- Stirrup area of bent up bars, AV = 1 "c" bar per 3 "a" bars per S-194 = [$\pi$(0.875 in)$^2$/4] x 12 in / (3 x 5 in) = 0.48 in$^2$
- Specs = 4 for the program to use the 1974 interim or later AASHTO Specifications for shear ratings of a reinforced concrete member.
- Alpha = tan$^{-1}$ (1 ft / 2 ft) = 26.6 degrees, Use 27 degrees. The angle is measured from the horizontal line parallel to the slab.
- Integral wearing surface = 0 in.




**HALF SECTION**

| TABLE OF BARS | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| c | 19 | 7/8"x21'-11" | | | | | | 2'-0" 7'-0" 2'-2 1/2" 1'-0" Symm. |

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 58-0049-0180-2680 | | |
| Description: | REINF CONCRETE SLAB | | |
| Bridge Type: | CSL | | Concrete Slab |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 0 | | Normal Output |

**Bridge Cross Section and Loading**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for CSL |
| Overhang or Spacing: | -- | ft | Leave blank for CSL |
| CL of Girder: | -- | ft | Leave blank for CSL |
| Roadway Width: | -- | ft | Leave blank for CSL |
| Distr Factor -Shear: | -- | | See Hand Calcs |
| Distr Factor -Moment: | -- | | See Hand Calcs |
| Distr Factor -Deflect: | -- | | See Hand Calcs |
| Slab Thickness: | 15.50 | in | Per plans |
| Haunch: | | in | |
| Bridge DL1: | 0.000 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.086 | kip/ft | See Hand Calcs |
| F'c: | 3.0 | ksi | Per plans |
| N: | 9 | | Per DM-4 8.2 |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple span |
| Span Length 1: | 19.25 | ft | Per plans/See Hand Calcs |

**Concrete Member Properties**

| | | | |
|---|---|---|---|
| Type: | S | | S forSlab Bridge |
| Depth: | 15.50 | in | See Hand Calcs |
| B: | 0.00 | in | See Hand Calcs |
| D: | 13.56 | in | See Hand Calcs |
| AS: | 1.06 | $in^2$ | See Hand Calcs |
| $f_y$: | 33 | | Per AASHTO MBE Table 6B.5.2.3-1 |
| AV: | 0.48 | $in^2$ | See Hand Calcs |
| Specs: | 4 | | See Hand Calcs |
| Alpha: | 27 | Deg | See Hand Calcs |
| Integral Wearing Surface: | 0 | in | See Hand Calcs |

3.4.8.2.2 BAR7 Output

```
******************************************************************************
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*                                                                            *
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                    BRIDGE ANALYSIS AND RATING (BAR7)                 330427

PROGRAM P4353000                                                06/19/2024 16:03
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: RC Slab Bridge BAR7 Analysis.dat




                           PROJECT IDENTIFICATION


STRUCTURE ID - 58004901802680 - REINFORCED CONCRETE SLAB

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 CSL                0   0.00  0.0  0.0        Y                  0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     0.000        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    0.00       0.00        0.00   0.000  0.000  0.000

   SLAB              DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
  15.50      0.00   0.000   0.086   3.000    9.

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00


                      SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     19.25


            CONCRETE MEMBER PROPERTIES

                                             FY
TYPE  DEPTH    B     D    AS     D'   A'S   REINF
 S    15.50   0.00 13.56  1.06  0.00  0.00   33.


     ALLOWABLE FS    ST                          INTEGRAL
       IR    OR      DET   AV   SPECS  ALPHA  WEARING SURFACE
       0.0   0.0          0.48    4     27.          0.0


                                                DEFAULT VALUES
```

| SLC LEVEL | GAGE DISTANCE | PASSING DISTANCE | UNIT WEIGHT DECK | FY REINF | ALLOWABLE FS IR | ALLOWABLE FS OR | ALPHA | INTEGRAL WEARING SURFACE |
|---|---|---|---|---|---|---|---|---|
| I | 6.0 | 4.0 | 150.0 | --- | 18.0 | 25.0 | --- | --- |

CONCRETE SECTION PROPERTIES

| DEPTH OF N.A. | MOMENT OF INERTIA | SECTION MODULUS CONCRETE | SECTION MODULUS TENSION STEEL | SECTION MODULUS COMPRESSION STEEL |
|---|---|---|---|---|
| 3.92 | 1127.5 | 287.93 | 12.99 | 0.00 |

LIVE LOAD IMPACT FACTOR FOR REACTION, MOMENT AND DEFLECTION : 1.30

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*  SLAB  -  LIVE LOAD  H20   *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR SHEAR 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR DEFLECTION 0.097

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 2.7 | 4.3 |
| 2 | 2.7 | 4.3 |

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 2.7 | 4.3 | 1.30 | 0.000 | 0.000 | 0.017 | 0.026 | 0.000 | 0.000 |
| 0.57 | 1.5 | 2.4 | 2.5 | 4.2 | 1.30 | 0.160 | 3.536 | 0.016 | 0.026 | 0.022 | 0.025 |
| 1.93 | 4.7 | 7.3 | 2.2 | 3.8 | 1.30 | 0.500 | 11.077 | 0.013 | 0.023 | 0.072 | 0.084 |
| 3.85 | 8.3 | 12.7 | 1.6 | 3.3 | 1.30 | 0.875 | 19.402 | 0.010 | 0.020 | 0.136 | 0.159 |
| 5.78 | 10.9 | 16.3 | 1.1 | 2.8 | 1.30 | 1.133 | 25.123 | 0.007 | 0.017 | 0.186 | 0.219 |
| 7.70 | 12.4 | 18.6 | 0.5 | 2.4 | 1.30 | 1.295 | 28.712 | 0.003 | 0.015 | 0.218 | 0.261 |
| 9.62 | 13.0 | 19.4 | 0.0 | 2.0 | 1.30 | 1.349 | 29.909 | 0.000 | 0.012 | 0.229 | 0.277 |

| X | MOMENT CAPACITY IR | MOMENT CAPACITY OR | MOMENT CAPACITY ULT | SHEAR CAPACITY IR | SHEAR CAPACITY OR | SHEAR CAPACITY ULT | | RATING FACTORS ALLOW. STRESS MOMENT | RATING FACTORS ALLOW. STRESS SHEAR | RATING FACTORS LOAD FACTOR MOMENT | RATING FACTORS LOAD FACTOR SHEAR |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.57 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 7.66 | 2.37 | 6.31 | 1.99 |
| | | | | | | | OR | 10.88 | 3.28 | 10.52 | 3.32 |
| 1.93 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 2.02 | 2.69 | 1.76 | 2.24 |
| | | | | | | | OR | 3.06 | 3.69 | 2.94 | 3.73 |
| 3.85 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.88 | 2.08 | 0.85 | 1.82 |
| | | | | | | | OR | 1.48 | 2.77 | 1.41 | 3.04 |
| 5.78 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.53 | 2.62 | 0.56 | 2.25 |
| | | | | | | | OR | 0.99 | 3.43 | 0.94 | 3.75 |
| 7.70 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.38 | 3.27 | 0.44 | 2.76 |
| | | | | | | | OR | 0.78 | 4.22 | 0.74 | 4.59 |
| 9.62 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.34 | 4.20 | 0.41 | 3.47 |
| | | | | | | | OR | 0.73 | 5.33 | 0.68 | 5.78 |

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*  SLAB  -  LIVE LOAD  HS20  *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR SHEAR 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR DEFLECTION 0.097

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 2.7 | 5.1 |
| 2 | 2.7 | 5.1 |

| | D.L. | LL+I | D.L. | LL+I | | FLEX STRESS | | SHEAR STRESS | | DEFLECTION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | MOMENT | MOMENT | SHEAR | SHEAR | I.F. | CONC | STEEL | D.L. | LL+I | D.L. | LL+I |
| 0.00 | 0.0 | 0.0 | 2.7 | 5.1 | 1.30 | 0.000 | 0.000 | 0.017 | 0.032 | 0.000 | 0.000 |
| 0.57 | 1.5 | 2.8 | 2.5 | 4.9 | 1.30 | 0.177 | 3.921 | 0.016 | 0.030 | 0.022 | 0.025 |
| 1.93 | 4.7 | 8.3 | 2.2 | 4.3 | 1.30 | 0.542 | 12.006 | 0.013 | 0.027 | 0.072 | 0.084 |
| 3.85 | 8.3 | 13.6 | 1.6 | 3.5 | 1.30 | 0.911 | 20.185 | 0.010 | 0.022 | 0.136 | 0.159 |
| 5.78 | 10.9 | 16.3 | 1.1 | 2.8 | 1.30 | 1.133 | 25.123 | 0.007 | 0.017 | 0.186 | 0.219 |
| 7.70 | 12.4 | 18.6 | 0.5 | 2.4 | 1.30 | 1.295 | 28.712 | 0.003 | 0.015 | 0.218 | 0.261 |
| 9.62 | 13.0 | 19.4 | 0.0 | 2.0 | 1.30 | 1.349 | 29.909 | 0.000 | 0.012 | 0.229 | 0.277 |

| | | | | | | | | RATING FACTORS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | ALLOW. STRESS | | LOAD FACTOR | |
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.57 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 6.51 | 2.01 | 5.36 | 1.69 |
| | | | | | | | OR | 9.24 | 2.79 | 8.94 | 2.82 |
| 1.93 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 1.78 | 2.36 | 1.55 | 1.97 |
| | | | | | | | OR | 2.69 | 3.25 | 2.59 | 3.28 |
| 3.85 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.83 | 1.95 | 0.79 | 1.71 |
| | | | | | | | OR | 1.38 | 2.59 | 1.32 | 2.85 |
| 5.78 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.53 | 2.62 | 0.56 | 2.25 |
| | | | | | | | OR | 0.99 | 3.43 | 0.94 | 3.75 |
| 7.70 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.38 | 3.27 | 0.44 | 2.76 |
| | | | | | | | OR | 0.78 | 4.22 | 0.74 | 4.59 |
| 9.62 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.34 | 4.20 | 0.41 | 3.47 |
| | | | | | | | OR | 0.73 | 5.33 | 0.68 | 5.78 |

******************************
*  SLAB  -  LIVE LOAD TK527  *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR SHEAR 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR DEFLECTION 0.097

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 2.7 | 5.8 |
| 2 | 2.7 | 5.8 |

| | D.L. | LL+I | D.L. | LL+I | | FLEX STRESS | | SHEAR STRESS | | DEFLECTION | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | MOMENT | MOMENT | SHEAR | SHEAR | I.F. | CONC | STEEL | D.L. | LL+I | D.L. | LL+I |
| 0.00 | 0.0 | 0.0 | 2.7 | 5.8 | 1.30 | 0.000 | 0.000 | 0.017 | 0.036 | 0.000 | 0.000 |
| 0.57 | 1.5 | 3.2 | 2.5 | 5.6 | 1.30 | 0.193 | 4.279 | 0.016 | 0.034 | 0.022 | 0.037 |
| 1.93 | 4.7 | 9.6 | 2.2 | 5.0 | 1.30 | 0.595 | 13.197 | 0.013 | 0.031 | 0.072 | 0.121 |
| 3.85 | 8.3 | 16.2 | 1.6 | 4.2 | 1.30 | 1.019 | 22.596 | 0.010 | 0.026 | 0.136 | 0.228 |
| 5.78 | 10.9 | 20.0 | 1.1 | 3.5 | 1.30 | 1.289 | 28.577 | 0.007 | 0.021 | 0.186 | 0.318 |
| 7.70 | 12.4 | 22.4 | 0.5 | 2.8 | 1.30 | 1.451 | 32.166 | 0.003 | 0.017 | 0.218 | 0.372 |
| 9.62 | 13.0 | 23.6 | 0.0 | 2.1 | 1.30 | 1.522 | 33.746 | 0.000 | 0.013 | 0.229 | 0.389 |

| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | RATING FACTORS ALLOW. STRESS | | RATING FACTORS LOAD FACTOR | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.57 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 5.71 | 1.76 | 4.70 | 1.48 |
| | | | | | | | OR | 8.11 | 2.45 | 7.84 | 2.47 |
| 1.93 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 1.54 | 2.05 | 1.34 | 1.71 |
| | | | | | | | OR | 2.33 | 2.81 | 2.24 | 2.84 |
| 3.85 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.69 | 1.63 | 0.67 | 1.43 |
| | | | | | | | OR | 1.16 | 2.18 | 1.11 | 2.39 |
| 5.78 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.43 | 2.13 | 0.46 | 1.83 |
| | | | | | | | OR | 0.81 | 2.79 | 0.76 | 3.05 |
| 7.70 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.31 | 2.86 | 0.37 | 2.41 |
| | | | | | | | OR | 0.65 | 3.69 | 0.62 | 4.01 |
| 9.62 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.28 | 3.95 | 0.34 | 3.26 |
| | | | | | | | OR | 0.60 | 5.01 | 0.56 | 5.43 |

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*  SLAB  -  LIVE LOAD  ML80  *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR SHEAR 0.097
LIVE LOAD DISTRIBUTION FACTOR FOR DEFLECTION 0.097

MAXIMUM REACTIONS

| SUPPORT | D.L. | LL+I |
|---|---|---|
| 1 | 2.7 | 6.3 |
| 2 | 2.7 | 6.3 |

| X | D.L. MOMENT | LL+I MOMENT | D.L. SHEAR | LL+I SHEAR | I.F. | FLEX STRESS CONC | FLEX STRESS STEEL | SHEAR STRESS D.L. | SHEAR STRESS LL+I | DEFLECTION D.L. | DEFLECTION LL+I |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 2.7 | 6.3 | 1.30 | 0.000 | 0.000 | 0.017 | 0.039 | 0.000 | 0.000 |
| 0.57 | 1.5 | 3.4 | 2.5 | 6.0 | 1.30 | 0.203 | 4.499 | 0.016 | 0.037 | 0.022 | 0.042 |
| 1.93 | 4.7 | 10.4 | 2.2 | 5.4 | 1.30 | 0.627 | 13.901 | 0.013 | 0.033 | 0.072 | 0.139 |
| 3.85 | 8.3 | 17.8 | 1.6 | 4.6 | 1.30 | 1.086 | 24.074 | 0.010 | 0.028 | 0.136 | 0.263 |
| 5.78 | 10.9 | 22.1 | 1.1 | 3.8 | 1.30 | 1.377 | 30.517 | 0.007 | 0.024 | 0.186 | 0.367 |
| 7.70 | 12.4 | 25.6 | 0.5 | 3.1 | 1.30 | 1.586 | 35.151 | 0.003 | 0.019 | 0.218 | 0.432 |
| 9.62 | 13.0 | 27.1 | 0.0 | 2.3 | 1.30 | 1.670 | 37.016 | 0.000 | 0.014 | 0.229 | 0.454 |

| | MOMENT CAPACITY | | | SHEAR CAPACITY | | | | RATING FACTORS ALLOW. STRESS | | RATING FACTORS LOAD FACTOR | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| X | IR | OR | ULT | IR | OR | ULT | | MOMENT | SHEAR | MOMENT | SHEAR |
| 0.00 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 0.00 | | 0.00 | |
| | | | | | | | OR | 0.00 | | 0.00 | |
| 0.57 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 5.31 | 1.64 | 4.37 | 1.38 |
| | | | | | | | OR | 7.54 | 2.28 | 7.29 | 2.30 |
| 1.93 | 19.5 | 27.1 | 34.1 | 12.4 | 16.2 | 21.3 | IR | 1.43 | 1.90 | 1.25 | 1.58 |
| | | | | | | | OR | 2.16 | 2.60 | 2.08 | 2.63 |
| 3.85 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.63 | 1.48 | 0.61 | 1.31 |
| | | | | | | | OR | 1.06 | 1.98 | 1.01 | 2.18 |
| 5.78 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.39 | 1.93 | 0.42 | 1.65 |
| | | | | | | | OR | 0.73 | 2.52 | 0.69 | 2.76 |
| 7.70 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.28 | 2.59 | 0.32 | 2.18 |
| | | | | | | | OR | 0.57 | 3.34 | 0.54 | 3.64 |
| 9.62 | 19.5 | 27.1 | 34.1 | 8.5 | 10.8 | 15.2 | IR | 0.24 | 3.72 | 0.29 | 3.07 |
| | | | | | | | OR | 0.52 | 4.72 | 0.49 | 5.12 |

```
+++++++++++++++++++++++++++++++++++++++++++++
+                                           +
```

```
                +        R A T I N G   S U M M A R Y        +
                +                                           +
                +++++++++++++++++++++++++++++++++++++++++++++

                        ALLOWABLE STRESS RATING         LOAD FACTOR RATING
  LOAD                   FACTOR    TONS      X         FACTOR    TONS      X
  H20     IR (DESIGN)     0.34 M    6.7     9.62        0.41 M    8.2     9.62
          OR (DESIGN)     0.73 M   14.5     9.62        0.68 M   13.7     9.62
  HS20    IR (DESIGN)     0.34 M   12.1     9.62        0.41 M   14.7     9.62
          OR (DESIGN)     0.73 M   26.1     9.62        0.68 M   24.6     9.62
 TK527    IR (DESIGN)     0.28 M   11.1     9.62        0.34 M   13.5     9.62
          OR (DESIGN)     0.60 M   23.9     9.62        0.56 M   22.5     9.62
  ML80    IR (DESIGN)     0.24 M    8.8     9.62        0.29 M   10.7     9.62
          OR (DESIGN)     0.52 M   19.1     9.62        0.49 M   17.9     9.62

      RATING FACTOR CODES:
      M - MAXIMUM MOMENT STRENGTH GOVERNS
      V - MAXIMUM SHEAR STRENGTH GOVERNS

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

## 3.5 COMPOSITE PRESTRESSED I-GIRDERS

This Section covers the rating of precast prestressed reinforced concrete I-girder bridges. Policies recommended herein should be applied to precast prestressed reinforced concrete PA Bulb-T girder bridges.

### 3.5.1 Policies and Guidelines

Most prestressed I-girder bridges can be rated by analytical methods based on design plans and field measurements. If design plans are not available, field measurements alone cannot be used to rate prestressed beams since the strand pattern cannot be verified in the field.

If plans are not available and field measurements are not in reasonable agreement with PennDOT standard plans, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.5 for guidance on prestressed concrete, MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.5.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating. Allowable stress design method would not have been used for these bridges.

#### 3.5.2.1 LFR or ASR Method

For bridges built prior to 2011, if load factor design methodology was used then the load rating should be performed using PS3. For bridges built prior to 2011 and designed with LRFD, LFR or LRFR can be utilized for the rating.

#### 3.5.2.2 LRFR Method

For prestressed I-girder bridges designed with LRFD on or after 2011, load ratings should be performed using LRFD in PSLRFD.

### 3.5.3 Live Load and Dead Load Distribution

#### 3.5.3.1 LFR or ASR Method

Pub. 238 Section 3.3 provides guidance on the distribution of live load to longitudinal girders. Typically, the live load distribution factor for exterior beams is in accordance with the AASHTO Standard Specification.

The typical distribution factors are as follows when AASHTO distribution factors are applicable:

| Exterior | | Interior | |
|---|---|---|---|
| Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.3.1.2) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.2 and Table 3.23.1) |
| Lever Rule* | Lever Rule* | Lever Rule* | • S/7.0 (one lane)<br>If S > 10' use Lever Rule<br>• S/5.5 (2 or more lanes)<br>If S > 14' use Lever Rule |

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2.2.3 and Pub. 238 IP 3.3.3.1 for additional discussion of skew.

Dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all girders (AASHTO Standard Specification 3.23.2.3.1.1).

#### 3.5.3.2 LRFR Method

For prestressed I-girder bridges designed with LRFD, distribution factors may be computed by PSLRFD. For analysis outside of the program, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.2.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO distribution factors. See DM-4 Table 4.6.2.2.3c-1 for the formulas for the skew correction factor.

### 3.5.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1941-1960 Volume 3 | Prestressed Concrete Bridge Standards – I-Beams Composite | 9/19/1960 | S-3908 |
| Standards for Old Bridges 1961-1965 Volume 4 | Prestressed Concrete Bridge Standards | 3/8/1962 | S-3909 |
| Standards for Old Bridges 1965-1972 Vol. 5 | Instructions for Acceptance of Prestressed Beams with Cracks | 3/17/70 | P-800 |
| BD-201 March 1973 | Prestressed Concrete Beam Standards | 3/1/1973 | 4 through 22 |
| BD-600 Series, Jan. 1989 Edition | Various | 1/20/1989 | BD-652, BD-662, BD-666 |

| | | | |
|---|---|---|---|
| BD-600 Series, 1989 Edition, Change 3 | Various | 9/11/1989 | BD-652, BD-662 |
| Reference Document | Topic | Approval Date | Relevant Drawings |
| BD-600 Series, July 1993 Edition | Various | 7/1/1993 | BD-652, BD-662, BD-666 |
| BD-600 Series, Sept. 1994 Edition | Various | 9/30/1994 | BD-652, BD-662, BD-666 |
| BD-651 and BD-662 12/24/1999; BD-652 and BD-666 deleted 6/30/2000 | | | |
| Online document not available | I-Beam and PA Bulb-Tee Beam Reinforcement Details | 12/24/1999 | BD-662 |
| Online document not available | Various | 6/30/2000 | BD-652, BD-664 |
| Online document not available | Various | 1/21/2003 | BD-652, BD-662, BD-664 |
| Online document not available | Various | 4/15/2004 | BD-652, BD-662, BD-664 |
| Online document not available | Various | 7/29/2005 | BD-662, BD-664 |
| Online document not available | Various | 7/24/2006 | BD-662, BD-664 |
| Online document not available | Various | 7/20/2007 | BD-652, BD-662, BD-664 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 12/29/2008 | BD-652 |
| Online document not available | Various | 9/20/2010 | BD-652, BD-662, BD-664 |
| Online document not available | Various | 8/31/2012 | BD-652, BD-662, BD-664 |
| Standards for Bridge Design April 2016 Edition Change 7 | Various | 4/29/2016 (Latest Revision 10/7/2024) | BD-652, BD-662, BD-664 |

### 3.5.5 Modeling Section Properties and Deterioration

See Section 2.5.4.2 for a discussion of modeling section loss.

See Section 2.5.5 for discussion on Post-Tensioned Concrete Superstructures.

See Pub. 238, Section IE 6.1.5.3I for guidance on removing strands from analysis based on various defects in prestressed beams. In addition, see Pub. 238, Appendix IE 04-D for an example of adjusting strand patterns based on this guidance. It is acceptable to apply this methodology when rating I-girders with deterioration. For composite beams, method A would likely be overly conservative; therefore, method B would be more appropriate.

### 3.5.6 Standard Practices

For continuous prestressed concrete bridges, if it is determined that the deck steel over the pier will control the rating, the bridge may be rated as a simple span if it increases the rating. Provide documentation in the load rating report that the deck steel controlled and the ratings are based on a simple span analysis.

For prestressed bridges designed using LFD, the shear capacity of the beam should be determined as per AASHTO Std. Spec. 9.20 with a shear capacity reduction factor of 0.9. If shear controls the ratings for a bridge built prior to 1992 and there is no sign of shear distress in the beams, the shear capacity reduction factor may be increased to 1.0 as per the 1979 edition of the AASHTO Std. Spec.

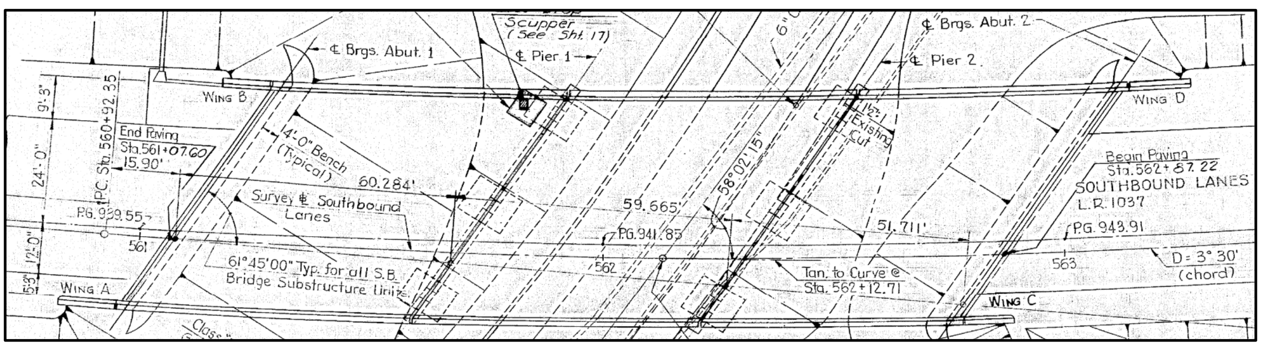
### 3.5.7 Common QA Findings

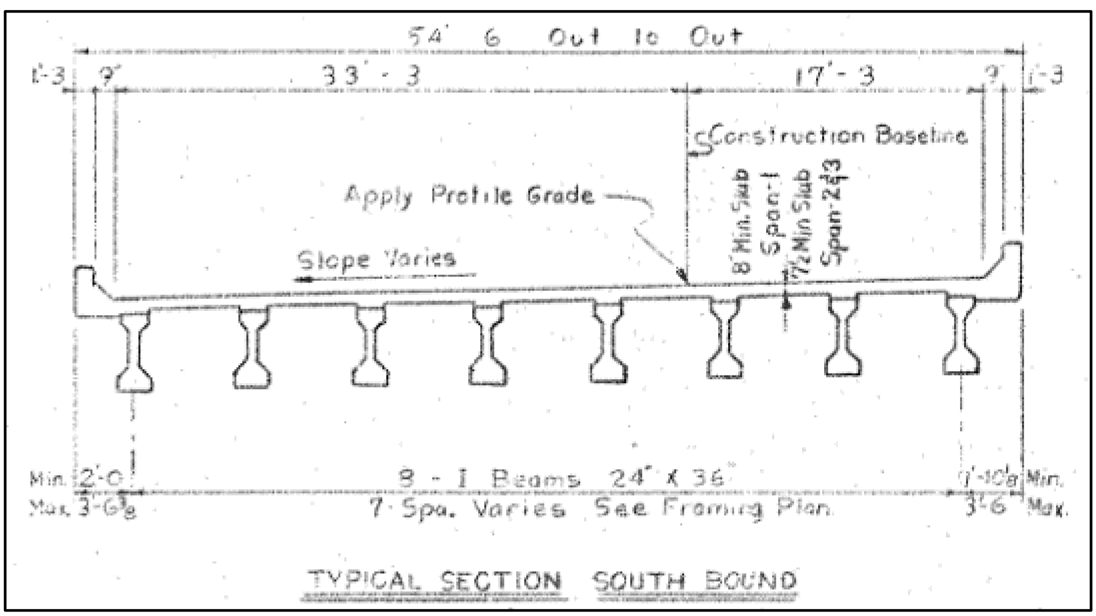
This Section is in development and will be provided in future editions of this manual.

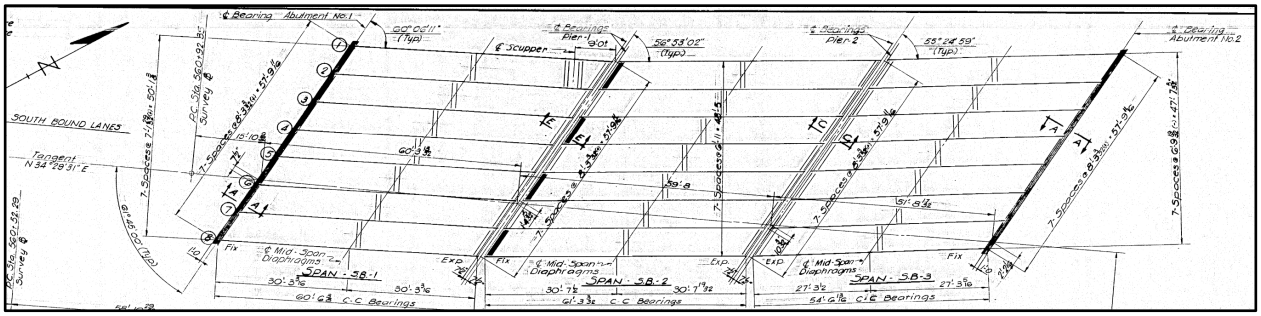
### 3.5.8 Sample Load Rating

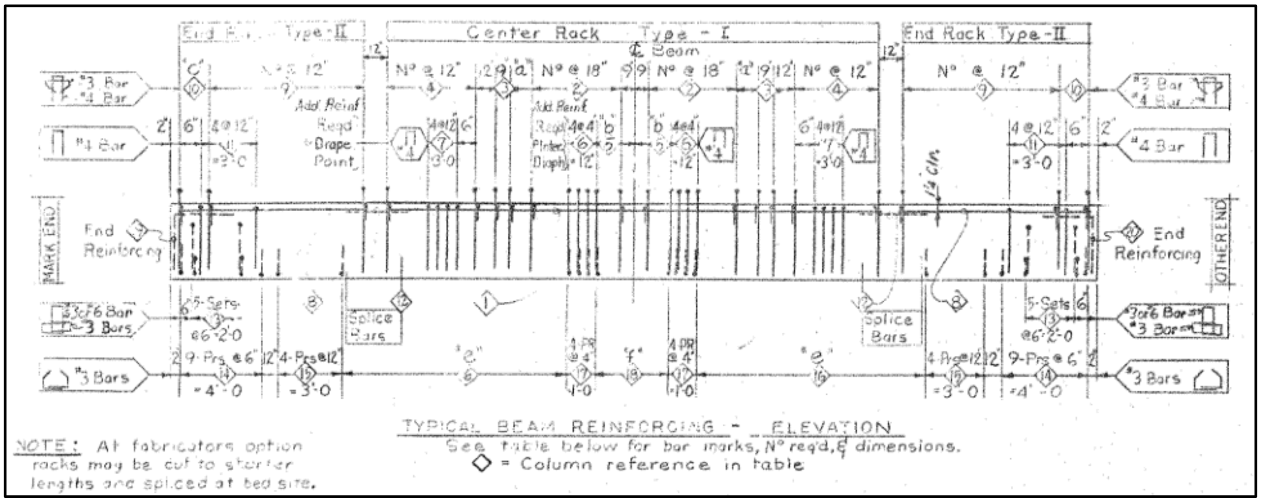
This Section contains sample load rating calculations for an interior beam of a composite prestressed concrete I-beam structure shown below. The analysis will be performed using PennDOT's LFD Prestressed Concrete Girder Design and Rating (PS3) Program, Version 3.6.0.5 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part-IV (DM-4).

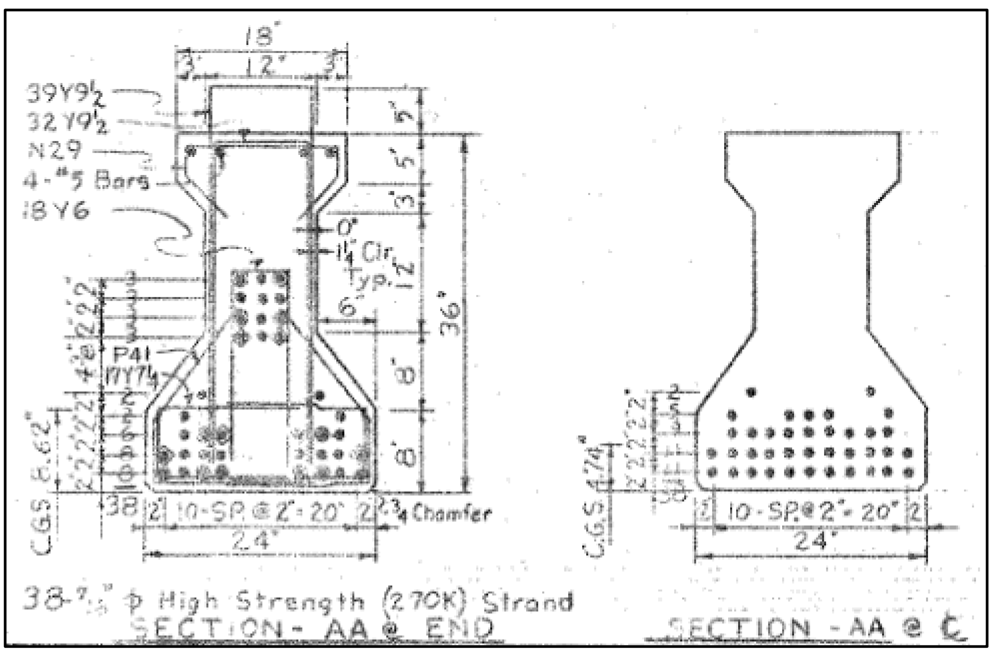
The superstructure contains three (3) simple spans with of an 8" composite reinforced concrete deck in Span 1 supported on eight (8) prestressed concrete I-beams. The 24" wide x 36" deep concrete beams are spaced at approximately 7'- 1 29/32" with varying deck overhang. For simplicity, only an interior beam in Span 1 will be rated as it has a higher distribution factor and dead load and by inspection, will control.






TYPICAL BEAM REINFORCING - ELEVATION
See table below for bar marks, N° req'd, & dimensions.
◇ = Column reference in table
NOTE: At fabricators option racks may be cut to shorter lengths and spliced at bed site.


38-7/16" ɸ High Strength (270K) Strand
SECTION - AA @ END
SECTION - AA @ ℄

3.5.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

| | |
|---|---|
| Done By: | Date: |
| Checked By: | Date: |
| Structure ID (5A01): | Inspection Date (7A01): 6/21/2023 |
| Facility Carried (5A08): | SR 0028 |
| Feature Intersected (5A07): | Powers Run Road |
| Structure Type (6A26 - 6A29): | 3-Span P/S I-Beam |
| Spans / Members Analyzed: | Span 1, Beam 3 |
| Analysis Method: | LFD |
| PennDOT Program / Version: | PS3 Version 3.6.0.5 |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 1.23 | 24.7 | 3.80 | 76.1 | 3.80 | 76.1 | * | * | M | V |
| HS20 | 0.86 | 31.0 | 2.68 | 96.5 | 2.68 | 96.5 | * | * | M | M |
| ML80 | 0.72 | 26.5 | 2.25 | 82.6 | 2.25 | 82.6 | * | * | M | M |
| TK527 | 0.76 | 30.4 | 2.35 | 94.0 | 2.35 | 94.0 | * | * | M | M |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 1.01 | 29.1 | 3.12 | 89.8 | 3.12 | 89.8 | * | * | M | V |
| EV3 | 0.66 | 28.3 | 2.05 | 88.2 | 2.05 | 88.2 | * | * | M | M |

Comments/Assumptions*:

Superstructure and substructure condition ratings are both 5. Therefore, the SLC Factor equals 1.0 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is Beam 3 in Span 1.

PS3 analysis accounts for exposed corner strand/beam end deterioration and 1.25" latex wearing surface.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

### 3.5.8.2 Interior Girder Load Rating Analysis

#### 3.5.8.2.1 PS3 Input Parameters

Interior beam 3 in Span 1, which has concrete spalling and a debonded strand, was selected and will be rated using PS3 if the default values are correct. Many of the PS3 input parameters can be left blank. Only the required input values are discussed below. Refer to the PS3 User's Manual for additional information.

<u>Project Identification</u>

- Project Identification = "=PRSTR"
- Structure ID = 02002802410000
- Description = INTERIOR BEAM 3
- Live Load = Blank for H, HS, ML80, TK527, EV2, and EV3 vehicles.
- Output = 0 for normal output
- I or F = I for Interior Beam
- Design = R for rating only
- Skew Correction Factor = Blank for Interior Beam

<u>Bridge Cross Section and Loading</u>

- Beam Spacing, S = 7'-1 29/32" = 85.9 in per Contract Drawings
- Distribution Factor – Shear
  Per AASHTO 3.23.1, the interior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the two lane condition shown below, the reduction in load intensity factor is 1.00.

  2 Lanes Loaded, $DF_V$ = P[1 + (7.16' - 6')/7.16' + (7.16' – 4')/7.16'] x 1.00 = 1.603 wheels = 1.603 wheels / (2 wheels/axle) = <u>0.802 axles</u> (Controls)

- Distribution Factor – Moment
  Per AASHTO Table 3.23.1, the interior beam distribution factor for moment is calculated as S/5.5 for bridges with two or more traffic lanes. In accordance with AASHTO 3.12.2, the reduction in load intensity is not applicable when Table 3.23.1 is used for moment in longitudinal beams.

  $DF_M$ = S / 5.5 = 7.159 ft / 5.5 = 1.302 wheels x (1 axle/2 wheels) = <u>0.651 axles</u>

- Distribution Factor – Deflection
  Per AASHTO 8.13.2, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes = 50.5 ft = 12 ft = 4.21, Use 4 lanes
  Reduction Factor = 0.75 for 4 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 4 lanes x 0.75 / 8 beams = <u>0.375 lanes/beam</u>

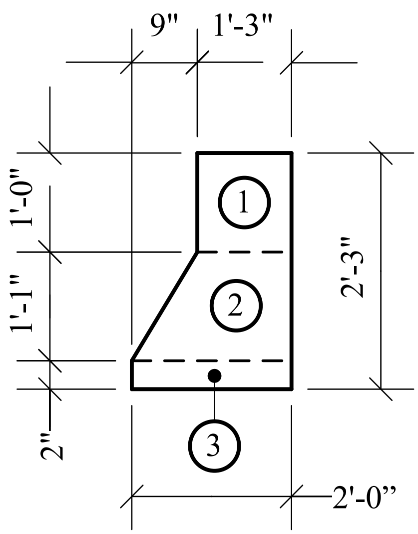
- Uniform Dead Loads – UDLF
  UDLF = Blank. PS3 will internally calculate the weight of the stay-in-place formwork using 0.015 ksf for I-beams.

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight, deck slab weight, SIP forms, haunch concrete, and diaphragms. Girder self-weight, deck slab weight, and diaphragm weight are calculated by PS3. For this analysis run, PS3 assumed one interior diaphragm located at midspan.
  DL1 Haunch Weight = (1 in x 18 in) / 144 $in^2/ft^2$ x 0.150 kcf = 0.019 kips/ft

- Future Wearing Surface – FWS
  Future wearing surfaces are not included in the rating analysis.
  FWS = 0 kips/ft or blank.

- Dead Loads – DL2
  The dead loads acting on the composite section include outside barriers. Additionally, a latex overlay was placed on the structure. Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  Roadway Barriers




| | |
|---|---|
| Area 1 = 15 in x 12 in = | 180 $in^2$ |
| Area 2 = (15 in + 24 in)/2 x 13 in = | 253.5 $in^2$ |
| Area 3 = 24 in x 2 in = | 48 $in^2$ |
| Total Area = | 481.5 $in^2$ |

  Barrier Weight = 481.5 $in^2$ / 144 $in^2/ft^2$ x 0.150 kcf = 0.502 kips/ft

  Metal Railing
  Metal Railing = 0.015 kips/ft

  Latex Overlay
  A 1 ¼" latex overlay was previously placed on the structure.
  Overlay Weight = (1.25 in /12 in/ft) x 50.5 ft x 0.150 kcf = 0.789 kips/ft

  Total Dead Load
  Total DL 2 = [2 x (0.502 kips/ft + 0.015 kips/ft) + 0.789 kips/ft] / 8 beams = 0.228 kips/ft

- End Eccentricity = 16.17 in (distance from neutral axis to bottom of beam) – 8.52 in (center of gravity of strands at centerline of bearing) = 7.65 in. Note that the Contract Drawings indicate the

center of gravity of 8.62 in at the end of the beam. Use of similar triangles results in the center of gravity 8.52 in at the centerline of bearings.
- Drape Point Location = (60.53 ft – 9 ft)/60.53 ft = 0.3513
- Strand L or S = S for Stress-Relieved Strands

Span Lengths

- Span 1 Length = 60'-6 3/8" = 60.53 ft
- Beam Projection = (61.6979 ft – 60.5313 ft) = 0.5833 ft = 7 in

Exterior Diaphragm Details

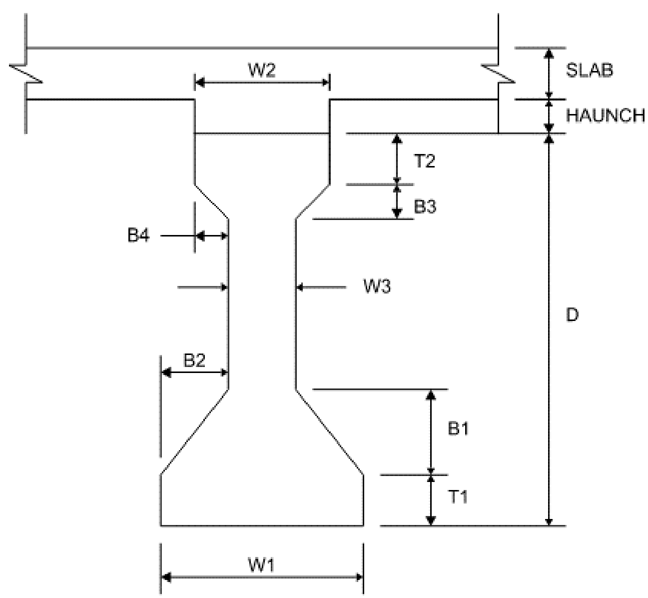
- Identify = E for Exterior Diaphragms
- Thickness = 12.00 in
- Number of Diaphragms = 3
- Distance 1 = 0 ft from centerline of left bearing
- Distance 2 = 60.53 ft / 2 = 30.27 ft from centerline of left bearing
- Distance 3 = 60.53 ft from centerline of left bearing

Prestress Criteria

- 28-Day Compressive Strength of Beam Concrete = 5.5 ksi
- 28-Day Compressive Strength of Slab Concrete = 3.5 ksi
- Compressive Strength of Beam Concrete at Initial Prestressing = 5.0 ksi
- Ultimate Strength of Prestressing Steel = 270 ksi
- Strand Diameter = 7/16" = 0.4375 in
- Number of Strand Rows = 5
- Number of Debonded Lengths = 1
- Stirrup Details = Y indicating stirrup reinforcement will be entered

Prestressed Concrete Beam Dimensions

- Type = I for an I-beam
- Comp = Y for a composite beam
- Design or D = 36 in beam depth
- W1 = 24.0 in
- W2 = 18.0 in
- W3 = 12.0 in
- T1 = 8.0 in
- T2 = 5.0 in
- B1 = 8.0 in
- B2 = 6.0 in
- B3 = 3.0 in
- B4 = 3.0 in
- D1 = 0 in for I-beam
- D2 = 0 in for I-beam
- X1 = 0 in for I-beam



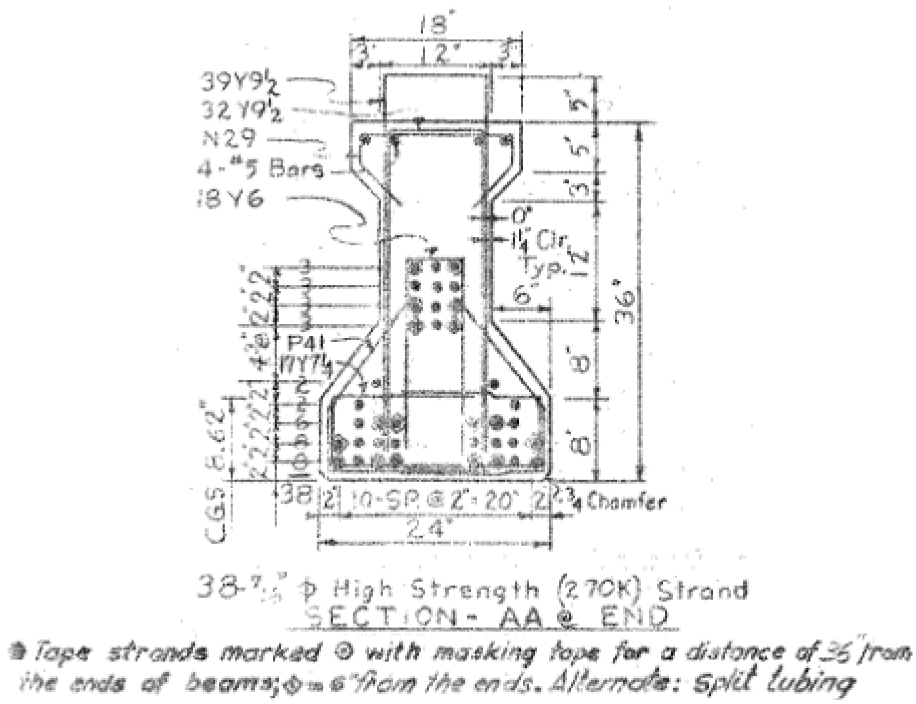

Regular I-beam

- X2 = 0 in for I-beam
- Slab Thickness = 8” – 1 ¼” Scarification = 6.75 in (effective). As noted in the PS3 Users Manual Section 5.7 for slab thickness, the program adds an additional ½” of deck concrete for sacrificial wearing surface for the dead load moment and shear calculations for composite beams. Since the bridge has been scarified with a 1 ¼” latex overlay added, the overlay thickness could have been reduced by ½”.
- Haunch Thickness = 0 in (May be included if actual haunch height measured in field.)

Strand Details

- Strand Area = 0.115 in$^2$
- G1 = 2.00 in
- G2 = 2.00 in
- R1 = 11
- R2 = 11
- R3 = 9
- R4 = 5
- R5 = 2

Debonded Strand Details
Based on the Contract Drawings, there are 2 separate debonding lengths at 6 inches and 36 inches from the beam end. In Row 1, there are 6 strands debonded for 36 inches and 2 strands debonded for 6 inches. In Rows 2 and 3, there are 2 strands debonded for 36 inches and 2 strands debonded for 6 inches. In Row 4, there are 2 strands debonded for 36 inches. Since the debonding length in PS3 is measured from the centerline of bearing and must be greater than zero, the debonding at 6 inches cannot be entered in PS3 since it is beyond the centerline of bearing. Based on a recent inspection, part of the bottom flange has spalled off for 24 inches from the beam end. This has exposed the corner strand for approximately 18 inches. Since the corner strand is already debonded for 36 inches, no further adjustment is needed.



- Debonded Length, $L_x$ = 36 in – 7 in = 29 in = 2.417 ft from centerline of bearing
- Row 1 – 6 strands debonded
- Row 2 – 2 strands debonded
- Row 3 – 2 strands debonded
- Row 4 – 2 strands debonded

Stirrup Details

- Spec = Blank for shear values to be computed per 1979 AASHTO Interim Specifications
- Stirrup Area = 0.20 $in^2$ for one leg of a #4 stirrup
- Yield Strength of Reinforcement Stirrups, $f_y$ = 40 ksi
- Location = 0.00 ft
- Spacing = 6.0 in
- Location = 4.00 ft
- Spacing = 12.0 in

**TITLE**

| | | | |
|---|---|---|---|
| Structure ID: | 02-0028-0241-0000 | | |
| Description: | INTERIOR BEAM 3 | | |
| SLC Level: | | | |
| Live Load: | | | H20, HS20, ML80, TK527, EV2, and EV3 |
| Output: | 0 | | Normal output |
| Impact Factor: | | | |
| Gage Distance: | | ft | |
| Passing Distance: | | ft | |
| Roadway Width: | 48.00 | ft | Per Contract Drawings |
| DLF: | | | |
| LLF: | | | |
| I or F: | I | | Interior beam |
| Principal: | | | |
| Design: | R | | Rating only problem |
| Skew Correction Factor: | | | NA for Interior Beam |
| IR Stress Level: | | | |
| AASHTO fc: | | | |

**Bridge Cross-Section and Loading**

| | | | |
|---|---|---|---|
| Beam Spacing: | 85.900 | in | Per Contract Drawings |
| Distr Factor -Shear: | 0.802 | | See Hand Calcs |
| Distr Factor -Moment: | 0.651 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.375 | | See Hand Calcs |
| Unit Weight of Deck Concrete: | | | |
| UDLF: | 0.000 | | See Hand Calcs |
| DL1: | 0.019 | kip/ft | See Hand Calcs |
| FWS: | 0.000 | kip/ft | See Hand Calcs |
| DL2: | 0.228 | kip/ft | See Hand Calcs |
| Initial P/S Force: | | kips | |
| Midspan Eccentricity: | | in | |
| End Eccentricity: | 7.650 | in | See Hand Calcs |
| P/S Loss %: | | | |
| Drape Point: | 0.3513 | | See Hand Calcs |
| Strand L or S: | S | | Stress Relieved Strands |
| Rate FWS?: | | | |

**Span Lengths**

| | | | |
|---|---|---|---|
| Span Length 1: | 60.53 | ft | Per Contract Drawings/See Hand Calcs |
| Beam Projection: | 7.000 | in | Per Contract Drawings |

**Diaphragm Details**

| | | | |
|---|---|---|---|
| Identification: | E | | Per Contract Drawings |
| Thickness: | 12.00 | in | Per Contract Drawings |
| No. of Diaphragms: | 3 | | Per Contract Drawings |
| Distance 1: | 0.00 | ft | Per Contract Drawings |
| Distance 2: | 30.27 | ft | Per Contract Drawings |
| Distance 3: | 60.53 | ft | Per Contract Drawings |
| Distance 4: | | ft | |

**Prestress Criteria**

| | | | |
|---|---|---|---|
| Beam Conc $f'_{cb}$: | 5.5 | ksi | Per Contract Drawings |
| Slab Conc $f'_{cs}$: | 3.5 | ksi | Per Contract Drawings |
| Conc Init $f'_{ci}$: | 5.0 | ksi | Per Contract Drawings |
| Strand Ult $f'_{s}$: | 270 | ksi | Per Contract Drawings |
| Strand Diameter: | 0.4375 | in | Per Contract Drawings |
| No. of Rows: | 5 | | Per Contract Drawings |
| No. Lx: | 1 | | Per Contract Drawings |
| St. Det: | Y | | Stirrup Details |

**Beam Dimensions**

| | | | |
|---|---|---|---|
| Beam Type: | I | | I-Beam |
| Composite: | Y | | Composite |
| Designation or D: | 36 | | Per Contract Drawings |
| W1: | 24 | in | |
| W2: | 18 | in | |
| W3: | 12 | in | |
| T1: | 8 | in | |
| T2: | 5 | in | |
| B1: | 8 | in | |
| B2: | 6 | in | |
| B3: | 3 | in | |
| B4: | 3 | in | |
| D1: | 0 | in | |
| D2: | 0 | in | |
| X1: | 0 | in | |
| X2: | 0 | in | |
| Slab Thickness: | 6.75 | in | See Hand Calcs |
| Haunch: | 0 | in | |

**Strands**

| | | | |
|---|---|---|---|
| Strand Area = | 0.115 | $in^2$ | Per Contract Drawings |
| G1: | 2.000 | in | Per Contract Drawings |
| G2: | 2.000 | in | Per Contract Drawings |
| R1: | 11 | | Per Contract Drawings |
| R2: | 11 | | Per Contract Drawings |
| R3: | 9 | | Per Contract Drawings |
| R4: | 5 | | Per Contract Drawings |
| R5: | 2 | | Per Contract Drawings |

**Debonded**

| | | | |
|---|---|---|---|
| Debonded Length= | 2.417 | ft | See Hand Calcs |
| R1: | 1 | | |
| No. Strands: | 6 | | |
| R2: | 2 | | |
| No. Strands: | 2 | | |
| R3: | 3 | | |
| No. Strands: | 2 | | |
| R4: | 4 | | |
| No. Strands: | 2 | | |

**Stirrups**

| | | | |
|---|---|---|---|
| Spec: | | | |
| Area: | 0.200 | $in^2$ | Per Contract Drawings |
| $f'_y$: | 40 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 6.000 | in | Per Contract Drawings |
| Location 2: | 4.00 | ft | Per Contract Drawings |
| Spacing 2: | 12.000 | in | Per Contract Drawings |
| Location 3: | | ft | Per Contract Drawings |
| Spacing 3: | | in | Per Contract Drawings |
| Location 4: | | ft | |
| Spacing 4: | | in | |
| Location 5: | | ft | |
| Spacing 5: | | in | |

3.5.8.2.2 PS3 Output

```
            LFD Prestressed Concrete Girder Design and Rating         330740

PROGRAM P4353050                                                05/06/2024 10:29
VERSION 3.6.0.5          LAST UPDATED 12/22/2023        DOCUMENTATION 12/2023

INPUT: Interior PS I-Beam PS3 Analysis.dat




STRUCTURE ID - 02002802410000 - INTERIOR BEAM 3

 SLC   LIVE   OUT-  IMPACT     GAGE    PASSING   ROADWAY  LOAD FACTORS
LEVEL  LOAD   PUT   FACTOR   DISTANCE  DISTANCE   WIDTH     DLF    LLF   I OR F
               0     0.000      0.0       0.0      0.00    0.00  0.00      I

                      SKEW      IR
 PRINCIPAL          CORRECTION  STRESS  AASHTO
 STRESSES     DESIGN   FACTOR    LEVEL    FC
                 R      0.000     0.000


                       BRIDGE CROSS SECTION AND LOADING

                           UNIT WEIGHT                            INITIAL
 BEAM     DISTRIBUTION FACTORS    OF DECK        DEAD LOADS          P/S
SPACING  SHEAR MOMENT DEFLECTION  CONCRETE   UDLF   DL1   FWS   DL2    FORCE
  85.9   0.802  0.651  0.375       0.0000   0.0000 0.019 0.000 0.228    0.000

                                                                 RATINGS
ECCENTRICITY    P/S            LEHIGH LOSS METHOD      STRAND  w/ & w/o
MIDSPAN  END   LOSS %   XDRAPE  T0  TS  TD IC MFG IST   L or S    FWS
 0.000  7.650   0.00   0.3513   0   0   0  0  0   0       S


                            SPAN LENGTHS (SIMPLE)
                                                                      BEAM
SPAN #     1       2       3       4       5       6       7       8    PROJ
LENGTH   60.53                                                          7.000


                          EXTERIOR DIAPHRAGM DETAILS

ID  WEIGHT  THICK  #DIA  DIST   DIST   DIST   DIST   DIST   DIST   DIST   DIST
 E   0.000  12.00    3   0.00  30.27  60.53   0.00   0.00   0.00   0.00   0.00


                              PRESTRESS CRITERIA

  BEAM     SLAB     CONC     STEEL    STEEL    STEEL      INITIAL ALLOWABLE
  CONC     CONC     INIT     INIT     YIELD     ULT     COMP    TENS  DRP/DBND
  F'CB     F'CS     F'CI      FSI      Fy       F'S      FCI     FTI    FTFD
 5.500    3.500    5.000      0.0      0.0    270.0    0.000   0.000   0.000

  FINAL ALLOWABLE    ALLOW    OR              MODULAR             EST.
 COMP  TENS  SLAB    SHEAR  STRESS  STEEL      RATIOS     CREEP     %     STRAND
  FC    FT    FCS     VHA   LEVEL     E       DES  ULT    FACTOR  LOSS  DIAMETER
0.000 0.000 0.000   0.000  0.000       0   0.000 0.000     0.0    0.0   0.4375

  NUMBER OF    NUMBER OF    STIRRUP
    ROWS          Lx        DETAILS
      5            1           Y
```

PRESTRESSED CONCRETE BEAM DIMENSIONS

| TYPE | COMP | D | W1 | W2 | W3 | T1 | T2 |
|---|---|---|---|---|---|---|---|
| I | Y | 36.000 | 24.000 | 18.000 | 12.000 | 8.000 | 5.000 |

| B1 | B2 | B3 | B4 | D1 | D2 | X1 | X2 | SLAB THICK | HAUNCH |
|---|---|---|---|---|---|---|---|---|---|
| 8.00 | 6.00 | 3.00 | 3.00 | 0.00 | 0.00 | 0.000 | 0.000 | 6.75 | 0.00 |

STRAND DETAILS

| AREA | G1 | G2 | R1 | R2 | R3 | R4 | R5 | R6 | R7 | R8 | R9 | R10 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.115 | 2.00 | 2.000 | 11 | 11 | 9 | 5 | 2 | | | | | |

DEBONDED STRAND DETAILS

| DEBONDED | 1 | | 2 | | 3 | | 4 | | 5 | | 6 | | 7 | | 8 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LENGTH | ROW NO | | ROW NO | | ROW NO | | ROW NO | | ROW NO | | ROW NO | | ROW NO | | ROW NO | |
| LX | NO | STR | NO | STR | NO | STR | NO | STR | NO | STR | NO | STR | NO | STR | NO | STR |
| 2.417 | 1 | 6 | 2 | 2 | 3 | 2 | 4 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |

STIRRUP DETAILS

| SPEC. FOR ANAL/RATE | STIRRUP AREA | FSY | LOCATION | SPACING | LOCATION | SPACING | LOCATION | SPACING |
|---|---|---|---|---|---|---|---|---|
| | 0.200 | 40 | 0.00 | 6.000 | 4.00 | 12.000 | 0.00 | 0.000 |
| | | | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 |

DEFAULT VALUES

| GAGE DIST | PASS DIST | DLF | LLF | SKEW C.F. | UDLF | P/S LOSS % | UNIT WT DK CONC |
|---|---|---|---|---|---|---|---|
| 6.0 | 4.0 | 1.30 | 2.17 | 1.000 | 0.0150 | 0.04 | 0.150 |

| EXT DIA WEIGHT | ST INI FSI | ST YLD Fy | AASHTO FC | COMP FC | TENS FT | SLAB FCS | ALLOW SHR-VHA |
|---|---|---|---|---|---|---|---|
| 0.847 | 189.0 | 229.5 | N | 2.200 | 0.222 | 1.400 | 0.300 |

| OR STR LEVEL | IR STR LEVEL | STEEL E | N DES | N ULT | CREEP FACTOR | SPEC A/R |
|---|---|---|---|---|---|---|
| 0.900 | 0.800 | 28000 | 1.254 | 1.571 | 1.6 | 1979 |

```
******************************************************************************
*                         RATING OF AN INTERIOR BEAM                         *
******************************************************************************
```

BASIC BEAM SECTION PROPERTIES

| DEPTH IN | AREA IN.2 | WEIGHT LBS/FT | M OF I IN.4 | N.A. TO TOP YT IN. | N.A. TO BOT YB IN | Z TOP IN.3 | Z BOT IN.3 |
|---|---|---|---|---|---|---|---|
| 36.00 | 615.0 | 640.62 | 75255.9 | 19.83 | 16.17 | 3795.8 | 4652.9 |

COMPOSITE SECTION PROPERTIES

| SLAB WIDTH | AREA IN.2 | M OF I IN.4 | N.A. TO SLAB TOP | N.A. TO BEAM TOP | N.A. TO BEAM BOT | Z TOP SLAB | Z TOP BEAM | Z BOT BEAM |
|---|---|---|---|---|---|---|---|---|
| 85.90 | 1077.5 | 219115.7 | 16.62 | 9.87 | 26.13 | 13186.4 | 22207.3 | 8384.6 |

UNIFORM DEAD LOADS ACTING ON GIRDER (KIPS/FT)

| GIRDER WEIGHT | SLAB WEIGHT | HAUNCH WEIGHT | FORMWORK WEIGHT | INPUT DL1 | FUTURE WEARING SURFACE | INPUT DL2 | TOTAL DL1 | TOTAL DL2 |
|---|---|---|---|---|---|---|---|---|
| 0.6406 | 0.6487 | 0.0000 | 0.1074 | 0.0190 | 0.0000 | 0.2280 | 1.4157 | 0.2280 |

DEAD LOAD AND LIVE LOAD REACTIONS

| DL1 REACTION | DL2 REACTION | IMPACT FACTOR | LL+I H20 REACTION | LL+I HS20 REACTION | LL+I ML80 REACTION | LL+I TK527 REACTION |
|---|---|---|---|---|---|---|
| 44.1 | 6.9 | 1.269 | 42.5 L | 56.5 T | 59.6 T | 60.8 T |

| LL+I EV2 REACTION | LL+I EV3 REACTION |
|---|---|
| 49.0 T | 69.1 T |

PRESTRESSING FORCE (STRAND PATTERN KNOWN)

AT CL BRG.:

| INITIAL | LOSS % | EFFECTIVE | EFF NO OF STRANDS | ECCENTRICITY | C.G.S. |
|---|---|---|---|---|---|
| 565.110 | 26.17 | 417.236 | 26 | 11.097 | 5.077 |

AT DEBONDED LENGTH 2.417 FT. FROM CL BRG.:

| INITIAL | LOSS % | EFFECTIVE | EFF NO OF STRANDS | ECCENTRICITY | C.G.S. |
|---|---|---|---|---|---|
| 825.930 | 26.17 | 609.806 | 38 | 11.437 | 4.737 |

AT MID SPAN:

| INITIAL | LOSS % | EFFECTIVE | EFF NO OF STRANDS | ECCENTRICITY | C.G.S. |
|---|---|---|---|---|---|
| 825.930 | 26.17 | 609.806 | 38 | 11.437 | 4.737 |

*******************************************************************************
* RATING SUMMARY *
*******************************************************************************

FLEXURAL RATINGS (BASED ON MOMENT) — SHEAR RATINGS (1979 I)

| LOAD | | FACTOR | TONS | LOCATION FROM CL BRG | | FACTOR | TONS | LOCATION FROM CL BRG |
|---|---|---|---|---|---|---|---|---|
| H20 | IR | 1.234 | 24.68 | 30.265 | IR | 2.279 | 45.57 | 15.133 |
| | OR | 3.842 | 76.85 | 30.265 | OR | 3.804 | 76.07 | 15.133 |
| HS20 | IR | 0.862 | 31.03 | 30.265 | IR | 1.615 | 58.12 | 15.133 |

```
      OR   2.680    96.47   27.238           OR   2.695    97.02   15.133
ML80  IR   0.724    26.52   30.265           IR   1.429    52.37   15.133
      OR   2.254    82.59   30.265           OR   2.386    87.42   15.133
TK527 IR   0.761    30.42   30.265           IR   1.437    57.49   15.133
      OR   2.351    94.03   27.238           OR   2.399    95.97   15.133
EV2   IR   1.011    29.07   30.265           IR   1.863    53.56   15.133
      OR   3.124    89.82   27.238           OR   3.110    89.41   15.133
EV3   IR   0.659    28.32   30.265           IR   1.261    54.23   15.133
      OR   2.051    88.20   30.265           OR   2.105    90.52   15.133
```

```
******************************************************************************
*                             CONTROLLING RATINGS                            *
******************************************************************************


     VEHICLE TYPE                IR           OR

     H20   LOADING (TONS)    24.68 F     76.07 S
     HS20  LOADING (TONS)    31.03 F     96.47 F
     ML80  LOADING (TONS)    26.52 F     82.59 F
     TK527 LOADING (TONS)    30.42 F     94.03 F
     EV2   LOADING (TONS)    29.07 F     89.41 S
     EV3   LOADING (TONS)    28.32 F     88.20 F

     F = FLEXURAL RATING        S = SHEAR RATING
```

### 3.6 COMPOSITE PRESTRESSED SPREAD BOX BEAMS

This Section covers the rating of composite prestressed spread box beam bridges. In addition, see Section 2.5.4.1 for additional information.

#### 3.6.1 Policies and Guidelines

Most spread composite prestressed box beam bridges can be rated by analytical methods based on design plans and field measurements.

If plans are not available, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.5 for guidance on prestressed concrete, MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

#### 3.6.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating. Allowable stress design method would not have been used for these bridges.

##### 3.6.2.1 LFR or ASR Method

For bridges built prior to 2011, if load factor design methodology was used then the load rating should be performed using PS3. For bridges built prior to 2011 and designed with LRFD, LFR or LRFR can be utilized for the rating.

##### 3.6.2.2 LRFR Method

For spread composite prestressed box beam bridges designed with LRFD on or after 2011, load ratings should be performed using LRFD in PSLRFD.

#### 3.6.3 Live Load and Dead Load Distribution

##### 3.6.3.1 LFR or ASR Method

Pub. 238 Section 3.3 provides guidance on the distribution of live load to longitudinal girders. Typically, the live load distribution factor for exterior beams is in accordance with the AASHTO Standard Specification.

The typical distribution factors are as follows when AASHTO distribution factors are applicable:

| Exterior | | Interior | |
|---|---|---|---|
| Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.28.2) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.28.1)# |
| Lever Rule* | Use the larger of:<br>• $S \geq 2N_L/N_B$<br>• Lever Rule* | Lever Rule* | $\frac{2N_L}{N_B} + k\frac{S}{L}$<br>(AASHTO Std. Spec. Eq. 3-33) |

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

#For spread box beams not meeting the requirements of AASHTO 3.28.1, the live load shall be distributed as indicated in AASHTO Table 3.23.1 for prestressed concrete girders as indicated in the 1993 DM-4, Section 3.28.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2.2.3 and Pub. 238 IP 3.3.3.1 for additional discussion of skew.

Dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all girders (AASHTO Standard Specification 3.23.2.3.1.1).

Dead load from concrete end blocks is typically not added to the DL1 for box beams for a load rating since this load is assumed to be transferred directly into the bearing (Note this load should be considered for bearing and substructure design).

#### 3.6.3.2 LRFR Method

For composite prestressed spread box beam bridges designed with LRFD, distribution factors may be computed by PSLRFD. For analysis outside of the program, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.2.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO distribution factors. See DM-4 Table 4.6.2.2.3c-1 for the formulas for the skew correction factor.

### 3.6.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1941-1960 Volume 3 | Spread Box Beams (Composite) | 9/19/1960 | S-3904, S-3905 |
| Standards for Old Bridges 1961-1965 Volume 4 | Prestressed Concrete Bridge Standards | 3/8/1962 | S-3909 |
| Standards for Old Bridges 1965-1972 Vol. 5 | Instructions for Acceptance of Prestressed Beams with Cracks | 3/17/70 | P-800 |
| BD-201 March 1973 | Prestressed Concrete Beam Standards | 3/1/1973 | 4 through 22 |
| BD-600 Series, Jan. 1989 Edition | Various | 1/20/1989 | BD-652, BD-666 |
| BD-600 Series, 1989 Edition, Change 3 | Prestressed Beam Sizes and Section Properties | 9/11/1989 | BD-652 |

| | | | |
|---|---|---|---|
| BD-600 Series, July 1993 Edition | Various | 7/1/1993 | BD-652, BD-666 |
| BD-600 Series, Sept. 1994 Edition | Various | 9/30/1994 | BD-652, BD-666 |
| BD-651 and BD-662 deleted 12/24/1999; BD-652 and BD-666 deleted 6/30/2000 | | | |
| Online document not available | Box Beam Reinforcement Details | 12/24/1999 | BD-661 |
| Reference Document | Topic | Approval Date | Relevant Drawings |
| Online document not available | Prestressed Beam Sizes and Section Properties | 6/30/2000 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 1/21/2003 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 4/15/2004 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 7/20/2007 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 12/29/2008 | BD-652 |
| Online document not available | Various | 9/20/2010 | BD-652, BD-665 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 8/31/2012 | BD-652 |
| Standards for Bridge Design September 2010 Edition Change 3 | Box Beam Reinforcement Details | 11/21/2014 | BD-661 |
| Standards for Bridge Design April 2016 Edition Change 7 | Prestressed Beam Sizes and Section Properties | 4/29/2016 (Latest Revision 10/7/2024) | BD-652 |

### 3.6.5 Modeling Section Properties and Deterioration

See Section 2.5.4.1 for a discussion of modeling section loss.

See Pub. 238, Section IE 6.1.5.3I for guidance on removing strands from analysis based on various defects in prestressed beams. In addition, see Pub. 238, Appendix IE 04-D for an example of adjusting strand patterns based on this guidance. It is acceptable to apply this methodology when rating spread box beams with deterioration. For composite beams, method A would likely be overly conservative; therefore, method B would be more appropriate.

### 3.6.6 Standard Practices

For continuous prestressed concrete bridges, if it is determined that the deck steel over the pier will control the rating, the bridge may be rated as a simple span if it increases the rating. Provide documentation in the load rating report that the deck steel controlled and the ratings are based on simple span analysis.

For prestressed bridges designed using LFD, the shear capacity of the beam should be determined as per AASHTO Std. Spec. 9.20 with a shear capacity reduction factor of 0.9. If shear controls the ratings for a bridge built prior to 1992 and there is no sign of shear distress in the beams, the shear capacity reduction factor may be increased to 1.0 as per the 1979 edition of the AASHTO Std. Spec.

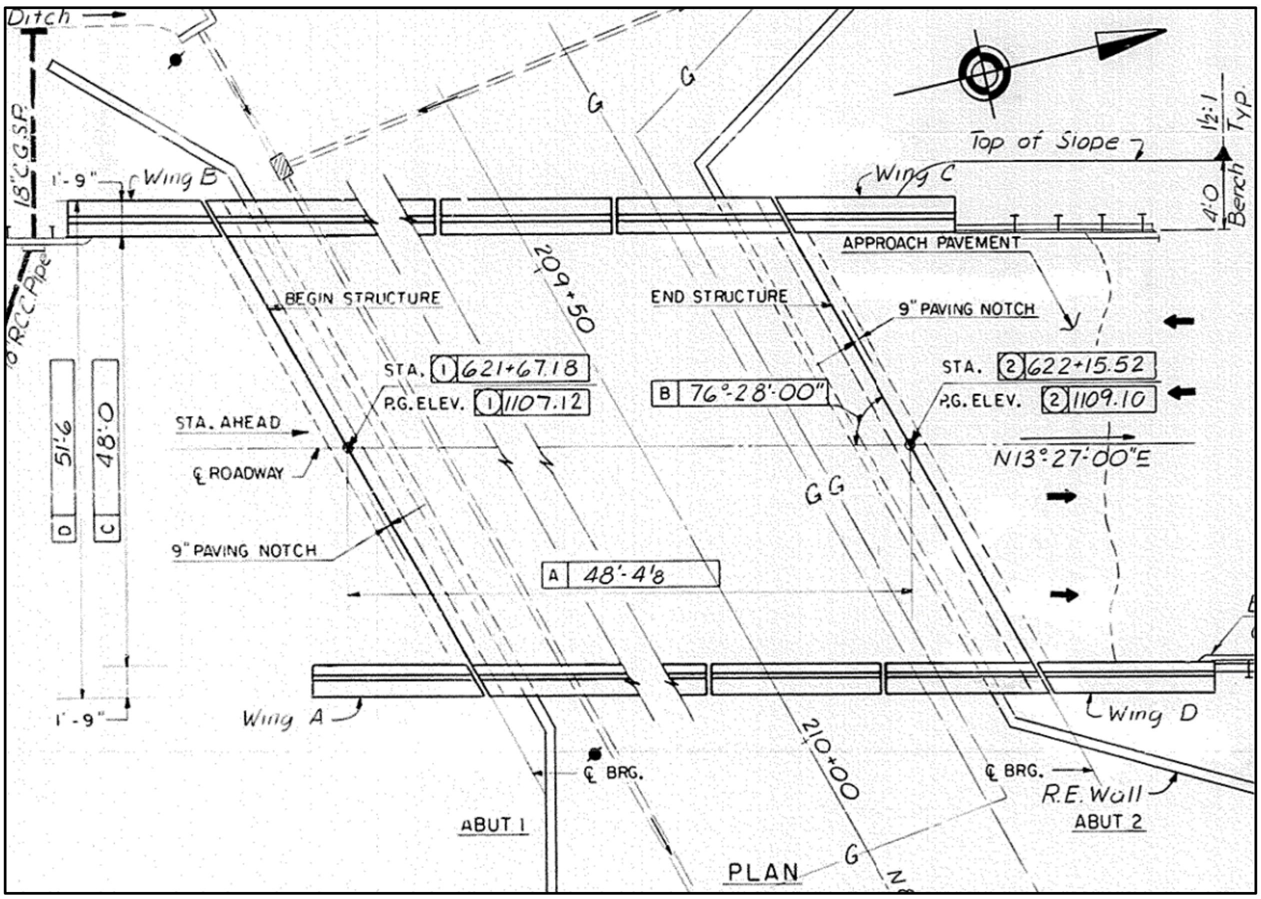
### 3.6.7 Common QA Findings

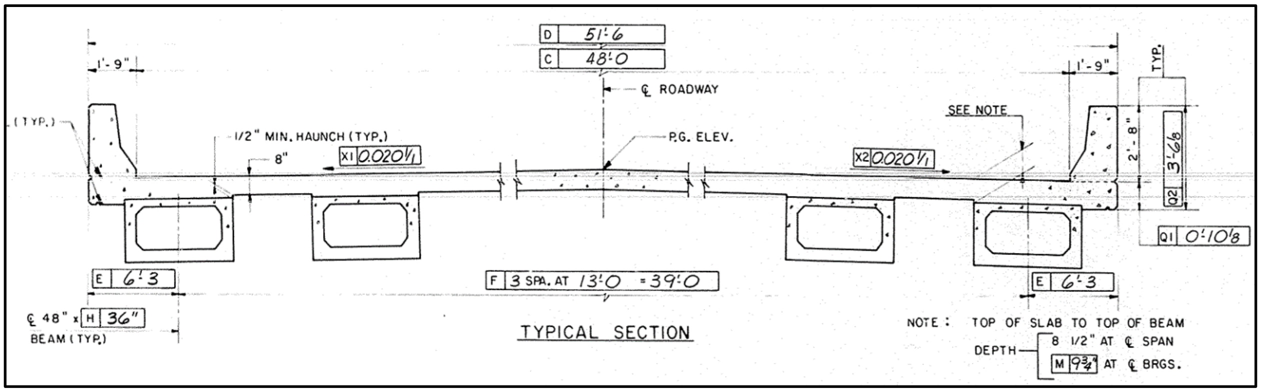
This Section is in development and will be provided in future editions of this manual.

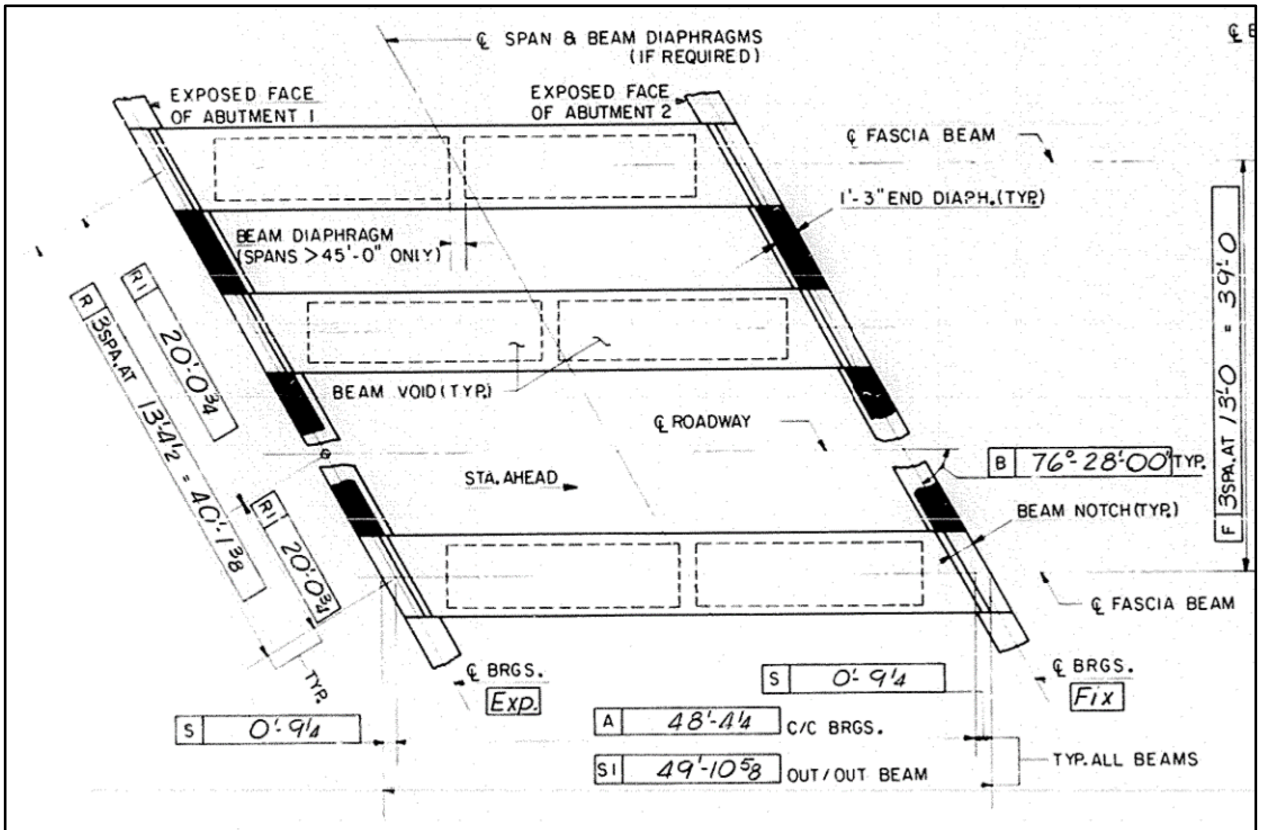
### 3.6.8 Sample Load Rating

This Section contains sample load rating calculations for an interior and exterior girder of a single span composite prestressed concrete spread box beam structure shown below. The analysis will be performed using PennDOT's LFD Prestressed Concrete Girder Design and Rating (PS3) Program, Version 3.6.0.5 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part-IV (DM-4).

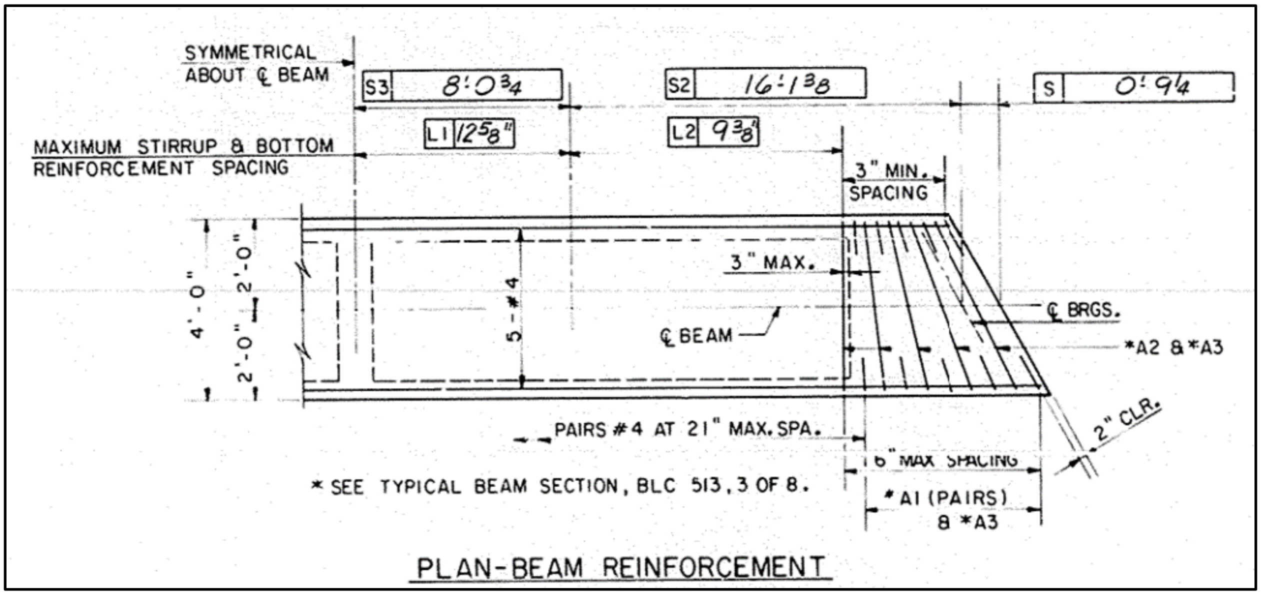
The single span superstructure consists of an 8" reinforced concrete deck supported on four (4) prestressed concrete spread box beams. The 36" deep x 48" width concrete beams are spaced at 13'-0", with deck overhangs of 6'-3", and are composite with the concrete deck.

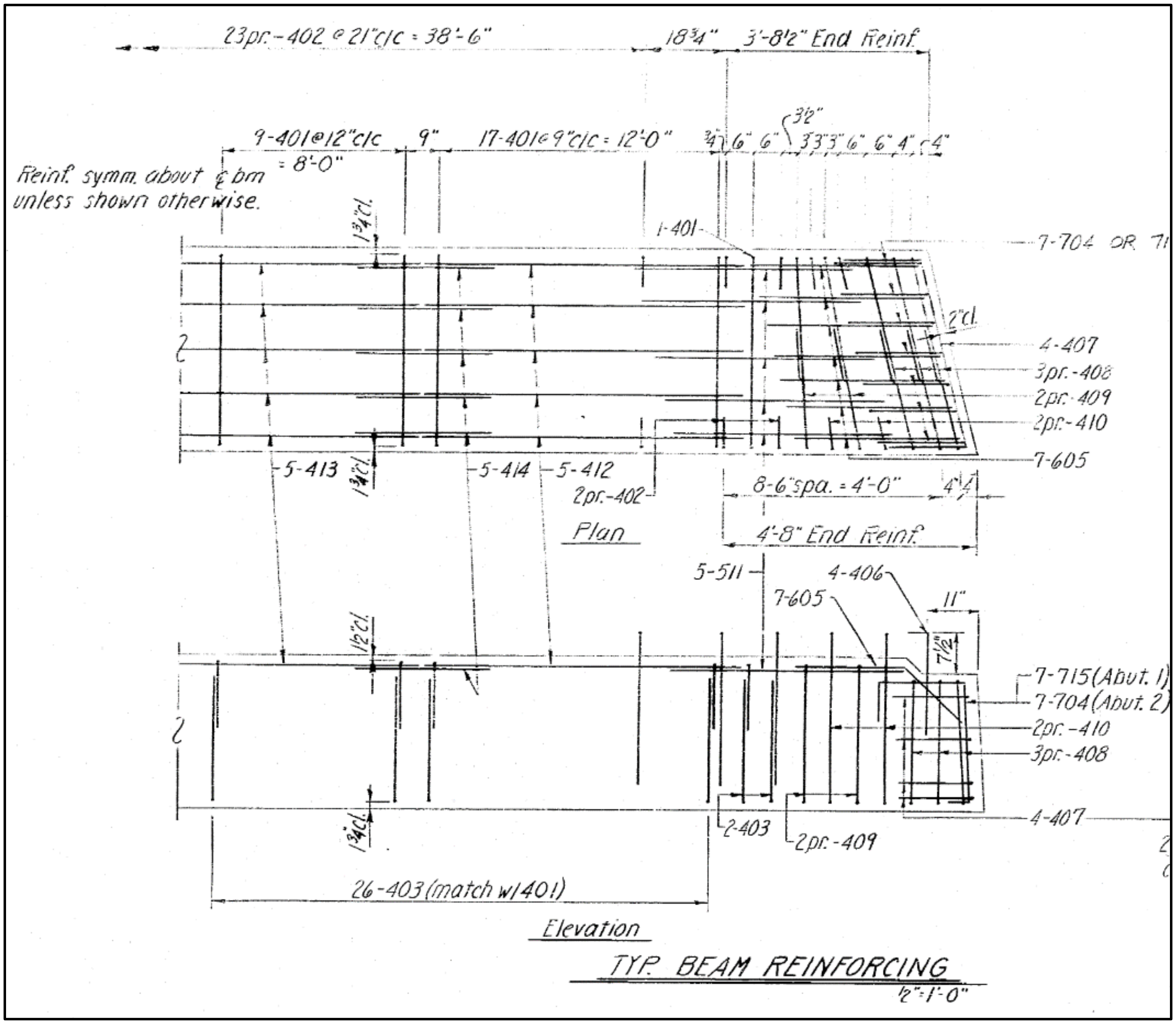




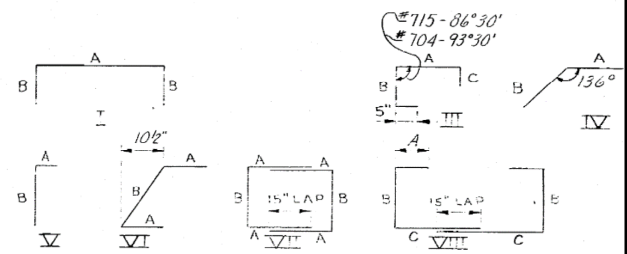

FRAMING PLAN

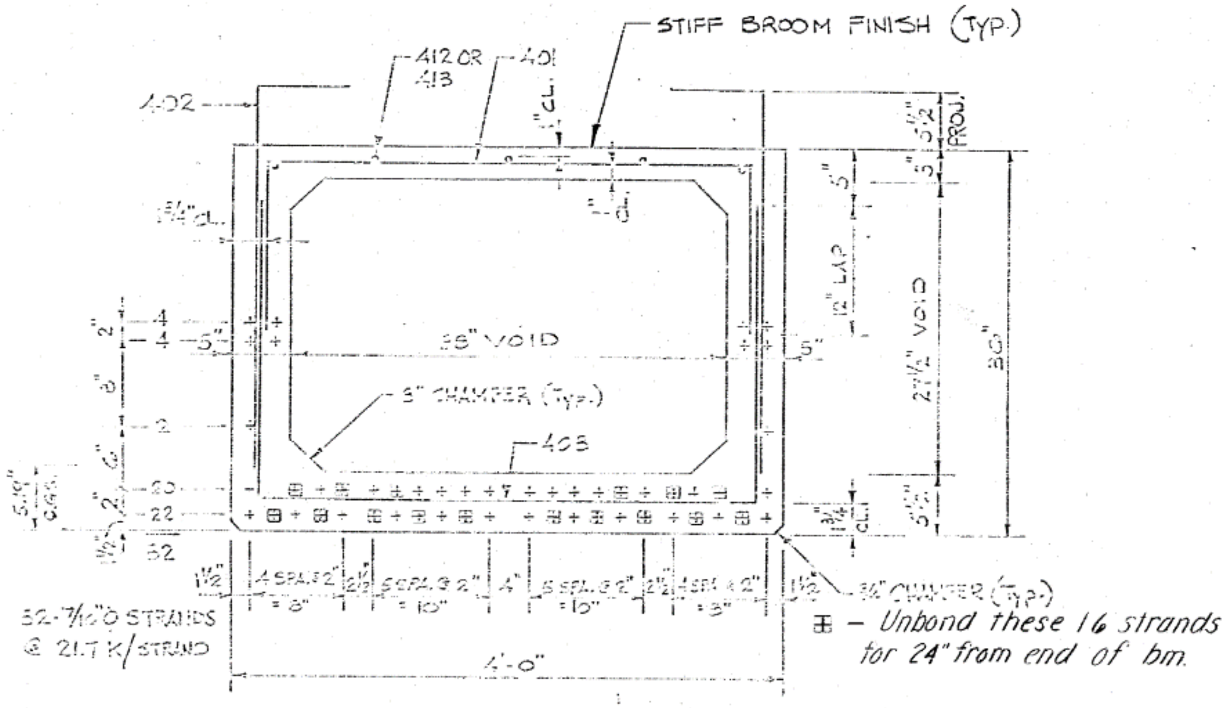

SYMMETRICAL ABOUT ℄ BEAM
S3 8'-0¾
S2 16'-1⅜
S 0'-9¼
L1 12⅝"
L2 9⅜"
MAXIMUM STIRRUP & BOTTOM REINFORCEMENT SPACING
3" MIN. SPACING
3" MAX.
4'-0"
2'-0"
2'-0"
5-#4
℄ BEAM
℄ BRGS.
*A2 & *A3
PAIRS #4 AT 21" MAX. SPA.
6" MAX SPACING
2" CLR.
* SEE TYPICAL BEAM SECTION, BLC 513, 3 OF 8.
*A1 (PAIRS)
8 *A3
PLAN-BEAM REINFORCEMENT


23pr.-402 @ 21"c/c = 38'-6"
18¾"
3'-8½" End Reinf.
9-401@12"c/c = 8'-0"
9"
17-401@9"c/c = 12'-0"
Reinf. symm. about ℄ bm unless shown otherwise.
1-401
7-704 OR 7
2"cl.
4-407
3pr.-408
2pr.-409
2pr.-410
7-605
5-413
5-414
5-412
2pr.-402
Plan
8-6"spa. = 4'-0"
4'-8" End Reinf.
5-511
4-406
7-605
11"
7-715(Abut. 1)
7-704(Abut. 2)
2pr.-410
3pr.-408
4-407
2-403
2pr.-409
26-403(match w/401)
Elevation
TYP. BEAM REINFORCING
½"=1'-0"

Grade 60 **REBAR SCHEDULE** 36"x4'-0"

| MARK | NO. PER BEAM | | | | SIZE # | LGTH | TYPE | DESCRIPTION | | | TOTAL LBS. REINF. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Typ. | | TOTAL | | | | A | B | C | |
| 401 | | 53 | | 212 | 4 | 6'-3½" | I | 44½" | 15½" | | |
| 402 | | 27pr. | | 108 | 4 | 3'-8" | V | 8" | 36" | | |
| 403 | | 55 | | 220 | 4 | 8'-5" | I | 44½" | 28¼" | | |
| 704 | | 7 | | 28 | 7 | 5'-4¼" | III | 18½" | 28¼" | 12½" | |
| 605 | | 14 | | 56 | 6 | 3'-6" | IV | 24" | 18" | | |
| 406 | | 8 | | 32 | 4 | 2'-4" | V | 4" | 24" | | |
| 407 | | 8 | | 32 | 4 | 6'-8¾" | VI | 18" | 44¾" | | |
| 408 | | 6pr. | | 24pr. | 4 | 7'-7¼" | VII | 31" | 29¼" | | |
| 409 | | 4pr. | | 16pr. | 4 | 7'-10¾" | VII | 31" | 32¾" | | |
| 410 | | 4pr. | | 16pr. | 4 | 6'-6¾" | VIII | 8" | 39¾" | 31" | |
| 511 | | 10 | | 40 | 5 | 5'-0" | Str. | | | | |
| 412 | | 5 | | 20 | 4 | 12'-7" | Str. | | | | |
| 413 | | 5 | | 20 | 4 | 30'-0" | Str. | | | | |
| 414 | | 5 | | 20 | 4 | 5'-0" | Str. | | | | |
| 715 | | 7 | | 28 | 7 | 5'-4¼" | III | 18½" | 28¼" | 12½" | |




TYP. SECTION THRU BEAMS
1" = 1'-0"

**PRESTRESSING DATA**

| | | | |
|---|---|---|---|
| PRESTRESSED BEAM SIZE 48 x | H | 36" | |
| INITIAL PRESTRESSING FORCE PER BEAM (Pi) | I | 1099 | KIPS |
| CONCRETE STRENGTH AT STRAND RELEASE (f'ci) | J1 | 5500 | PSI |
| CONCRETE STRENGTH AT 28 DAYS (f'c) | J2 | 6500 | PSI |

USE ONLY LOW RELAXATION STRANDS

## GENERAL NOTES

- DESIGN LIVE LOAD – HS25 Loading, Mod. Military Loading (2 axles of 30K ea @ 4'-0" c/c) & 204K Permit Loading.
- PRESTRESSING UNITS – 7/16" ϕ Armco Low - Relaxation strand
- ULTIMATE STRENGTH OF 7/16" ϕ STRAND – 270,000 P.S.I. OR 31 K. PER STRAND
- INITIAL PRESTRESSING – 183,900 P.S.I. OR 21.7 K. PER STRAND.
- CONCRETE CYLINDER STRENGTH – 6,500 P.S.I. AT 28 DAYS.
- CONCRETE STRENGTH AT RELEASE · $f'ci$ – 5,500 P.S.I.
- ALL MATERIALS AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH P.D.H Publication 408/83 & the Contract Special Provisions.
- ALL REINFORCEMENT TO BE INTERMEDIATE GRADE STEEL w/ $f_s$ – 24,000 P.S.I.

### 3.6.8.1 Load Rating Summary Form Including Section Loss – Method A

LOAD RATING SUMMARY FORM

Done By: ABC Date:

Checked By: XYZ Date:

| | | | |
|---|---|---|---|
| Structure ID (5A01): | 02-0008-0440-0000 | Inspection Date (7A01): | 10/13/2017 |
| Facility Carried (5A08): | William Finn Highway | | |
| Feature Intersected (5A07): | Bakerstown Road | | |
| Structure Type (6A26 - 6A29): | Single Span P/S Spread Box Beam | | |
| Spans / Members Analyzed: | Span 1/Interior Beam 3 | | |
| Analysis Method: | LFD | | |
| PennDOT Program / Version: | PS3 Version 3.6.0.5 | | |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 1.66 | 33.3 | 3.40 | 68.1 | 2.72 | 54.4 | * | * | M | V |
| HS20 | 1.20 | 43.4 | 2.39 | 86.3 | 1.91 | 69.0 | * | * | M | V |
| ML80 | 0.97 | 35.5 | 2.06 | 75.6 | 1.64 | 60.4 | * | * | M | M |
| TK527 | 1.03 | 41.2 | 2.14 | 85.9 | 1.71 | 68.7 | * | * | M | V |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 1.37 | 39.6 | 2.69 | 77.4 | 2.15 | 61.9 | * | * | M | V |
| EV3 | 0.89 | 38.3 | 1.83 | 78.7 | 1.46 | 62.9 | * | * | M | V |

Comments/Assumptions*:

Superstructure and substructure condition ratings are both 4. Therefore, for an ADTT > 500,the SLC Factor equals 0.80 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is Beam 3 in Span 1.

PS3 analysis includes 6 strands removed based on Method A analysis per Pub 238, IE 6.1.5.3I and also includes 4 psf epoxy overlay.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

3.6.8.2 Interior Beam Load Rating Analysis – Method A

3.6.8.2.1 PS3 Input Parameters

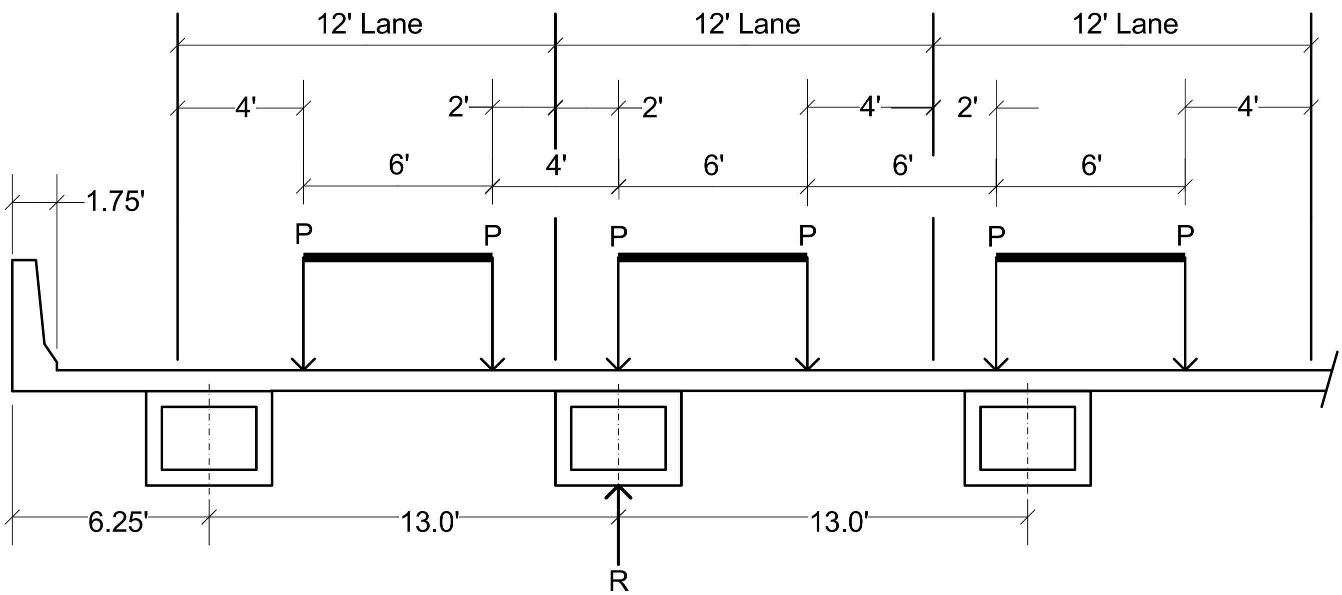
Many of the PS3 input parameters can be left blank. Only the required input values are discussed below. Refer to the PS3 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02000804400000
- Description = INTERIOR – METHOD A
- Live Load = Blank for H, HS, ML80, TK527, EV2, and EV3 vehicles.
- Output = 0 for normal output
- Roadway Width = 48.00 ft
- I or F = I for Interior Beam
- Design = R for rating only
- Skew Correction Factor = Blank for Interior Beam

Bridge Cross Section and Loading

- Beam Spacing, S = 13'-0" = 156 in per Contract Drawings
- Distribution Factor – Shear
  Per AASHTO 3.23.1, the interior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the three lane condition shown below, the reduction in load intensity factor is 0.90.




2 Lanes Loaded, $DF_V$ = P[1 + (13' - 6')/13' + (13' – 4')/13' + (13' – 10')/13'] x 1.00 = 2.462 wheels = 2.462 wheels / (2 wheels/axle) = 1.231 axles (Controls)

3 Lanes Loaded, $DF_V$ = P[1 + (13' - 6')/13' + (13' – 4')/13' + (13' – 10')/13' + (13' – 12')/13'] x 0.90 = 2.285 wheels = 2.285 wheels / (2 wheels/axle) = 1.142 axles

- Distribution Factor – Moment

  Per AASHTO Table 3.23.1, the interior beam moment distribution factor for concrete decks on prestressed concrete spread box beams is calculated per AASHTO 3.28.

  Per Equation 3-33 of AASHTO 3.28.1, the moment distribution factor is equal to

$$DF_M = \frac{2N_L}{N_B} + k\frac{S}{L}$$

  where,

  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
  Roadway Width, W = 51.50 ft – 2 x 1.75 ft = 48.0 ft
  $N_L$ = 48.0 ft / 12 ft = 4.0 ==> Use 4 lanes

  Number of Beams, $N_B$ = 4

  k Factor, k = 0.07W – $N_L$(0.10$N_L$ – 0.26) - 0.20$N_B$ – 0.12 per AASHTO Equation 3-34
  k = 0.07 x 48 ft – 4 x (0.10 x 4 – 0.26) – 0.20 x 4 – 0.12 = 1.88

  Beam Spacing, S = 13.0 ft

  Span Length, L = 48'-4 1/8" = 48.34 ft

  $DF_M$ = 2 x 4 lanes / 4 beams + 1.88 x 13 ft / 48.34 ft = 2.506 wheels x (1 axle/2 wheels) = 1.253 axles

  The Range of Applicability checks per AASHTO 3.28.1:

| | |
|---|---|
| $4 \le N_b \le 10$ | $N_b$ = 4, OK |
| 6.57 ft ≤ S ≤ 11.00 ft | S = 13.0 ft, NG |
| 32 ft ≤ W ≤ 66 ft | W = 48.0 ft, OK |

  Per DM-4 3.28, the distribution factor from AASHTO 3.28 shall only be used if all the conditions are met. Since the 13.00 ft spacing exceeds the upper limit of 11.00 ft, DM-4 3.28 indicates to use the same distribution factor as specified in AASHTO's Table 3.23.1 for prestressed concrete girders. Per AASHTO's Table 3.23.1, the distribution factor for moment in an interior prestressed concrete girder is S/5.5.

  S/5.5 = 13.00 ft / 5.5 = 2.364 wheels = 2.364 wheels / (2 wheels/axle) = 1.182 axles

- Distribution Factor – Deflection
  Per AASHTO 8.13.2, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Reduction Factor = 0.75 for 4 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 4 lanes x 0.75 / 4 beams = 0.750 lanes/beam

- Uniform Dead Loads – UDLF

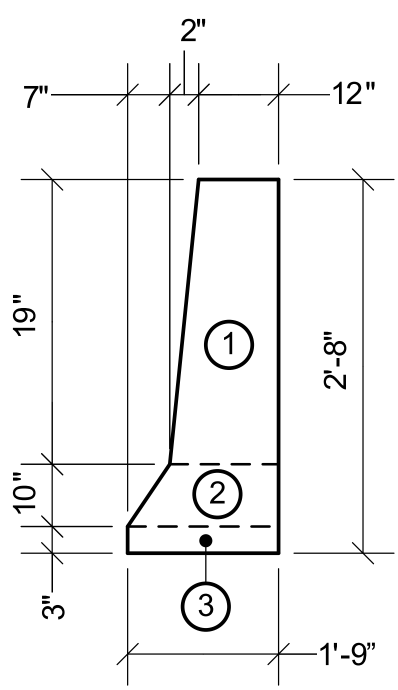
UDLF = Blank. PS3 will internally calculate the weight of the stay-in-place formwork using 0.015 ksf for spread box beams. The weight of the concrete haunch is assumed to be included in the UDLF calculated by the program since the UDLF is multiplied by the beam spacing specified by the user. Alternatively, the user can calculate and enter the UDLF based on the equations in the PS3 Manual.

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight, deck slab weight, SIP forms, haunch concrete, and diaphragms. Girder self-weight, deck slab weight, and diaphragm weight are calculated by PS3. For this analysis run, PS3 assumed one interior diaphragm located at midspan. The SIP form weight and concrete haunch weight are calculated using the UDLF loading input. Therefore, there isn't any hardware attached to the beam.
  DL1 = 0 kips/ft or blank.

- Future Wearing Surface – FWS
  Future wearing surface is not typically included in the rating analysis.
  FWS = 0 kips/ft or blank.

- Dead Loads – DL2
  The dead loads acting on the composite section include outside barriers. Additionally, an epoxy overlay was placed on the structure. Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

Roadway Barriers




Area 1 = (12 in + 14 in)/2 x 19 in = 247 in$^2$
Area 2 = (14 in + 21 in)/2 x 10 in = 175 in$^2$
Area 3 = 21 in x 3 in = 63 in$^2$
Total Area = 485 in$^2$

Barrier Weight = 485 in$^2$ / 144 in$^2$/ft$^2$ x 0.150 kcf = 0.505 kips/ft

Epoxy Overlay
An epoxy overlay weighing 4 psf was placed on the structure.
Overlay Weight = 0.004 ksf x 48 ft = 0.192 kips/ft

Total Dead Load 2 = (2 x 0.505 kips/ft + 0.192 kips/ft) / 4 beams = 0.301 kips/ft

- Strand L or S = L for Low-Relaxation Strands per the notes on the Contract Drawings

Span Lengths

- Span 1 Length = 48'-4 1/8" = 48.34 ft
- Beam Projection = 9.25 in

Diaphragm Details

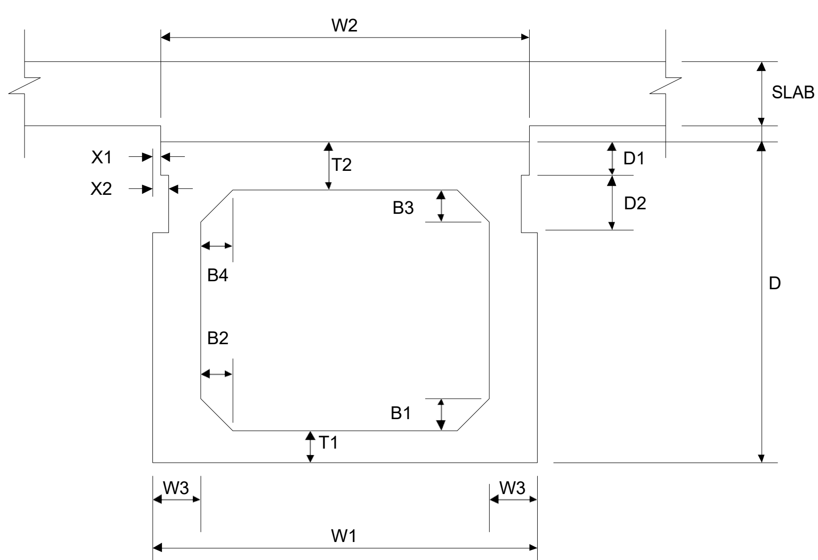
- Identify = E for Exterior Diaphragms (Note: Interior diaphragms assumed by PS3)
- Thickness = 15.00 in
- Number of Diaphragms = 2
- Distance 1 = 0 ft from centerline of left bearing
- Distance 2 = 48.34 ft from centerline of left bearing

Prestress Criteria

- 28-Day Compressive Strength of Beam Concrete = 6.5 ksi per the Contract Drawings
- 28-Day Compressive Strength of Slab Concrete = 4.0 ksi for Class AAA per the Contract Drawings
- Compressive Strength of Beam Concrete at Initial Prestressing = 5.5 ksi per the Contract Drawings
- Ultimate Strength of Prestressing Steel = 270 ksi
- Strand Diameter = 7/16" = 0.4375 in
- Number of Strand Rows = 10
- Number of Debonded Lengths = 1
- Stirrup Details = Y indicating stirrup reinforcement will be entered

Prestressed Concrete Beam Dimensions

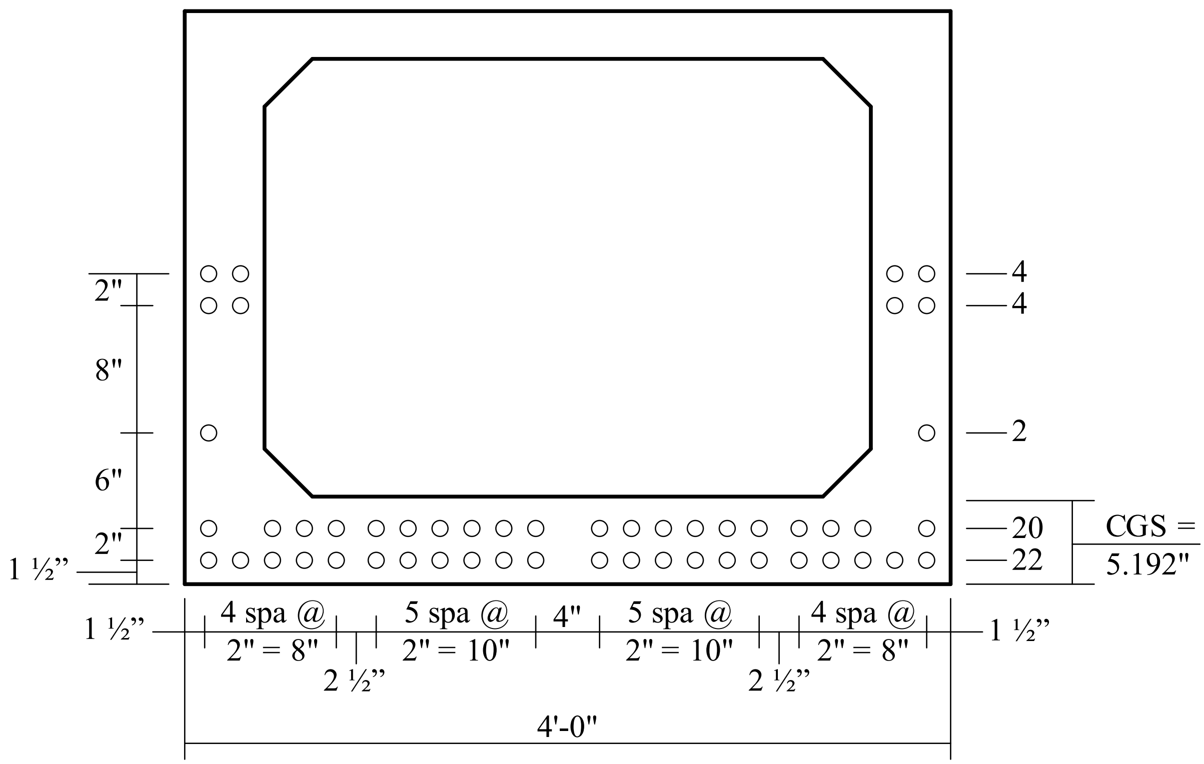
- Type = B for a box with rectangular void
- Comp = Y for a composite beam
- Design or D = 48/36
- Slab Thickness = 7.5 in (effective)
- Haunch Thickness = 0 in (Haunch may be included in the analysis if the actual height was measured in the field.)



Box Beam – Rectangular Void

Strand Details
A field inspection identified spalling at the bottom flange on both sides of an interior beam. Exposed strands have been observed in the middle 1/3 of the span for an interior beam. The concrete adjacent to and above the exposed strands has been found to be sound concrete. Per Section 2.5.4.1 of this manual, refer to Pub 238 IE Article 6.1.5.3I to evaluate box girders with section loss and deterioration. The conservative simplified Method A will be used to check the load ratings.

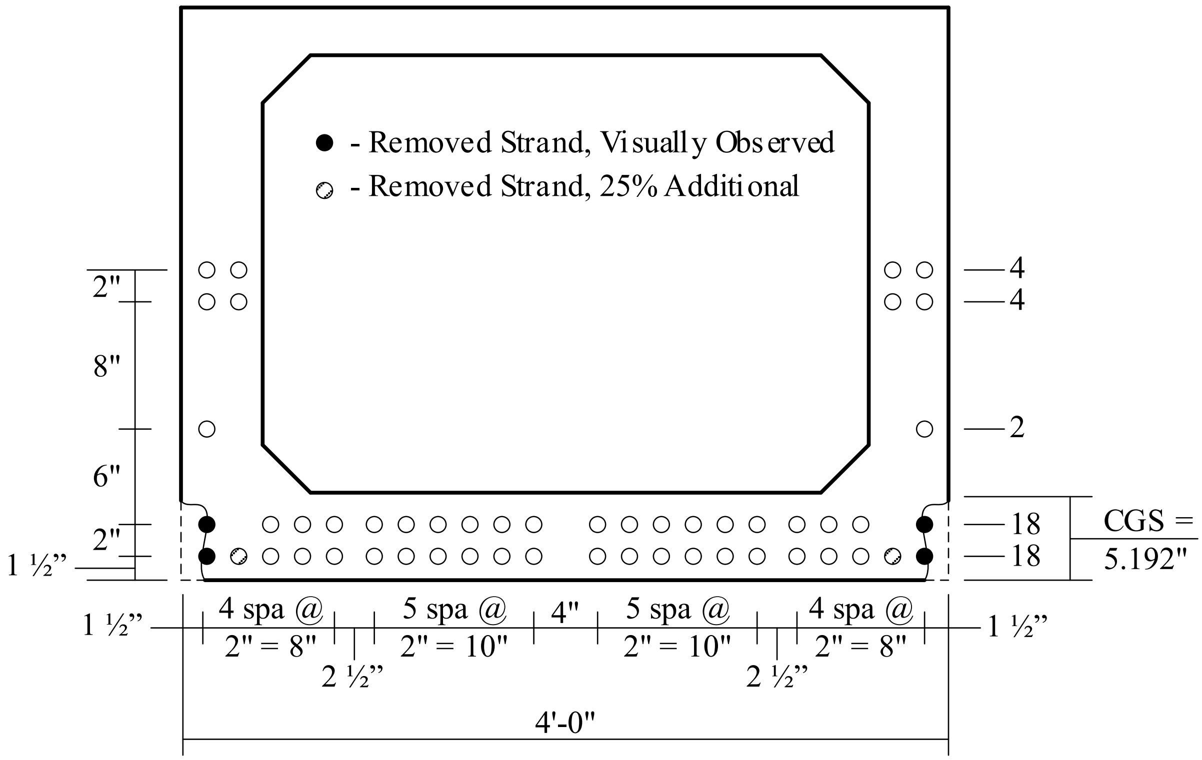
Perform a load rating analysis using Method A, including section loss of reinforcement, for the interior beam. Since the exposed strands are in the middle 1/3 of the span, they will be considered ineffective for the full beam lengths. The beam section with strands from the existing structure are shown below.




**Original Beam**

| Row | # of Strands | Distance from Bottom of Beam | # Strands x Distance |
|---|---|---|---|
| 1 | 22 | 1.5” | 33.0” |
| 2 | 20 | 3.5” | 70.0” |
| 3 | 2 | 9.5” | 19.0” |
| 4 | 4 | 17.5” | 70.0” |
| 5 | 4 | 19.5” | 78.0” |
| Total = | 52 | CGS = | 5.192” |

The beam section with strands removed per the Method A analysis are shown below. Since the outside strands are visually exposed, they are removed from the analysis. To meet the additional 25% criteria per Publication 238, Section IE 6.1.5.3I, the adjacent strands are also neglected. This results in a 50% reduction which is greater than 25% required.




| Method A | | | |
|---|---|---|---|
| Row | # of Strands | Distance from Bottom of Beam | # Strands x Distance |
| 1 | 18 | 1.5" | 27.0" |
| 2 | 18 | 3.5" | 63.0" |
| 3 | 2 | 9.5" | 19.0" |
| 4 | 4 | 17.5" | 70.0" |
| 5 | 4 | 19.5" | 78.0" |
| Total = | 46 | CGS = | 5.587" |
| | | | |

- Strand Area = 0.115 in$^2$
- G1 = 1.50 in, G2 = 2.00 in
- R1 = R2 = 18
- R3 = R4 = R6 = R7 = R8 = 0
- R5 = 2
- R9 = R10 = 4

Debonded Strand Details

- Debonded Length, $L_x$ = 24 in – 9.25 in = 14.75 in = 1.229 ft from centerline of bearing
- Row 1 – 8 Strands Debonded, Row 2 – 6 Strands Debonded

Stirrup Details

- Spec = Blank for shear values to be computed per 1979 AASHTO Interim Specifications
- Stirrup Area = 0.20 in$^2$ for one leg of a #4 stirrup
- Yield Strength of Reinforcement Stirrups, $f_y$ = 60 ksi

- Location = 0.00 ft
- Spacing = 6.0 in (Note: On skewed ends, the average stirrup spacing can be utilized. The maximum spacing of 6" at the beam ends is used in this example to be conservative.)
- Location = 4.00 ft
- Spacing = 9.0 in
- Location = 48.344 ft / 2 – 8.000 ft = 16.17 ft
- Spacing = 12.0 in

**TITLE**

| | | | |
|---|---|---|---|
| Structure ID: | 02-0008-0440-0000 | | |
| Description: | INTERIOR - METHOD A | | |
| SLC Level: | | | |
| Live Load: | | | H20, HS20, ML80, TK527, EV2, and EV3 |
| Output: | 0 | | Normal output |
| Impact Factor: | | | |
| Gage Distance: | | ft | |
| Passing Distance: | | ft | |
| Roadway Width: | 48.00 | ft | Per Contract Drawings |
| DLF: | | | |
| LLF: | | | |
| I or F: | I | | Interior beam |
| Principal: | | | |
| Design: | R | | Rating only problem |
| Skew Correction Factor: | | | NA for Interior Beam |
| IR Stress Level: | | | |
| AASHTO fc: | | | |

**Bridge Cross-Section and Loading**

| | | | |
|---|---|---|---|
| Beam Spacing: | 156.000 | in | Per Contract Drawings |
| Distr Factor -Shear: | 1.231 | | See Hand Calcs |
| Distr Factor -Moment: | 1.182 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.750 | | See Hand Calcs |
| Unit Weight of Deck Concrete: | | | |
| UDLF: | 0.000 | | See Hand Calcs |
| DL1: | 0.000 | kip/ft | See Hand Calcs |
| FWS: | 0.000 | kip/ft | See Hand Calcs |
| DL2: | 0.301 | kip/ft | See Hand Calcs |
| Initial P/S Force: | | kips | |
| Midspan Eccentricity: | | in | |
| End Eccentricity: | | in | |
| P/S Loss %: | | | |
| Drape Point: | -- | | Decimal part of the span |
| Strand L or S: | L | | Low Relaxation Strands |
| Rate FWS?: | | | |

**Span Lengths**

| | | | |
|---|---|---|---|
| Span Length 1: | 48.34 | ft | Per Contract Drawings |
| Beam Projection: | 9.250 | in | Per Contract Drawings |

**Diaphragm Details**

| | | | |
|---|---|---|---|
| Identification: | E | | Per Contract Drawings |
| Thickness: | 15.00 | in | Per Contract Drawings |
| No. of Diaphragms: | 2 | | Per Contract Drawings |
| Distance 1: | 0.00 | ft | Per Contract Drawings |
| Distance 2: | 48.34 | ft | Per Contract Drawings |
| Distance 3: | | ft | |
| Distance 4: | | ft | |

**Prestress Criteria**

| | | | |
|---|---|---|---|
| Beam Conc $f'_{cb}$: | 6.5 | ksi | Per Contract Drawings |
| Slab Conc $f'_{cs}$: | 4.0 | ksi | Per Contract Drawings |
| Conc Init $f'_{ci}$: | 5.5 | ksi | Per Contract Drawings |
| Strand Ult $f'_{s}$: | 270 | ksi | Per Contract Drawings |
| Strand Diameter: | 0.4375 | in | Per Contract Drawings |
| No. of Rows: | 10 | | Per Contract Drawings |
| No. Lx: | 1 | | Per Contract Drawings |
| St. Det: | Y | | Stirrup Details |

**Beam Dimensions**

| | | | |
|---|---|---|---|
| Beam Type: | B | | Box Beam w/ Rectangular Void |
| Composite: | Y | | Composite |
| Designation or D: | 48.36 | | Per Contract Drawings |
| W1: | | in | |
| W2: | | in | |
| W3: | | in | |
| T1: | | in | |
| T2: | | in | |
| B1: | | in | |
| B2: | | in | |
| B3: | | in | |
| B4: | | in | |
| D1: | | in | |
| D2: | | in | |
| X1: | | in | |
| X2: | | in | |
| Slab Thickness: | 7.50 | in | Per Contract Drawings |
| Haunch: | 0 | in | |

**Strands**

| | | | |
|---|---|---|---|
| Strand Area = | 0.115 | $in^2$ | Per Contract Drawings |
| G1: | 1.500 | in | Per Contract Drawings |
| G2: | 2.000 | in | Per Contract Drawings |
| R1: | 18 | | Per Contract Drawings |
| R2: | 18 | | Per Contract Drawings |
| R3: | 0 | | Per Contract Drawings |
| R4: | 0 | | Per Contract Drawings |
| R5: | 2 | | Per Contract Drawings |
| R6: | 0 | | Per Contract Drawings |
| R7: | 0 | | Per Contract Drawings |
| R8: | 0 | | Per Contract Drawings |
| R9: | 4 | | Per Contract Drawings |
| R10: | 4 | | Per Contract Drawings |
| R11: | | | |
| R12: | | | |
| R13: | | | |
| R14: | | | |
| R15: | | | |
| R16: | | | |
| R17: | | | |
| R18: | | | |
| R19: | | | |
| R20: | | | |
| R21: | | | |
| R22: | | | |
| R23: | | | |
| R24: | | | |
| R25: | | | |
| R26: | | | |
| R27: | | | |
| R28: | | | |

**Debonded**

| | | | |
|---|---|---|---|
| Debonded Length= | 1.229 | ft | See Hand Calcs |
| R1: | 1 | | |
| No. Strands: | 10 | | |
| R2: | 2 | | |
| No. Strands: | 6 | | |
| R3: | | | |
| No. Strands: | | | |
| R4: | | | |
| No. Strands: | | | |

**Stirrups**

| | | | |
|---|---|---|---|
| Spec: | | | |
| Area: | 0.200 | $in^2$ | Per Contract Drawings |
| $f'_y$: | 60 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 6.000 | in | Per Contract Drawings |
| Location 2: | 4.00 | ft | Per Contract Drawings |
| Spacing 2: | 9.000 | in | Per Contract Drawings |
| Location 3: | 16.17 | ft | Per Contract Drawings |
| Spacing 3: | 12.000 | in | Per Contract Drawings |
| Location 4: | | ft | |
| Spacing 4: | | in | |
| Location 5: | | ft | |
| Spacing 5: | | in | |

### 3.6.8.2.2 PS3 Output

```
*****************************************************************************
*                                                                           *
*          LFD Prestressed Concrete Girder Design and Rating        330740 *
*                                                                           *
*                       COPYRIGHT (C) 1996-2024                             *
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*                     DEPARTMENT OF TRANSPORTATION                          *
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```

```
            LFD Prestressed Concrete Girder Design and Rating         330740

PROGRAM P4353050                                                06/11/2024 12:03
VERSION 3.6.0.5           LAST UPDATED 12/22/2023        DOCUMENTATION 12/2023

INPUT: ... PS Spread Box Beam PS3 Analysis - Method A.dat

ANALYSIS INCLUDES 3/8" EPOXY OVERLAY (4 PSF)



STRUCTURE ID - 02000804400000 - INTERIOR - METHOD A

 SLC   LIVE   OUT-  IMPACT     GAGE    PASSING   ROADWAY  LOAD FACTORS
LEVEL  LOAD   PUT   FACTOR   DISTANCE  DISTANCE   WIDTH     DLF    LLF   I OR F
               0     0.000      0.0       0.0     48.00    0.00  0.00      I

                      SKEW      IR
 PRINCIPAL          CORRECTION  STRESS  AASHTO
 STRESSES     DESIGN   FACTOR    LEVEL     FC
                 R      0.000     0.000


                       BRIDGE CROSS SECTION AND LOADING

                                  UNIT WEIGHT                             INITIAL
 BEAM     DISTRIBUTION FACTORS     OF DECK         DEAD LOADS               P/S
SPACING  SHEAR MOMENT DEFLECTION  CONCRETE    UDLF   DL1    FWS   DL2      FORCE
 156.0   1.231  1.182  0.750       0.0000   0.0000 0.000 0.000 0.301     0.000

                                                                      RATINGS
ECCENTRICITY    P/S              LEHIGH LOSS METHOD     STRAND   w/ & w/o
MIDSPAN  END   LOSS %   XDRAPE  T0  TS  TD IC MFG IST   L or S     FWS
 0.000  0.000    0.00   0.0000   0   0   0  0  0   0       L


                          SPAN LENGTHS (SIMPLE)
                                                                          BEAM
SPAN #     1       2       3       4       5       6       7       8      PROJ
LENGTH   48.34                                                            9.250


                       EXTERIOR DIAPHRAGM DETAILS

ID  WEIGHT  THICK  #DIA  DIST   DIST   DIST   DIST   DIST   DIST   DIST   DIST
 E   0.000  15.00    2   0.00  48.34   0.00   0.00   0.00   0.00   0.00   0.00


                           PRESTRESS CRITERIA

  BEAM     SLAB     CONC     STEEL    STEEL    STEEL      INITIAL ALLOWABLE
  CONC     CONC     INIT     INIT     YIELD     ULT      COMP    TENS  DRP/DBND
  F'CB     F'CS     F'CI      FSI      Fy       F'S       FCI     FTI     FTFD
 6.500    4.000    5.500      0.0      0.0    270.0     0.000   0.000   0.000

  FINAL ALLOWABLE    ALLOW    OR               MODULAR            EST.
 COMP  TENS  SLAB    SHEAR  STRESS  STEEL      RATIOS     CREEP    %      STRAND
  FC    FT    FCS     VHA   LEVEL     E       DES  ULT   FACTOR  LOSS  DIAMETER
0.000 0.000 0.000   0.000  0.000        0  0.000 0.000    0.0    0.0   0.4375

  NUMBER OF    NUMBER OF    STIRRUP
    ROWS          Lx        DETAILS
     10            1           Y
```

```
                          PRESTRESSED CONCRETE BEAM DIMENSIONS

 TYPE    COMP   DESIGNATION    D         W1        W2        W3        T1        T2
   B       Y        48/36    36.000   48.000    48.000     5.000     5.500     3.000

                                                                      SLAB
    B1      B2      B3      B4      D1       D2       X1       X2     THICK  HAUNCH
   3.00    3.00    3.00    3.00    0.00     0.00    0.000    0.000    7.50    0.00


                                     STRAND DETAILS

  AREA    G1     G2      R1    R2    R3    R4    R5    R6    R7    R8    R9    R10
  0.115  1.50  2.000    18    18     0     0     2     0     0     0     4     4


                                DEBONDED STRAND DETAILS

DEBONDED     1         2         3         4         5         6         7         8
 LENGTH  ROW NO    ROW NO    ROW NO    ROW NO    ROW NO    ROW NO    ROW NO    ROW NO
   LX    NO STR    NO STR    NO STR    NO STR    NO STR    NO STR    NO STR    NO STR
  1.229   1  8      2  6      0  0      0  0      0  0      0  0      0  0      0  0


                                    STIRRUP DETAILS
SPEC. FOR  STIRRUP
ANAL/RATE   AREA    FSY  LOCATION SPACING   LOCATION SPACING   LOCATION SPACING
            0.200   60     0.00    6.000       4.00    9.000     16.17   12.000
                           0.00    0.000       0.00    0.000      0.00    0.000


                                     DEFAULT VALUES

      GAGE     PASS                        SKEW                 P/S    UNIT WT
      DIST     DIST      DLF       LLF     C.F.       UDLF    LOSS %  DK CONC
       6.0      4.0     1.30      2.17    1.000     0.0150     0.04    0.150

   INT DIA  INT DIA  EXT DIA   ST INI   ST YLD   AASHTO     COMP      TENS
    THICK    WEIGHT   WEIGHT    FSI      Fy       FC         FC        FT
      10.0    0.891    2.953   189.0    229.5      N       2.600     0.242

      SLAB   ALLOW    OR STR   IR STR    STEEL                         CREEP
       FCS  SHR-VHA    LEVEL    LEVEL      E      N DES    N ULT    FACTOR
     1.600    0.300    0.900    0.800    28000    1.275    1.625       1.6

      SPEC
       A/R
      1979

    ONE INTERIOR DIAPHRAGM IS ASSUMED AT MIDSPAN



******************************************************************************
*                          RATING OF AN INTERIOR BEAM                        *
******************************************************************************


                           BASIC BEAM SECTION PROPERTIES
```

| DEPTH IN | AREA IN.2 | WEIGHT LBS/FT | M OF I IN.4 | N.A. TO TOP YT IN. | N.A. TO BOT YB IN | Z TOP IN.3 | Z BOT IN.3 |
|---|---|---|---|---|---|---|---|
| 36.00 | 701.0 | 762.49 | 119746.7 | 19.83 | 16.17 | 6038.3 | 7406.1 |

COMPOSITE SECTION PROPERTIES

| SLAB WIDTH | AREA IN.2 | M OF I IN.4 | N.A. TO SLAB TOP | N.A. TO BEAM TOP | N.A. TO BEAM BOT | Z TOP SLAB | Z TOP BEAM | Z BOT BEAM |
|---|---|---|---|---|---|---|---|---|
| 138.00 | 1512.9 | 332747.7 | 14.68 | 7.18 | 28.82 | 22672.6 | 46368.1 | 11544.2 |

UNIFORM DEAD LOADS ACTING ON GIRDER (KIPS/FT)

| GIRDER WEIGHT | SLAB WEIGHT | HAUNCH WEIGHT | FORMWORK WEIGHT | INPUT DL1 | FUTURE WEARING SURFACE | INPUT DL2 | TOTAL DL1 | TOTAL DL2 |
|---|---|---|---|---|---|---|---|---|
| 0.7625 | 1.3000 | 0.0000 | 0.1950 | 0.0000 | 0.0000 | 0.3010 | 2.2575 | 0.3010 |

DEAD LOAD AND LIVE LOAD REACTIONS

| DL1 REACTION | DL2 REACTION | IMPACT FACTOR | LL+I H20 REACTION | LL+I HS20 REACTION | LL+I ML80 REACTION | LL+I TK527 REACTION |
|---|---|---|---|---|---|---|
| 58.0 | 7.3 | 1.288 | 64.8 L | 90.5 T | 100.7 T | 100.8 T |

| LL+I EV2 REACTION | LL+I EV3 REACTION |
|---|---|
| 78.3 T | 114.7 T |

PRESTRESSING FORCE (STRAND PATTERN KNOWN)

AT CL BRG.:
-----------

| INITIAL | LOSS % | EFFECTIVE | EFF NO OF STRANDS | ECCENTRICITY | C.G.S. |
|---|---|---|---|---|---|
| 695.520 | 25.07 | 521.172 | 32 | 9.169 | 7.000 |

AT DEBONDED LENGTH 1.229 FT. FROM CL BRG.:
------------------------------------------

| INITIAL | LOSS % | EFFECTIVE | EFF NO OF STRANDS | ECCENTRICITY | C.G.S. |
|---|---|---|---|---|---|
| 999.810 | 25.07 | 749.185 | 46 | 10.582 | 5.587 |

AT MID SPAN:
------------

| INITIAL | LOSS % | EFFECTIVE | EFF NO OF STRANDS | ECCENTRICITY | C.G.S. |
|---|---|---|---|---|---|
| 999.810 | 25.07 | 749.185 | 46 | 10.582 | 5.587 |

```
******************************************************************************
*                              RATING SUMMARY                                *
******************************************************************************
```

FLEXURAL RATINGS (BASED ON MOMENT)          SHEAR RATINGS (1979 I)

| LOAD | | FACTOR | TONS | LOCATION FROM CL BRG | | FACTOR | TONS | LOCATION FROM CL BRG |
|---|---|---|---|---|---|---|---|---|
| H20 | IR | 1.665 | 33.30 | 24.170 | IR | 2.042 | 40.84 | 12.085 |
| | OR | 3.545 | 70.91 | 24.170 | OR | 3.409 | 68.17 | 12.085 |
| HS20 | IR | 1.206 | 43.42 | 24.170 | IR | 1.436 | 51.70 | 12.085 |
| | OR | 2.548 | 91.71 | 21.753 | OR | 2.397 | 86.30 | 12.085 |
| ML80 | IR | 0.970 | 35.53 | 24.170 | IR | 1.241 | 45.47 | 12.085 |
| | OR | 2.064 | 75.64 | 24.170 | OR | 2.072 | 75.91 | 12.085 |
| TK527 | IR | 1.031 | 41.24 | 24.170 | IR | 1.287 | 51.47 | 12.085 |
| | OR | 2.195 | 87.81 | 24.170 | OR | 2.148 | 85.92 | 12.085 |
| EV2 | IR | 1.378 | 39.63 | 21.753 | IR | 1.614 | 46.41 | 12.085 |
| | OR | 2.898 | 83.31 | 21.753 | OR | 2.694 | 77.46 | 12.085 |
| EV3 | IR | 0.893 | 38.38 | 24.170 | IR | 1.097 | 47.17 | 12.085 |
| | OR | 1.900 | 81.72 | 24.170 | OR | 1.831 | 78.73 | 12.085 |

```
******************************************************************************
*                             CONTROLLING RATINGS                            *
******************************************************************************
```

| VEHICLE TYPE | IR | OR |
|---|---|---|
| H20 LOADING (TONS) | 33.30 F | 68.17 S |
| HS20 LOADING (TONS) | 43.42 F | 86.30 S |
| ML80 LOADING (TONS) | 35.53 F | 75.64 F |
| TK527 LOADING (TONS) | 41.24 F | 85.92 S |
| EV2 LOADING (TONS) | 39.63 F | 77.46 S |
| EV3 LOADING (TONS) | 38.38 F | 78.73 S |

F = FLEXURAL RATING        S = SHEAR RATING

3.6.8.3 Exterior Beam Load Rating Analysis

3.6.8.3.1 PS3 Input Parameters

A typical exterior girder was selected and will be rated using PS3. Many of the PS3 input parameters can be left blank. Only the required input values are discussed below. Refer to the PS3 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02000804400000
- Description = EXTERIOR BEAM
- Live Load = Blank for H, HS, ML80, TK527, EV2, and EV3 vehicles.
- Output = 0 for normal output
- Roadway Width = 48.00 ft
- I or F = F for Fascia Beam
- Design = R for rating only
- Skew Correction Factor = 1.037 (See below). Per DM-4 Table 3.23.2(A), shear in the exterior beam at the obtuse corner of the bridge shall be adjusted when the line of support is skewed.

  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

$$\text{SCF} = 1.0 + \frac{\sqrt{\frac{Ld}{12.0}}}{6S}\tan\ \theta$$

where,

Span Length, L = 48'-4 1/8" = 48.34 ft

Beam Depth, d = 36 in

Beam Spacing, S = 13.0 ft

AASHTO Skew Angle, θ = (90° – 76° 28' 00") = 13.533°

The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

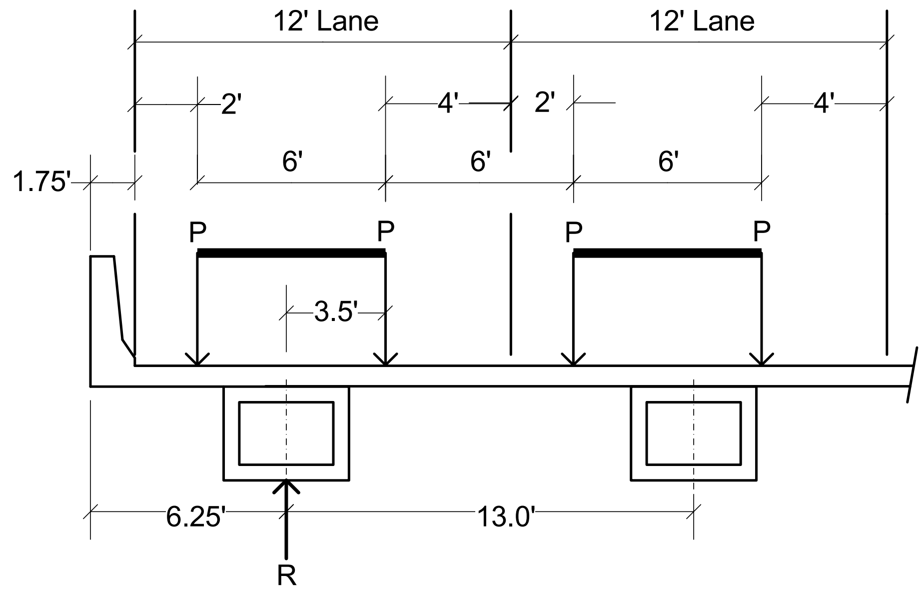
| | |
|---|---|
| $0° \leq \theta \leq 60°$ | $\theta$ = 13.533°, OK |
| 6.0 ft ≤ S ≤ 11.5 ft | S = 13.0 ft, NG…Call OK |
| 20 ft ≤ L ≤ 140 ft | L = 48.34 ft, OK |
| 18 in ≤ d ≤ 66 in | d = 36 in, OK |
| $N_b \geq 3$ | $N_b$ = 4, OK |

Bridge Cross Section and Loading

- Beam Spacing = 6.25 ft Overhang + 13.0 ft Spacing / 2 = 12.75 ft = 153 in per Contract Drawings

- Distribution Factor – Shear

Per AASHTO 3.23.1, the exterior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the two-lane condition shown below, the reduction in load intensity factor is 1.00.




$DF_V$ = P[(13’ + 6.25’ – 1.75’ – 2’)/13’ + (13’ – 3.5’)/13’ + (13’ – 9.5’)/13’] x 1.00 = 2.192 wheels

$DF_V$ = 2.192 wheels / (2 wheels/axle) = <u>1.096 axles</u>

- Distribution Factor – Moment

  Per AASHTO 3.28.2, the exterior beam distribution factor for moment is based on the Lever Rule but shall not be less than $2N_L/N_B$.

  $2N_L/N_B$ = 2 x 4 / 4 beams = 2.0 wheels / (2 wheels/axle) = 1.000 axles

  $DF_M$ = <u>1.096 axles</u>

- Distribution Factor – Deflection
  Per AASHTO 10.6.4, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Reduction Factor = 0.75 for 4 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 4 lanes x 0.75 / 4 beams = 0.750 lanes/beam

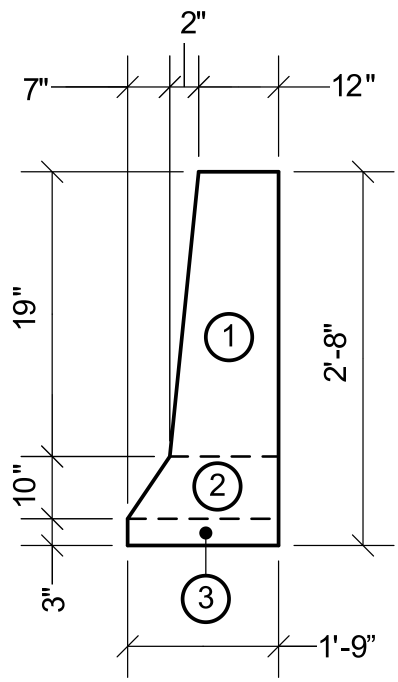
- Uniform Dead Loads – UDLF
  UDLF = Blank. PS3 will internally calculate the weight of the stay-in-place formwork using 0.015 ksf for spread box beams. The weight of the concrete haunch is assumed to be included in the UDLF calculated by the program since the UDLF is multiplied by the beam spacing specified by the user. Alternatively, the user can calculate and enter the UDLF based on the equations in the PS3 Manual.

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight, deck slab weight, SIP forms, haunch concrete, and diaphragms. Girder self-weight, deck slab weight, and diaphragm weight are calculated by PS3. For this analysis run, PS3 assumed one interior diaphragm located at midspan. The SIP form weight and concrete haunch weight are calculated using the UDLF loading input. Therefore, there isn't any hardware attached to the beam.
  DL1 = 0 kips/ft or blank.

- Future Wearing Surface – FWS
  Future wearing surface is not typically included in the rating analysis.
  FWS = 0 kips/ft or blank.

- Dead Loads – DL2
  The dead loads acting on the composite section include outside barriers. Additionally, an epoxy overlay was placed on the structure. Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  Roadway Barriers




Area 1 = (12 in + 14 in)/2 x 19 in = 247 $in^2$
Area 2 = (14 in + 21 in)/2 x 10 in = 175 $in^2$
Area 3 = 21 in x 3 in = 63 $in^2$
Total Area = 485 $in^2$

Barrier Weight = 485 $in^2$ / 144 $in^2/ft^2$ x 0.150 kcf = 0.505 kips/ft

  Epoxy Overlay
  An epoxy overlay weighing 4 psf was placed on the structure.
  Overlay Weight = 0.004 ksf x 48 ft = 0.192 kips/ft
  Total Dead Load 2 = (2 x 0.505 kips/ft + 0.192 kips/ft) / 4 beams = 0.301 kips/ft

- Strand L or S = L for Low-Relaxation Strands per the notes on the Contract Drawings

Span Lengths

- Span 1 Length = 48'-4 1/8" = 48.34 ft
- Beam Projection = 9.25 in

Diaphragm Details

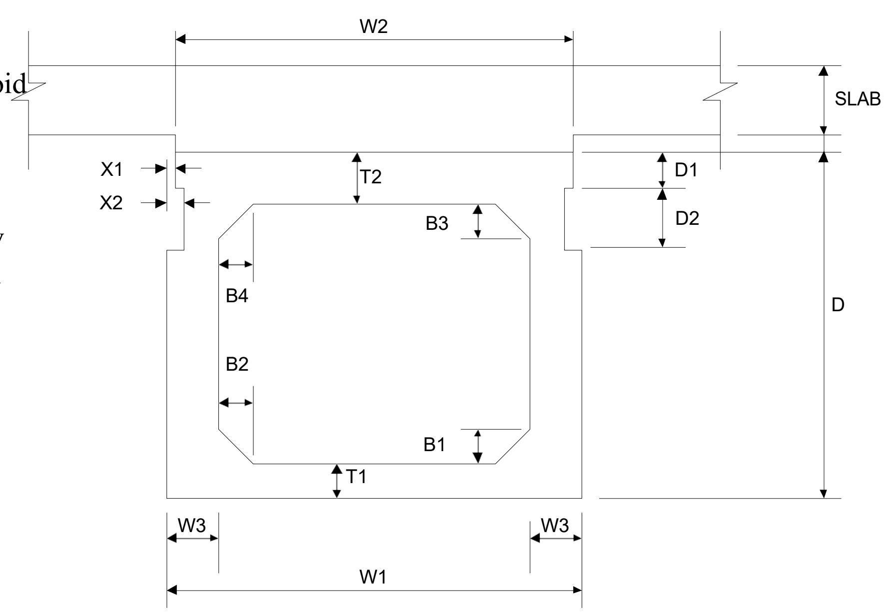
- Identify = E for Exterior Diaphragms (Note: Interior diaphragms assumed by PS3)
- Thickness = 15.00 in
- Number of Diaphragms = 2
- Distance 1 = 0 ft from centerline of left bearing
- Distance 2 = 48.34 ft from centerline of left bearing

Prestress Criteria

- 28-Day Compressive Strength of Beam Concrete = 6.5 ksi per the Contract Drawings
- 28-Day Compressive Strength of Slab Concrete = 4.0 ksi for Class AAA per the Contract Drawings
- Compressive Strength of Beam Concrete at Initial Prestressing = 5.5 ksi per the Contract Drawings
- Ultimate Strength of Prestressing Steel = 270 ksi
- Strand Diameter = 7/16" = 0.4375 in
- Number of Strand Rows = 10
- Number of Debonded Lengths = 1
- Stirrup Details = Y indicating stirrup reinforcement will be entered

Prestressed Concrete Beam Dimensions

- Type = B for a box with rectangular void
- Comp = Y for a composite beam
- Design or D = 48/36
- Slab Thickness = 7.5 in (effective)
- Haunch Thickness = 0 in (Haunch may be included in the analysis if the actual height was measured in the field.)



Box Beam – Rectangular Void

Strand Details

- Strand Area = 0.115 in$^2$
- G1 = 1.50 in
- G2 = 2.00 in
- R1 = 22
- R2 = 20
- R3 = R4 = R6 = R7 = R8 = 0
- R5 = 2
- R9 = R10 = 4

Debonded Strand Details

- Debonded Length, $L_x$ = 24 in – 9.25 in = 14.75 in = 1.229 ft from centerline of bearing
- Row 1 – 10 Strands Debonded
- Row 2 – 6 Strands Debonded

Stirrup Details

- Spec = Blank for shear values to be computed per 1979 AASHTO Interim Specifications
- Stirrup Area = 0.20 in$^2$ for one leg of a #4 stirrup
- Yield Strength of Reinforcement Stirrups, $f_y$ = 60 ksi
- Location = 0.00 ft
- Spacing = 6.0 in (Note: On skewed ends, the average stirrup spacing can be utilized. The maximum spacing of 6" at the beam ends is used in this example to be conservative.)
- Location = 4.00 ft
- Spacing = 9.0 in
- Location = 48.344 ft / 2 – 8.000 ft = 16.17 ft
- Spacing = 12.0 in

**TITLE**

| | | | |
|---|---|---|---|
| Structure ID: | 02-0008-0440-0000 | | |
| Description: | EXTERIOR BEAM | | |
| SLC Level: | | | |
| Live Load: | | | H20, HS20, ML80, TK527, EV2, and EV3 |
| Output: | 0 | | Normal output |
| Impact Factor: | | | |
| Gage Distance: | | ft | |
| Passing Distance: | | ft | |
| Roadway Width: | 48.00 | ft | Per Contract Drawings |
| DLF: | | | |
| LLF: | | | |
| I or F: | F | | Fascia beam |
| Principal: | | | |
| Design: | R | | Rating only problem |
| Skew Correction Factor: | 1.037 | | See Hand Calcs |
| IR Stress Level: | | | |
| AASHTO fc: | | | |

**Bridge Cross-Section and Loading**

| | | | |
|---|---|---|---|
| Beam Spacing: | 153.0 | in | Per Contract Drawings |
| Distr Factor -Shear: | 1.096 | | See Hand Calcs |
| Distr Factor -Moment: | 1.096 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.750 | | See Hand Calcs |
| Unit Weight of Deck Concrete: | | | |
| UDLF: | 0.000 | | See Hand Calcs |
| DL1: | 0.000 | kip/ft | See Hand Calcs |
| FWS: | 0.000 | kip/ft | See Hand Calcs |
| DL2: | 0.301 | kip/ft | See Hand Calcs |
| Initial P/S Force: | | kips | |
| Midspan Eccentricity: | | in | |
| End Eccentricity: | | in | |
| P/S Loss %: | | | |
| Drape Point: | -- | | Decimal part of the span |
| Strand L or S: | L | | Low Relaxation Strands |
| Rate FWS?: | | | |

**Span Lengths**

| | | | |
|---|---|---|---|
| Span Length 1: | 48.34 | ft | Per Contract Drawings |
| Beam Projection: | 9.25 | in | Per Contract Drawings |

**Diaphragm Details**

| | | | |
|---|---|---|---|
| Identification: | E | | Per Contract Drawings |
| Thickness: | 15.00 | in | Per Contract Drawings |
| No. of Diaphragms: | 2 | | Per Contract Drawings |
| Distance 1: | 0.00 | ft | Per Contract Drawings |
| Distance 2: | 48.34 | ft | Per Contract Drawings |
| Distance 3: | | ft | |
| Distance 4: | | ft | |

**Prestress Criteria**

| | | | |
|---|---|---|---|
| Beam Conc $f'_{cb}$: | 6.5 | ksi | Per Contract Drawings |
| Slab Conc $f'_{cs}$: | 4.0 | ksi | Per Contract Drawings |
| Conc Init $f'_{ci}$: | 5.5 | ksi | Per Contract Drawings |
| Strand Ult $f'_{s}$: | 270 | ksi | Per Contract Drawings |
| Strand Diameter: | 0.4375 | in | Per Contract Drawings |
| No. of Rows: | 10 | | Per Contract Drawings |
| No. Lx: | 1 | | Per Contract Drawings |
| St. Det: | Y | | Stirrup Details |

**Beam Dimensions**

| | | | |
|---|---|---|---|
| Beam Type: | B | | Box Beam w/ Rectangular Void |
| Composite: | Y | | Composite |
| Designation or D: | 48.36 | | Per Contract Drawings |
| W1: | | in | |
| W2: | | in | |
| W3: | | in | |
| T1: | | in | |
| T2: | | in | |
| B1: | | in | |
| B2: | | in | |
| B3: | | in | |
| B4: | | in | |
| D1: | | in | |
| D2: | | in | |
| X1: | | in | |
| X2: | | in | |
| Slab Thickness: | 7.50 | in | Per Contract Drawings |
| Haunch: | 0 | in | |

**Strands**

| | | | |
|---|---|---|---|
| Strand Area = | 0.115 | $in^2$ | Per Contract Drawings |
| G1: | 1.500 | in | Per Contract Drawings |
| G2: | 2.000 | in | Per Contract Drawings |
| R1: | 22 | | Per Contract Drawings |
| R2: | 20 | | Per Contract Drawings |
| R3: | 0 | | Per Contract Drawings |
| R4: | 0 | | Per Contract Drawings |
| R5: | 2 | | Per Contract Drawings |
| R6: | 0 | | Per Contract Drawings |
| R7: | 0 | | Per Contract Drawings |
| R8: | 0 | | Per Contract Drawings |
| R9: | 4 | | Per Contract Drawings |
| R10: | 4 | | Per Contract Drawings |
| R11: | | | |
| R12: | | | |
| R13: | | | |
| R14: | | | |
| R15: | | | |
| R16: | | | |
| R17: | | | |
| R18: | | | |
| R19: | | | |
| R20: | | | |
| R21: | | | |
| R22: | | | |
| R23: | | | |
| R24: | | | |
| R25: | | | |
| R26: | | | |
| R27: | | | |
| R28: | | | |

**Debonded**

| | | | |
|---|---|---|---|
| Debonded Length= | 1.229 | ft | See Hand Calcs |
| R1: | 1 | | |
| No. Strands: | 10 | | |
| R2: | 2 | | |
| No. Strands: | 6 | | |
| R3: | | | |
| No. Strands: | | | |
| R4: | | | |
| No. Strands: | | | |

**Stirrups**

| | | | |
|---|---|---|---|
| Spec: | | | |
| Area: | 0.200 | $in^2$ | Per Contract Drawings |
| $f'_y$: | 60 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 6.000 | in | Per Contract Drawings |
| Location 2: | 4.00 | ft | Per Contract Drawings |
| Spacing 2: | 9.000 | in | Per Contract Drawings |
| Location 3: | 16.17 | ft | Per Contract Drawings |
| Spacing 3: | 12.000 | in | Per Contract Drawings |
| Location 4: | | ft | |
| Spacing 4: | | in | |
| Location 5: | | ft | |
| Spacing 5: | | in | |

### 3.6.8.3.2 PS3 Output

```
******************************************************************************
*                                                                            *
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*                                                                            *
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            LFD Prestressed Concrete Girder Design and Rating          330740

PROGRAM P4353050                                                 06/11/2024 12:03
VERSION 3.6.0.5          LAST UPDATED 12/22/2023        DOCUMENTATION 12/2023

INPUT: Exterior PS Spread Box Beam PS3 Analysis.dat

ANALYSIS INCLUDES 3/8" EPOXY OVERLAY (4 PSF)



STRUCTURE ID - 02000804400000 - EXTERIOR BEAM

 SLC   LIVE   OUT-  IMPACT     GAGE     PASSING   ROADWAY  LOAD FACTORS
LEVEL  LOAD   PUT   FACTOR   DISTANCE  DISTANCE   WIDTH      DLF   LLF   I OR F
               0     0.000      0.0       0.0     48.00     0.00  0.00      F

                         SKEW      IR
 PRINCIPAL            CORRECTION  STRESS  AASHTO
 STRESSES     DESIGN    FACTOR    LEVEL     FC
                 R      1.037     0.000


                       BRIDGE CROSS SECTION AND LOADING

                                  UNIT WEIGHT                           INITIAL
 BEAM     DISTRIBUTION FACTORS    OF DECK         DEAD LOADS              P/S
SPACING  SHEAR MOMENT DEFLECTION  CONCRETE   UDLF   DL1   FWS   DL2     FORCE
 153.0   1.019  1.019  0.750       0.0000   0.0000 0.000 0.000 0.301    0.000

                                                                  RATINGS
ECCENTRICITY    P/S              LEHIGH LOSS METHOD     STRAND   w/ & w/o
MIDSPAN  END   LOSS %   XDRAPE  T0  TS  TD IC MFG IST   L or S     FWS
 0.000  0.000    0.00   0.0000   0   0   0  0  0   0       L


                           SPAN LENGTHS (SIMPLE)
                                                                     BEAM
SPAN #     1       2       3       4       5       6       7       8     PROJ
LENGTH   48.34                                                           9.250


                         EXTERIOR DIAPHRAGM DETAILS

ID  WEIGHT  THICK  #DIA  DIST   DIST   DIST   DIST   DIST   DIST   DIST   DIST
 E   0.000  15.00    2   0.00  48.34   0.00   0.00   0.00   0.00   0.00   0.00


                             PRESTRESS CRITERIA

  BEAM    SLAB    CONC    STEEL   STEEL   STEEL      INITIAL ALLOWABLE
  CONC    CONC    INIT    INIT    YIELD    ULT      COMP   TENS  DRP/DBND
  F'CB    F'CS    F'CI    FSI      Fy      F'S       FCI    FTI    FTFD
 6.500   4.000   5.500     0.0     0.0   270.0     0.000  0.000   0.000

  FINAL ALLOWABLE   ALLOW    OR              MODULAR           EST.
 COMP  TENS  SLAB   SHEAR  STRESS  STEEL      RATIOS    CREEP    %     STRAND
  FC    FT   FCS     VHA   LEVEL     E      DES  ULT   FACTOR  LOSS  DIAMETER
0.000 0.000 0.000   0.000  0.000       0  0.000 0.000    0.0    0.0   0.4375

  NUMBER OF    NUMBER OF    STIRRUP
    ROWS          Lx        DETAILS
     10           1            Y
```

PRESTRESSED CONCRETE BEAM DIMENSIONS

| TYPE | COMP | DESIGNATION | D | W1 | W2 | W3 | T1 | T2 |
|---|---|---|---|---|---|---|---|---|
| B | Y | 48/36 | 36.000 | 48.000 | 48.000 | 5.000 | 5.500 | 3.000 |

| B1 | B2 | B3 | B4 | D1 | D2 | X1 | X2 | SLAB THICK | HAUNCH |
|---|---|---|---|---|---|---|---|---|---|
| 3.00 | 3.00 | 3.00 | 3.00 | 0.00 | 0.00 | 0.000 | 0.000 | 7.50 | 0.00 |

STRAND DETAILS

| AREA | G1 | G2 | R1 | R2 | R3 | R4 | R5 | R6 | R7 | R8 | R9 | R10 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.115 | 1.50 | 2.000 | 22 | 20 | 0 | 0 | 2 | 0 | 0 | 0 | 4 | 4 |

DEBONDED STRAND DETAILS

| DEBONDED LENGTH LX | 1 ROW NO NO STR | 2 ROW NO NO STR | 3 ROW NO NO STR | 4 ROW NO NO STR | 5 ROW NO NO STR | 6 ROW NO NO STR | 7 ROW NO NO STR | 8 ROW NO NO STR |
|---|---|---|---|---|---|---|---|---|
| 1.229 | 1 10 | 2 6 | 0 0 | 0 0 | 0 0 | 0 0 | 0 0 | 0 0 |

STIRRUP DETAILS

| SPEC. FOR ANAL/RATE | STIRRUP AREA | FSY | LOCATION | SPACING | LOCATION | SPACING | LOCATION | SPACING |
|---|---|---|---|---|---|---|---|---|
| | 0.200 | 60 | 0.00 | 6.000 | 4.00 | 9.000 | 16.17 | 12.000 |
| | | | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 |

DEFAULT VALUES

| GAGE DIST | PASS DIST | DLF | LLF | UDLF | P/S LOSS % | UNIT WT DK CONC | INT DIA THICK |
|---|---|---|---|---|---|---|---|
| 6.0 | 4.0 | 1.30 | 2.17 | 0.0150 | 0.04 | 0.150 | 10.0 |

| INT DIA WEIGHT | EXT DIA WEIGHT | ST INI FSI | ST YLD Fy | AASHTO FC | COMP FC | TENS FT | SLAB FCS |
|---|---|---|---|---|---|---|---|
| 0.891 | 2.871 | 189.0 | 229.5 | N | 2.600 | 0.242 | 1.600 |

| ALLOW SHR-VHA | OR STR LEVEL | IR STR LEVEL | STEEL E | N DES | N ULT | CREEP FACTOR | SPEC A/R |
|---|---|---|---|---|---|---|---|
| 0.300 | 0.900 | 0.800 | 28000 | 1.275 | 1.625 | 1.6 | 1979 |

ONE INTERIOR DIAPHRAGM IS ASSUMED AT MIDSPAN

```
*****************************************************************************
*                           RATING OF A FACIA BEAM                          *
*****************************************************************************
```

BASIC BEAM SECTION PROPERTIES

| DEPTH IN | AREA IN.2 | WEIGHT LBS/FT | M OF I IN.4 | N.A. TO TOP YT IN. | N.A. TO BOT YB IN | Z TOP IN.3 | Z BOT IN.3 |
|---|---|---|---|---|---|---|---|
| 36.00 | 701.0 | 762.49 | 119746.7 | 19.83 | 16.17 | 6038.3 | 7406.1 |

```
                    COMPOSITE SECTION PROPERTIES

 SLAB   AREA    M OF I  N.A. TO   N.A. TO   N.A. TO   Z TOP    Z TOP    Z BOT
 WIDTH  IN.2     IN.4   SLAB TOP  BEAM TOP  BEAM BOT  SLAB     BEAM     BEAM
138.00 1512.9  332747.7  14.68      7.18     28.82   22672.6  46368.1  11544.2


                UNIFORM DEAD LOADS ACTING ON GIRDER  (KIPS/FT)

                                                FUTURE
GIRDER    SLAB    HAUNCH  FORMWORK   INPUT    WEARING   INPUT    TOTAL    TOTAL
WEIGHT   WEIGHT   WEIGHT   WEIGHT     DL1     SURFACE    DL2      DL1      DL2
0.7625   1.2750   0.0000   0.1913   0.0000    0.0000   0.3010   2.2287   0.3010


                        DEAD LOAD AND LIVE LOAD REACTIONS

    DL1       DL2     IMPACT  LL+I H20   LL+I HS20  LL+I ML80  LL+I TK527
  REACTION  REACTION  FACTOR  REACTION   REACTION   REACTION   REACTION
    55.7       7.3    1.288    56.5 L     79.1 T     88.9 T     89.0 T

                              LL+I EV2   LL+I EV3
                              REACTION   REACTION
                                68.1 T    100.8 T


                  PRESTRESSING FORCE (STRAND PATTERN KNOWN)


    AT CL BRG.:
    -----------
     INITIAL    LOSS %  EFFECTIVE  EFF NO OF STRANDS  ECCENTRICITY  C.G.S.
     782.460     28.28   561.211         36                9.669      6.500


    AT DEBONDED LENGTH 1.229 FT. FROM CL BRG.:
    ------------------------------------------
     INITIAL    LOSS %  EFFECTIVE  EFF NO OF STRANDS  ECCENTRICITY  C.G.S.
    1130.220     28.28   810.638         52               10.976      5.192


    AT MID SPAN:
    ------------
     INITIAL    LOSS %  EFFECTIVE  EFF NO OF STRANDS  ECCENTRICITY  C.G.S.
    1130.220     28.28   810.638         52               10.976      5.192




******************************************************************************
*                               RATING SUMMARY                               *
******************************************************************************


     FLEXURAL RATINGS (BASED ON MOMENT)            SHEAR RATINGS (1979 I)

LOAD        FACTOR   TONS   LOCATION                 FACTOR   TONS   LOCATION
                           FROM CL BRG                              FROM CL BRG
H20   IR   2.398    47.95   24.170              IR   2.342    46.85   12.085
```

```
      OR   4.953    99.07   24.170           OR   3.910    78.20   12.085
HS20  IR   1.734    62.42   21.753           IR   1.647    59.31   12.085
      OR   3.553   127.90   21.753           OR   2.750    98.99   12.085
ML80  IR   1.396    51.15   24.170           IR   1.424    52.16   12.085
      OR   2.884   105.68   24.170           OR   2.376    87.07   12.085
TK527 IR   1.485    59.39   24.170           IR   1.476    59.04   12.085
      OR   3.064   122.56   21.753           OR   2.464    98.55   12.085
EV2   IR   1.972    56.70   21.753           IR   1.852    53.23   12.085
      OR   4.041   116.18   21.753           OR   3.091    88.85   12.085
EV3   IR   1.285    55.27   24.170           IR   1.258    54.10   12.085
      OR   2.655   114.18   24.170           OR   2.100    90.31   12.085



******************************************************************************
*                             CONTROLLING RATINGS                            *
******************************************************************************


     VEHICLE TYPE               IR           OR

     H20   LOADING (TONS)    46.85 S     78.20 S
     HS20  LOADING (TONS)    59.31 S     98.99 S
     ML80  LOADING (TONS)    51.15 F     87.07 S
     TK527 LOADING (TONS)    59.04 S     98.55 S
     EV2   LOADING (TONS)    53.23 S     88.85 S
     EV3   LOADING (TONS)    54.10 S     90.31 S

     F = FLEXURAL RATING        S = SHEAR RATING
```

## 3.7 ADJACENT PRESTRESSED BOX/PLANK BEAMS

This Section covers the rating Adjacent Prestressed Box Beams for both composite and non-composite situations. Plank beams are treated as non-voided adjacent box beams for both composite and non-composite situations.

### 3.7.1 Policies and Guidelines

Most adjacent box beam bridges can be rated by analytical methods based on design plans and field measurements.

If plans are not available, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized for composite beams. However, the engineering judgement procedure is not permitted for non-composite adjacent box beam bridges.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.5 for guidance on prestressed concrete, MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.7.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating. Allowable stress design method would not have been used for these bridges.

#### 3.7.2.1 LFR or ASR Method

For bridges built prior to 2011, if load factor design methodology was used then the load rating should be performed using PS3. For bridges built prior to 2011 and designed with LRFD, LFR or LRFR can be utilized for the rating.

#### 3.7.2.2 LRFR Method

For adjacent prestressed box beam bridges designed with LRFD on or after 2011, load ratings should be performed using LRFD in PSLRFD.

### 3.7.3 Live Load and Dead Load Distribution

#### 3.7.3.1 LFR or ASR Method

Pub. 238 Section 3.3 provides guidance on the distribution of live load to longitudinal girders. Typically, the live load distribution factor for exterior beams is in accordance with the AASHTO Standard Specification.

The typical distribution factors are as follows when AASHTO distribution factors are applicable:

- Composite

| Exterior | | Interior | |
|---|---|---|---|
| Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.4.3) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.4.3) |
| Lever Rule* | $S/D$ | Lever Rule* | $S/D$ |

- Non-Composite

| Exterior | Interior | |
|---|---|---|
| Shear and Moment (Pub 238 6B.6.3) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.4.3) |
| Use the Larger of:<br>- *S/D* (AASHTO 3.23.4.3)<br>- Lever Rule* | Lever Rule* | *S/D* |

Note: If shear keys are determined to be ineffective, assume there is no distribution of load between beams (Pub 238 6B.6.3).

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2.2.3 and Pub. 238 IP 3.3.3.1 for additional discussion of skew. Note, for prestressed concrete adjacent box beams, the skew correction factor shall be applied to all the beams which are on a skew.

For composite adjacent prestressed box bridges, dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all girders (AASHTO Standard Specification 3.23.2.3.1.1).

For adjacent non-composite box beams, see Pub 238 6B.6.1 for distribution of barrier dead load.

3.7.3.2 LRFR Method

For spread composite prestressed box beam bridges designed with LRFD, distribution factors may be computed in PSLRFD. For analysis outside of the program, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.2.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO distribution factors. See DM-4 Table 4.6.2.2.3c-1 for the formulas for the skew correction factor.

Refer to DM-4 Section 3.5.1.1P for discussion of the application of dead load on girder and box beam structures.

### 3.7.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1941-1960 Volume 3 | Adjacent Box Beams (Composite) | 9/19/1960 | S-3902, S-3903 |
| Standards for Old Bridges 1941-1960 Volume 3 | Adjacent Box Beams (Non-Composite) | 9/19/1960 | S-3906, S-3907 |
| Standards for Old Bridges 1961-1965 Volume 4 | Prestressed Concrete Bridge Standards | 3/8/1962 | S-3909 |

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1965-1972 Vol. 5 | Instructions for Acceptance of Prestressed Beams with Cracks | 3/17/1970 | P-800 |
| Standards for Old Bridges 1965-1972 Vol. 5 | Prestressed Concrete Bridge – 4ft Adjacent Plank Beams (Composite) | 12/8/1966 | S-3912 |
| BD-200 Series_BD-201, March 1973 | Adjacent Box Beams | 3/1/1973 | 38 through 44 |
| Standard Plans for Low Cost Bridges (Series BLC-400,April | Prestressed Box Beam | 4/1/1982 | BLC-400, BLC-401, BLC-402 |
| Standard Plans for Low Cost Bridges (Series BLC-400, June 1982 | Prestressed Box Beam | 6/1/1982 | BLC-400, BLC-401, BLC-402 |
| BD-600 Series, 1989 Edition, Change 3 | Beam Sizes and Section Properties | 9/11/1989 | BD-652 |
| BD-600 Series, Sept. 1994 Edition | Various | 9/30/1994 | BD-652, BD-661, BD-666 |
| BD-664 deleted 12/24/1999; BD-652 and BD-666 deleted 6/30/2000 | | | |
| Online document not available | Box Beam Reinforcement Details | 12/24/1999 | BD-661 |
| Online document not available | Various | 6/30/2000 | BD-652, BD-665 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 1/21/2003 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 4/15/2004 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 7/20/2007 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 12/29/2008 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 9/20/2010 | BD-652 |
| Online document not available | Prestressed Beam Sizes and Section Properties | 8/31/2012 | BD-652 |
| Standards for Bridge Design September 2010 Edition Change 3 | Box Beam Reinforcement Details | 11/21/2014 | BD-661 |
| Standards for Bridge Design April 2016 Edition Change 7 | Prestressed Beam Sizes and Section Properties | 4/29/2016 (Latest Revision 10/7/2024) | BD-652 |

### 3.7.5 Modeling Section Properties and Deterioration

See Section 2.5.4.1 for a discussion of modeling section loss.

See Pub. 238, Section IE 6.1.5.3I for guidance on removing strands from analysis based on various defects in prestressed beams. In addition, see Pub. 238, Appendix IE 04-D for an example of adjusting strand patterns based on this guidance.

### 3.7.6 Standard Practices

If plans and/or details are unavailable to verify the design, assume that the beams are non-composite with the deck.

When looking at the shear ratings, shear capacity of the beam should be determined as per AASHTO Std. Spec. 9.20 with a shear capacity reduction factor of 0.9. If shear controls the ratings for a bridge built prior to 1992 and there is no sign of shear distress in the beams, the shear capacity reduction factor may be increased to 1.0 as per the 1979 edition of the AASHTO Std. Spec.

For non-composite adjacent box beams, the shear keys between beams may be broken making it so there is no lateral transfer of loads to the adjacent beams. When applying the barrier load, it may only be distributed among the beams between which the shear keys are still intact and functioning. If the shear key between the first interior beam and the fascia beam has failed, then the barrier load shall only be applied to the fascia beam for rating. A similar method should be used for any other DL2 loads.

The load rating engineer shall use their best judgment to determine the effectiveness of the shear keys. Evidence of possible shear key failure includes differential deflection of the beams under live load, reflective cracking in the wearing surface, and leakage between the beams. Loose post-tensioning ties can also contribute to independent action of the beams. Differential deflection of the beams under live load is a definite sign the beams are acting independently.

In the scenario in which non-composite adjacent box/plank beams are made composite with a deck replacement, the beams must be re-rated to determine the new capacity.

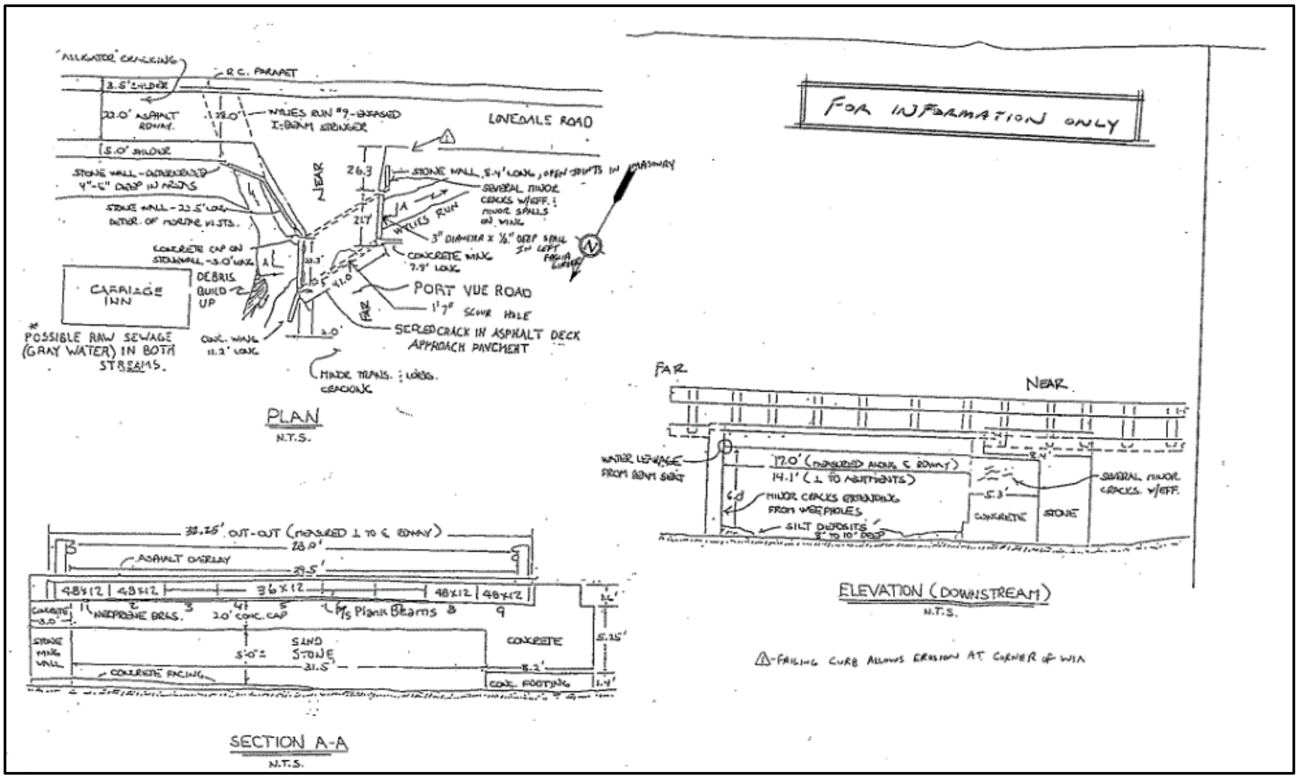
### 3.7.7 Common QA Findings

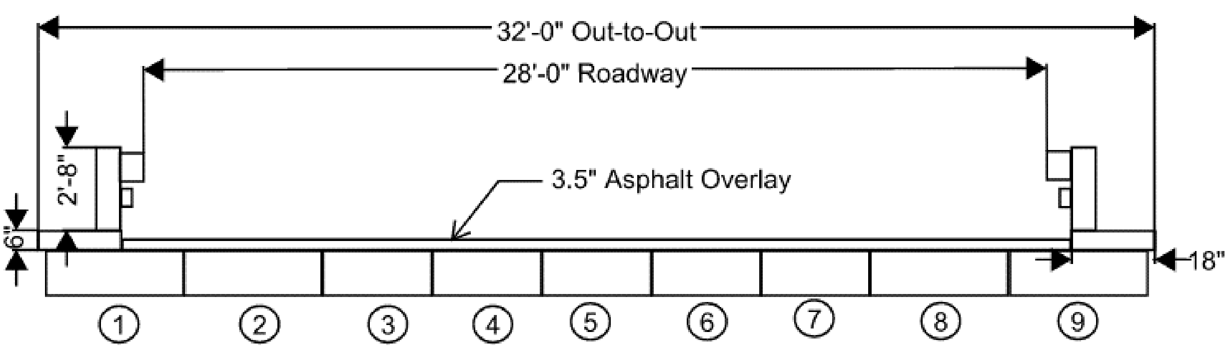
This Section is in development and will be provided in future editions of this manual.

### 3.7.8 Sample Load Rating

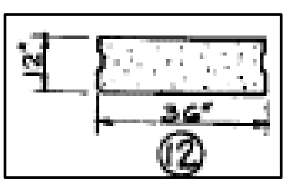
This Section contains sample load rating calculations for an interior and exterior plank beam of a single span non-composite prestressed concrete adjacent plank beam structure shown below. The analysis will be performed using PennDOT’s LFD Prestressed Concrete Girder Design and Rating (PS3) Program, Version 3.6.0.5 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part-IV (DM-4).

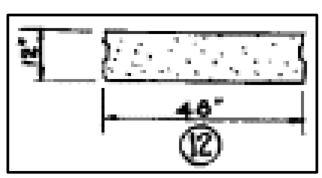
The single span superstructure consists of a nine (9) prestressed concrete plank beams with a 3 ½” asphalt overlay. There are four (4) 12” deep x 48” wide beams and five (5) 12” deep x 36” wide beams.




NOTES:
-Barrier/Curb is typical
-Beams 1, 2, 8, & 9 are 48x12 P/S Plank Beams
-Beams 3 to 7 are 36x12 P/S Plank Beams



### 3.7.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

Done By: Date:

Checked By: Date:

| | |
|---|---|
| Structure ID (5A01): | 02-7443-4256-2978 |
| Inspection Date (7A01): | 4/7/2021 |
| Facility Carried (5A08): | Liberty Way |
| Feature Intersected (5A07): | Liberty Way over Wylies Run |
| Structure Type (6A26 - 6A29): | 1-Span P/S Plank Beam |
| Spans / Members Analyzed: | Span 1, Beam 3 |
| Analysis Method: | LFD |
| PennDOT Program / Version: | PS3 Version 3.6.0.5 |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 0.87 | 17.4 | 1.66 | 33.3 | 1.50 | 29.9 | * | * | M | M |
| HS20 | 0.87 | 31.4 | 1.66 | 59.9 | 1.50 | 53.9 | * | * | M | M |
| ML80 | 0.66 | 24.1 | 1.26 | 46.0 | 1.13 | 41.4 | * | * | M | M |
| TK527 | 0.77 | 30.7 | 1.46 | 58.6 | 1.32 | 52.7 | * | * | M | M |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 0.83 | 23.9 | 1.59 | 45.7 | 1.43 | 41.1 | * | * | M | M |
| EV3 | 0.58 | 25.0 | 1.11 | 47.6 | 1.00 | 42.8 | * | * | M | M |

Comments/Assumptions*:

Superstructure and substructure condition ratings are both 4. Therefore, for an ADTT < 500, the SLC Factor equals 0.9 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is Exterior Beam 1 in Span 1.

PS3 analysis accounts for exposed stirrups/strands, longitudinal cracks, and 3.5" asphalt wearing surface.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

#### 3.7.8.2 Interior Girder Load Rating Analysis

##### 3.7.8.2.1 PS3 Input Parameters

A typical interior plank beam (beam 6) with loss was selected and will be rated using PS3. Many of the PS3 input parameters can be left blank. Only the required input values are discussed below. Refer to the PS3 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02744342562978
- Description = MIDDLE INTERIOR BEAM 6
- Live Load = Blank for H, HS, ML80, TK527, EV2, and EV3 vehicles.
- Output = 0 for normal output
- Roadway Width = 28.00 ft
- I or F = I for Interior Beam
- Design = R for rating only
- Skew Correction Factor = 1.155 (See below). Per DM-4 Table 3.23.2(A), in determining end shear on multi-beam bridges, all beams shall be treated like the beam at the obtuse corner, i.e., the adjustment is appliable to all beams and shall be applied to the distribution factor for interior beams.

  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

$$\text{SCF} = 1.0 + \left(\frac{12L}{90d}\right)\sqrt{\tan\ \theta}$$

where,

Span Length, L = 17.0 ft

Beam Depth, d = 12 in

AASHTO Skew Angle, θ = (90° – 56° 00' 00") = 34.0°

The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

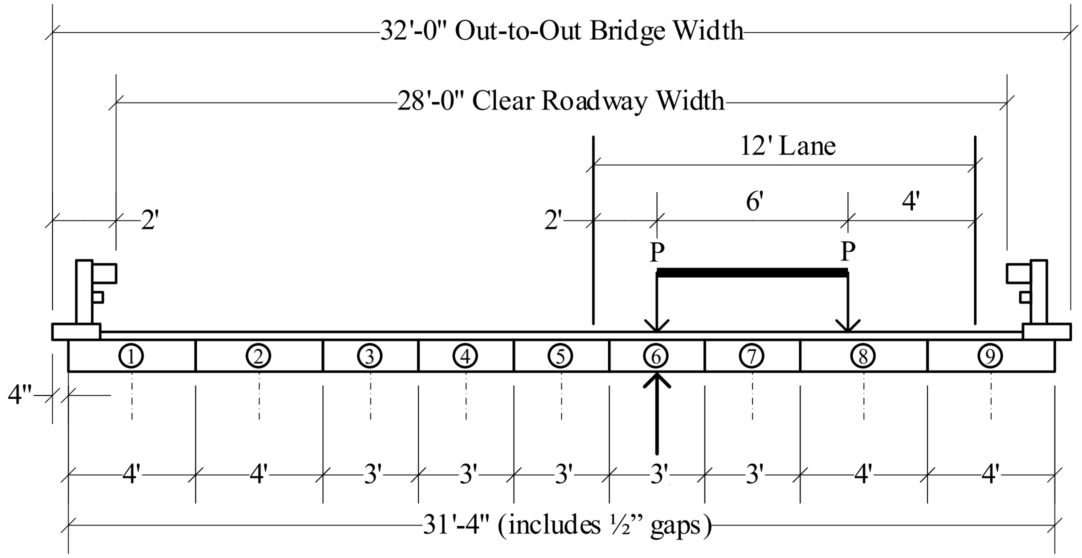
| | |
|---|---|
| $0° \leq \theta \leq 60°$ | $\theta$ = 34.0°, OK |
| 20 ft ≤ L ≤ 120 ft | L = 17.0 ft, NG…Call OK |
| 17 in ≤ d ≤ 60 in | d = 12 in, NG…Call OK |
| 3.5 ft ≤ b ≤ 6 ft | b = 3 ft, NG…Call OK. Current LRFD lower limit is 35 in. |
| $5 \leq N_b \leq 20$ | $N_b$ = 9, OK |

Although the range of applicability limits are exceeded for several variables, apply the calculated skew correction factor as calculated. Code guidance when the limits are exceeded is not provided.

Bridge Cross Section and Loading

- Beam Spacing, S = 3'-0" beam width + ½" gap = 36.5 in

- Distribution Factor – Shear
  Per AASHTO 3.23.1, the interior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the one lane condition shown below, the reduction in load intensity factor is 1.00.




1 Lane Loaded, $DF_V$ = P(1) x 1.00 = 1.000 wheels / (2 wheels/axle) = 0.500 axles

- Distribution Factor – Moment

  Per AASHTO 3.23.4.3, the interior beam moment distribution factor for multi-beam bridges is calculated per AASHTO Equation 3-11.

  Per AASHTO Equation 3-11, the moment distribution factor is equal to

$$DF_M = \frac{S}{D}$$

where,

Width of Precast Member, S = 3'-0" = 3.0 ft

Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
Roadway Width, W = 28.0 ft
$N_L$ = 28.0 ft / 12 ft = 2.33 ==> Use 2 lanes

Span Length, L = 17.0 ft

Factor, $K = \sqrt{[(1 + \mu)I/J)]}$ per AASHTO 3.23.4.3
Poisson's Ratio, $\mu = 0.20$ (assumed per AASHTO LRFD 5.4.2.5)
Moment of Inertia, $I = 5122.1$ in$^4$ per PS3 Output
St.-Venant Torsional Constant, $J = \Sigma[(1 - 0.630t/b)bt^3/3] = 16381.4$ in$^4$ where b = 36" and t = 12"
$K = \sqrt{[(1 + 0.20) \times 5122.1 \text{ in}^4 / 16381.4 \text{ in}^4]} = 0.6125$

W/L = 32.0 ft / 17.0 ft = 1.88 > 1, C = K = 0.6125

$D = (5.75 - 0.5N_L) + 0.7N_L(1 - 0.2C)^2$ per AASHTO Equation 3-12
$D = (5.75 - 0.5 \times 2) + 0.7 \times 2 \times (1 - 0.2 \times 0.6125)^2 = 5.828$

$DF_M$ = S / D = 3 / 5.828 = 0.515 wheels x (1 axle/2 wheels) = <u>0.257 axles</u>

The Range of Applicability checks per AASHTO 3.23.4.3:

| | |
|---|---|
| $\sqrt{I/J} \leq 5$ | $\sqrt{I/J} = 0.56$, OK |
| $\theta \leq 45°$ | $\theta = 34°$ ft, OK |

Per Pub 238 IP 6B.6.3, the distribution for moment and shear for an interior girder shall use a wheel load distribution factor = 1.0 where there is loss of grout in the shear key and/or tie rod. Based on the field inspection, there was no loss of grout in the shear key or tie rod. Therefore, the moment distribution factor is calculated per AASHTO 3.23.4.3.

- Distribution Factor – Deflection
  Per AASHTO 8.13.2, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Reduction Factor = 1.00 for 2 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 2 lanes x 1.00 / 9 beams = <u>0.222 lanes/beam</u>

- Uniform Dead Loads – UDLF
  UDLF = 0 ksf. There is not any uniform dead load applied to the beam. For plank beams, PS3 will use 0.015 ksf if this value is not entered.

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight. Girder self-weight is calculated by PS3. Per the PS3 User's Manual Section 5.3, adjacent non-composite box beam loads due to curbs, parapets, railing, bituminous surface…etc, can be applied as DL1 loads.

  <u>Asphalt Wearing Surface</u>
  A 3 ½" asphalt wearing surface is present on the bridge and will be included in the rating analysis. Since there isn't a composite deck, the dead load will be applied based on the tributary width.
  WS = 0.140 kcf x (3.5 in /12 in/ft) x (3.00 ft + ½" gap / 12 in/ft) = 0.124 kips/ft.

  <u>Roadway Barriers</u>
  Per Pub 238 IE 6B.6.1, the distribution of barrier dead load for the first interior beam shall be assumed to support 50% of the barrier dead load. Since this beam is a middle interior beam of a

non-composite adjacent beam bridge without a deck slab, the interior beam 6 barrier load will be zero.

DL1 = 0.124 kips/ft + 0 kips/ft = 0.124 kips/ft

- Future Wearing Surface – FWS
  Since there is an asphalt wearing surface present on the bridge, future wearing surface is not included in the rating analysis.
  FWS = 0 kips/ft or blank.

- Dead Loads – DL2
  Load due to barriers and wearing surfaces were entered as DL1 loads.
  DL 2 = 0 kips/ft

- P/S Loss = 0.08 for Stress-Relieved Strands
- Strand L or S = S for Stress-Relieved Strands

Span Lengths

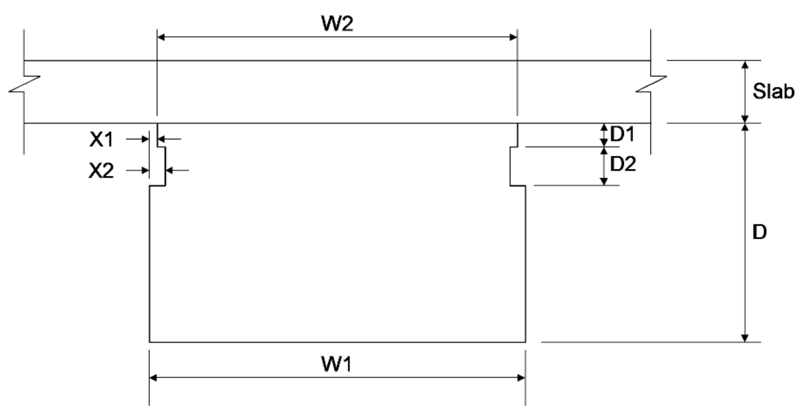
- Span 1 Length = 17.0 ft
- Beam Projection = 9.0 in

Prestress Criteria

- 28-Day Compressive Strength of Beam Concrete = 4.5 ksi
- 28-Day Compressive Strength of Slab Concrete = blank
- Compressive Strength of Beam Concrete at Initial Prestressing = 4.0 ksi
- Ultimate Strength of Prestressing Steel = 250 ksi
- Strand Diameter = 1/4" = 0.25 in
- Number of Strand Rows = 2
- Number of Debonded Lengths = 0
- Stirrup Details = Y indicating stirrup reinforcement will be entered

Prestressed Concrete Beam Dimensions

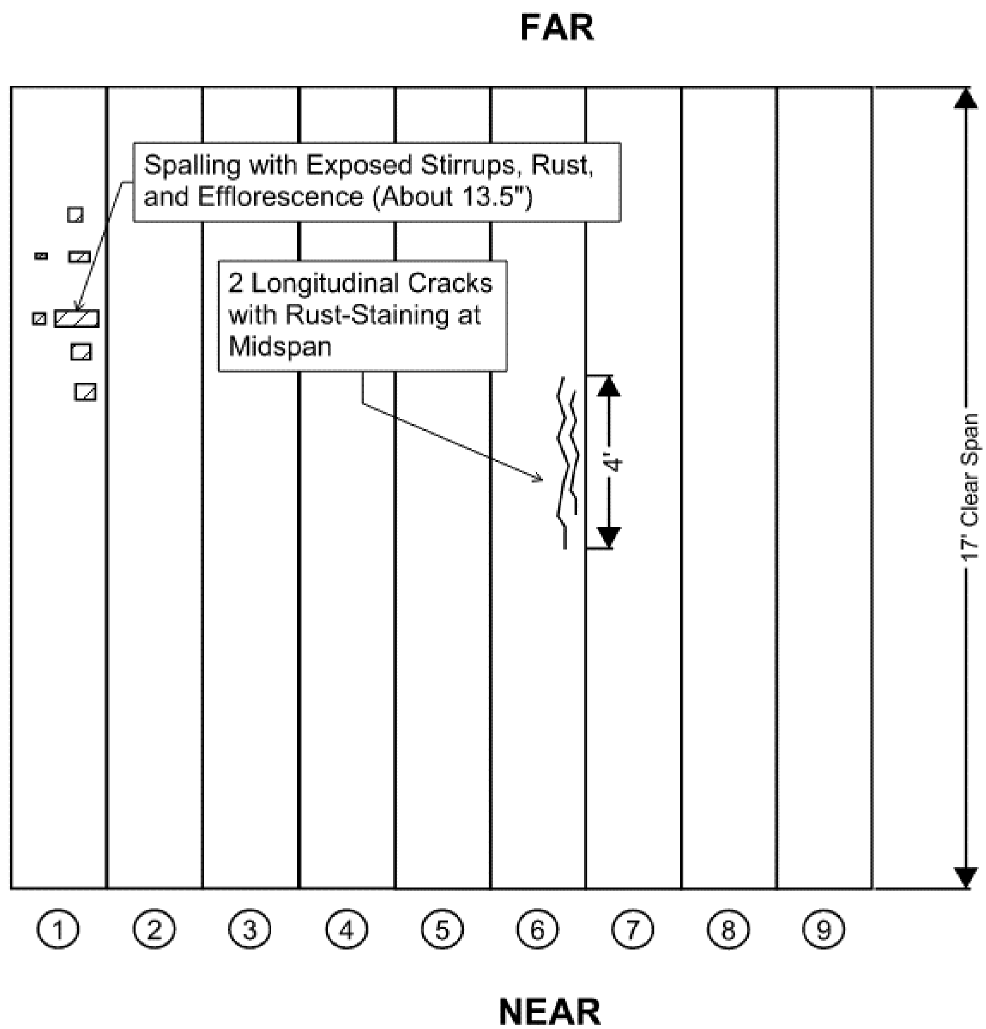
- Type = P for a plank beam
- Comp = N for a non-composite beam
- Design or D = 12.00 in beam depth
- W1 = 36.0 in
- D1 = 3 in
- D2 = 4 in
- X1 = 0.375 in
- X2 = 0.75 in
- Slab Thickness = 0 in
- Haunch Thickness = 0 in



Plank Beam

Strand Details

The following sketch was developed based on a field inspection. Beam 6 was noted as having 2 longitudinal cracks near midspan.

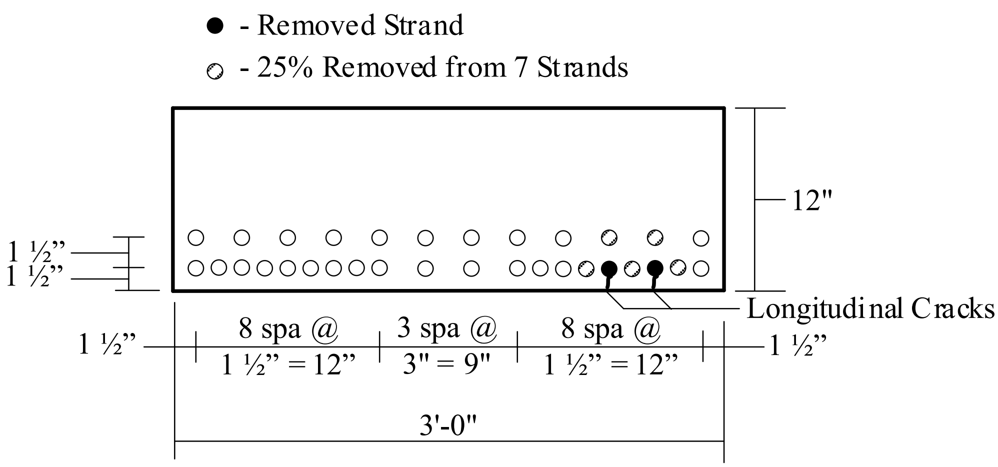



FRAMING PLAN

In accordance with Pub 238 IE 6.1.5.3I, Method B requires the strand area to be reduced to 75% of the original cross-sectional area for strands on each level directly in line with the crack and for the strands closest to the exterior surface adjacent to the longitudinal crack. As shown below, 5 strands in row 1 and 2 strands in row 2 would be reduced to 75% of the original area. Additionally, for beams with longitudinal cracking, all other strands shall be reduced to 95% of the original cross-sectional area.

(1 - 0.75) x 7 strands = 1.75 strands, Remove 2 strands entirely.

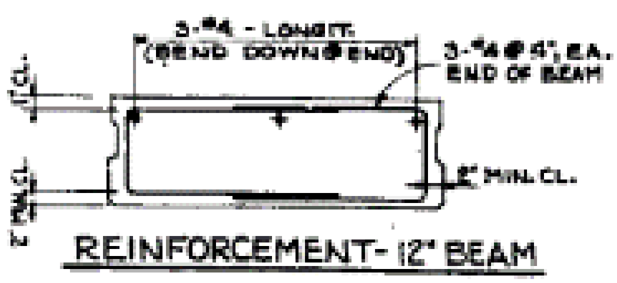
Strand Area = 0.036 in$^2$ for ¼” diameter strand




- Strand Area = 0.036 in$^2$ x 0.95 = 0.034 in$^2$
- G1 = 1.50 in
- G2 = 1.50 in
- R1 = 18
- R2 = 12

<u>Stirrup Details</u>

- Spec = “A” (Default)
- Stirrup Area = 0.20 in$^2$ for one leg of a #4 stirrup
- Yield Strength of Reinforcement Stirrups, $f_y$ = 40 ksi
- Location = 0.00 ft
- Spacing = 4.0 in
- Location = 0.67 ft (3 - #4 @ 4”)



**TITLE**

| | | | |
|---|---|---|---|
| Structure ID: | 02-7443-4256-2978 | | |
| Description: | INTERIOR BEAM 6 | | |
| SLC Level: | | | |
| Live Load: | | | H20, HS20, ML80, TK527, EV2, and EV3 |
| Output: | 0 | | Normal output |
| Impact Factor: | | | |
| Gage Distance: | | ft | |
| Passing Distance: | | ft | |
| Roadway Width: | 28.00 | ft | Per Contract Drawings |
| DLF: | | | |
| LLF: | | | |
| I or F: | I | | Interior beam |
| Principal: | | | |
| Design: | R | | Rating only problem |
| Skew Correction Factor: | 1.155 | | See Hand Calcs |
| IR Stress Level: | | | |
| AASHTO fc: | | | |

**Bridge Cross-Section and Loading**

| | | | |
|---|---|---|---|
| Beam Spacing: | 36.500 | in | Per Contract Drawings |
| Distr Factor -Shear: | 0.500 | | See Hand Calcs |
| Distr Factor -Moment: | 0.257 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.222 | | See Hand Calcs |
| Unit Weight of Deck Concrete: | | | |
| UDLF: | 0.000 | | See Hand Calcs |
| DL1: | 0.124 | kip/ft | See Hand Calcs |
| FWS: | 0.000 | kip/ft | See Hand Calcs |
| DL2: | 0.000 | kip/ft | See Hand Calcs |
| Initial P/S Force: | | kips | |
| Midspan Eccentricity: | | in | |
| End Eccentricity: | | in | |
| P/S Loss %: | 0.08 | | Stress Relieved Strands |
| Drape Point: | -- | | Decimal part of the span |
| Strand L or S: | S | | Stress Relieved Strands |
| Rate FWS?: | | | |

**Span Lengths**

| | | | |
|---|---|---|---|
| Span Length 1: | 17.00 | ft | Per Contract Drawings |
| Beam Projection: | 9.000 | in | Per Contract Drawings |

**Prestress Criteria**

| | | | |
|---|---|---|---|
| Beam Conc $f'_{cb}$: | 4.5 | ksi | Per Contract Drawings |
| Slab Conc $f'_{cs}$: | --- | ksi | Per Contract Drawings |
| Conc Init $f'_{ci}$: | 4.0 | ksi | Per Contract Drawings |
| Strand Ult $f'_{s}$: | 250 | ksi | Per Contract Drawings |
| Strand Diameter: | 0.2500 | in | Per Contract Drawings |
| No. of Rows: | 2 | | Per Contract Drawings |
| No. Lx: | 0 | | Per Contract Drawings |
| St. Det: | Y | | Stirrup Details |

**Beam Dimensions**

| | | | |
|---|---|---|---|
| Beam Type: | P | | Plank Beam |
| Composite: | N | | Non-Composite |
| Designation or D: | 12 | | Per Contract Drawings |
| W1: | 36 | in | Per Contract Drawings |
| W2: | | in | |
| W3: | | in | |
| T1: | | in | |
| T2: | | in | |
| B1: | | in | |
| B2: | | in | |
| B3: | | in | |
| B4: | | in | |
| D1: | 3 | in | Per Contract Drawings/Standard Drawings |
| D2: | 4 | in | Per Contract Drawings/Standard Drawings |
| X1: | 0.375 | in | Per Contract Drawings/Standard Drawings |
| X2: | 0.75 | in | Per Contract Drawings/Standard Drawings |
| Slab Thickness: | 0.00 | in | Per Contract Drawings |
| Haunch: | 0 | in | |

**Strands**

| | | | |
|---|---|---|---|
| Strand Area: | 0.034 | $in^2$ | Per Contract Drawings/See Hand Calcs |
| G1: | 1.500 | in | Per Contract Drawings/See Hand Calcs |
| G2: | 1.500 | in | Per Contract Drawings/See Hand Calcs |
| R1: | 18 | | Per Contract Drawings/See Hand Calcs |
| R2: | 12 | | Per Contract Drawings/See Hand Calcs |
| R3: | | | |
| R4: | | | |

**Debonded**

| | | | |
|---|---|---|---|
| Debonded Length: | 1.229 | ft | See Hand Calcs |
| R1: | 1 | | |
| No. Strands: | 10 | | |
| R2: | 2 | | |
| No. Strands: | 6 | | |
| R3: | | | |
| No. Strands: | | | |
| R4: | | | |
| No. Strands: | | | |

**Stirrups**

| | | | |
|---|---|---|---|
| Spec: | A | | Shear values are to be computed per AASHTO 9.20 |
| Area: | 0.200 | $in^2$ | Per Contract Drawings |
| $f'_y$: | 40 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 4.000 | in | Per Contract Drawings |
| Location 2: | 0.67 | ft | Per Contract Drawings |
| Spacing 2: | | in | |
| Location 3: | | ft | |

### 3.7.8.2.2 PS3 Output

```
******************************************************************************
*                                                                            *
*          LFD Prestressed Concrete Girder Design and Rating          330740 *
*                                                                            *
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******************************************************************************
```

```
             LFD Prestressed Concrete Girder Design and Rating          330740

PROGRAM P4353050                                                05/06/2024 09:28
VERSION 3.6.0.5           LAST UPDATED 12/22/2023        DOCUMENTATION 12/2023

INPUT: Interior PS Plank Beam PS3 Analysis.dat




STRUCTURE ID - 02744342562978 - MIDDLE INTERIOR BEAM 6

 SLC   LIVE   OUT-  IMPACT     GAGE    PASSING   ROADWAY  LOAD FACTORS
LEVEL  LOAD   PUT   FACTOR   DISTANCE  DISTANCE   WIDTH     DLF    LLF   I OR F
               0     0.000      0.0       0.0     28.00    0.00  0.00      I

                         SKEW      IR
 PRINCIPAL             CORRECTION  STRESS  AASHTO
 STRESSES     DESIGN     FACTOR    LEVEL     FC
                 R       1.155     0.000


                       BRIDGE CROSS SECTION AND LOADING

                                  UNIT WEIGHT                          INITIAL
 BEAM     DISTRIBUTION FACTORS    OF DECK        DEAD LOADS              P/S
SPACING  SHEAR MOMENT DEFLECTION  CONCRETE   UDLF   DL1   FWS   DL2     FORCE
  36.5   0.500  0.257  0.222       0.0000   0.0000 0.124 0.000 0.000    0.000

                                                                   RATINGS
ECCENTRICITY    P/S              LEHIGH LOSS METHOD      STRAND  w/ & w/o
MIDSPAN  END   LOSS %   XDRAPE  T0   TS   TD  IC MFG IST   L or S    FWS
 0.000  0.000    0.08   0.0000   0    0    0  0   0   0       S


                            SPAN LENGTHS (SIMPLE)
                                                                         BEAM
SPAN #     1       2       3       4       5       6       7       8     PROJ
LENGTH   17.00                                                           9.000


                              PRESTRESS CRITERIA

  BEAM    SLAB    CONC    STEEL   STEEL   STEEL      INITIAL ALLOWABLE
  CONC    CONC    INIT    INIT    YIELD    ULT     COMP   TENS  DRP/DBND
  F'CB    F'CS    F'CI    FSI      Fy      F'S      FCI    FTI    FTFD
 4.500   0.000   4.000    0.0     0.0   250.0    0.000  0.000   0.000

  FINAL ALLOWABLE   ALLOW    OR               MODULAR           EST.
 COMP  TENS  SLAB   SHEAR  STRESS  STEEL      RATIOS     CREEP    %     STRAND
  FC    FT    FCS    VHA   LEVEL     E       DES  ULT   FACTOR  LOSS  DIAMETER
0.000 0.000 0.000   0.000  0.000       0  0.000 0.000    0.0    0.0   0.2500

  NUMBER OF    NUMBER OF    STIRRUP
    ROWS          Lx        DETAILS
       2           0           Y


                     PRESTRESSED CONCRETE BEAM DIMENSIONS

  TYPE    COMP       D        W1       W2       W3       T1       T2
    P       N      12.000   36.000    0.000    0.000    0.000    0.000
```

| B1 | B2 | B3 | B4 | D1 | D2 | X1 | X2 | SLAB THICK | HAUNCH |
|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 0.00 | 0.00 | 3.00 | 4.00 | 0.375 | 0.750 | 0.00 | 0.00 |

STRAND DETAILS

| AREA | G1 | G2 | R1 | R2 | R3 | R4 | R5 | R6 | R7 | R8 | R9 | R10 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.034 | 1.50 | 1.500 | 18 | 12 | | | | | | | | |

WARNING ---- INPUT STRAND AREA DOES NOT CORRESPOND TO STANDARD STRAND SIZE FOR GRADE 250.

WARNING ---- INPUT STRAND DIAMETER/AREA COMBINATION NOT CONSISTENT FOR STANDARD STRAND SIZES FOR GRADE 250.

STIRRUP DETAILS

| SPEC. FOR ANAL/RATE | STIRRUP AREA | FSY | LOCATION | SPACING | LOCATION | SPACING | LOCATION | SPACING |
|---|---|---|---|---|---|---|---|---|
| A | 0.200 | 40 | 0.00 | 4.000 | 0.67 | 0.000 | 0.00 | 0.000 |
| | | | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 |

DEFAULT VALUES

| GAGE DIST | PASS DIST | DLF | LLF | UNIT WT DK CONC | ST INI FSI | ST YLD Fy | AASHTO FC |
|---|---|---|---|---|---|---|---|
| 6.0 | 4.0 | 1.30 | 2.17 | 0.150 | 175.0 | 212.5 | N |

| COMP FC | TENS FT | ALLOW SHR-VHA | OR STR LEVEL | IR STR LEVEL | STEEL E | CREEP FACTOR |
|---|---|---|---|---|---|---|
| 1.800 | 0.201 | 0.300 | 0.900 | 0.800 | 28000 | 1.6 |

*****************************************************************************
*                        RATING OF AN INTERIOR BEAM                         *
*****************************************************************************

BASIC BEAM SECTION PROPERTIES

| DEPTH IN | AREA IN.2 | WEIGHT LBS/FT | M OF I IN.4 | N.A. TO TOP YT IN. | N.A. TO BOT YB IN | Z TOP IN.3 | Z BOT IN.3 |
|---|---|---|---|---|---|---|---|
| 12.00 | 423.8 | 441.41 | 5122.1 | 6.04 | 5.96 | 848.3 | 859.1 |

UNIFORM DEAD LOADS ACTING ON GIRDER (KIPS/FT)

| GIRDER WEIGHT | FORMWORK WEIGHT | INPUT DL1 | FUTURE WEARING SURFACE | INPUT DL2 | TOTAL DL1 | TOTAL DL2 |
|---|---|---|---|---|---|---|
| 0.4414 | 0.0000 | 0.1240 | 0.0000 | 0.0000 | 0.5654 | 0.0000 |

DEAD LOAD AND LIVE LOAD REACTIONS

| DL1 REACTION | DL2 REACTION | IMPACT FACTOR | LL+I H20 REACTION | LL+I HS20 REACTION | LL+I ML80 REACTION | LL+I TK527 REACTION |
|---|---|---|---|---|---|---|

```
       4.8       0.0      1.300    24.6 T     26.2 T     25.8 T     24.3 T

                                   LL+I EV2   LL+I EV3
                                   REACTION   REACTION
                                     26.2 T     32.4 T


                     PRESTRESSING FORCE (STRAND PATTERN KNOWN)

    INITIAL    LOSS %    EFFECTIVE  NO. OF STRANDS  ECCENTRICITY  C.G.S.
    178.500     20.00     142.800         30             3.862      2.100



******************************************************************************
*                               RATING SUMMARY                               *
******************************************************************************


     FLEXURAL RATINGS (BASED ON MOMENT)            SHEAR RATINGS (AASHTO)

LOAD        FACTOR   TONS   LOCATION                FACTOR   TONS   LOCATION
                           FROM CL BRG                             FROM CL BRG
H20   IR    1.410    28.20    8.500            IR    3.056    61.13    4.250
      OR    2.548    50.96    8.500            OR    5.102   102.03    4.250
HS20  IR    1.410    50.76    8.500            IR    2.792   100.53    0.500
      OR    2.548    91.73    8.500            OR    4.661   167.80    0.500
ML80  IR    1.064    38.98    8.500            IR    2.198    80.53    0.500
      OR    1.923    70.44    8.500            OR    3.669   134.42    0.500
TK527 IR    1.241    49.65    8.500            IR    2.396    95.83    0.500
      OR    2.243    89.72    8.500            OR    3.999   159.97    0.500
EV2   IR    1.347    38.72    8.500            IR    2.884    82.91    0.500
      OR    2.434    69.98    8.500            OR    4.814   138.39    0.500
EV3   IR    0.938    40.34    7.650            IR    1.877    80.70    4.250
      OR    1.693    72.79    7.650            OR    3.133   134.70    4.250



******************************************************************************
*                            CONTROLLING RATINGS                             *
******************************************************************************


     VEHICLE TYPE              IR           OR

     H20   LOADING (TONS)    28.20 F     50.96 F
     HS20  LOADING (TONS)    50.76 F     91.73 F
     ML80  LOADING (TONS)    38.98 F     70.44 F
     TK527 LOADING (TONS)    49.65 F     89.72 F
     EV2   LOADING (TONS)    38.72 F     69.98 F
     EV3   LOADING (TONS)    40.34 F     72.79 F

     F = FLEXURAL RATING        S = SHEAR RATING
```

3.7.8.3 Exterior Girder Load Rating Analysis

3.7.8.3.1 PS3 Input Parameters

Exterior beam 1 was selected and will be rated using PS3. Many of the PS3 input parameters can be left blank if the default values are correct. Only the required input values are discussed below. Refer to the PS3 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02744342562978
- Description = EXTERIOR BEAM 1
- Live Load = Blank for H, HS, ML80, TK527, EV2, and EV3 vehicles.
- Output = 0 for normal output
- Roadway Width = 28.00 ft
- I or F = F for Interior Beam
- Design = R for rating only
- Skew Correction Factor = 1.155 (See below). Per DM-4 Table 3.23.2(A), in determining end shear on multi-beam bridges, all beams shall be treated like the beam at the obtuse corner, i.e., the adjustment is appliable to all beams and shall be applied to the distribution factor for exterior beams.

  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

  $$\text{SCF} = 1.0 + \left(\frac{12L}{90d}\right)\sqrt{\tan\ \theta}$$

  where,

  Span Length, L = 17.0 ft

  Beam Depth, d = 12 in

  AASHTO Skew Angle, θ = (90° – 56° 00' 00") = 34.0°

  The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

  | | |
  |---|---|
  | $0° \le \theta \le 60°$ | θ = 34.0°, OK |
  | 20 ft ≤ L ≤ 120 ft | L = 17.0 ft, NG…Call OK |
  | 17 in ≤ d ≤ 60 in | d = 12 in, NG…Call OK |
  | 3.5 ft ≤ b ≤ 6 ft | b = 4 ft, OK |
  | $5 \le N_b \le 20$ | $N_b = 9$, OK |

Bridge Cross Section and Loading

- Beam Spacing, S = 4'-0" beam width + ½" gap / 2 = 48.25 in. Use 48.3 in.

- Distribution Factor – Shear
  Per Pub 238 IE 6B.6.3, the exterior beam distribution factor for moment and shear shall use the larger of the LFD Distribution Factor per IP 3.3.2.2 or Lever Rule per AASHTO 3.23.2.3.

LFD Distribution Factor
Per AASHTO Equation 3-11, the moment distribution factor is equal to

$$DF_M = \frac{S}{D}$$

where,

Width of Precast Member, S = 4'-0" = 4.0 ft

Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
Roadway Width, W = 28.0 ft
$N_L$ = 28.0 ft / 12 ft = 2.33 ==> Use 2 lanes

Span Length, L = 17.0 ft

Factor, $K = \sqrt{[(1 + \mu)I/J)]}$ per AASHTO 3.23.4.3
Poisson's Ratio, $\mu = 0.20$ (assumed per AASHTO LRFD 5.4.2.5)
Moment of Inertia, I = 6850.3 $in^4$ per PS3 Output
St.-Venant Torsional Constant, $J = \Sigma[(1 – 0.630t/b)bt^3/3]$ = 23293.4 $in^4$ where b = 48" and t = 12"
$K = \sqrt{[(1 + 0.20) \times 6850.3 \text{ in}^4 / 23293.4 \text{ in}^4]} = 0.594$

W/L = 32.0 ft / 17.0 ft = 1.88 > 1, C = K = 0.594

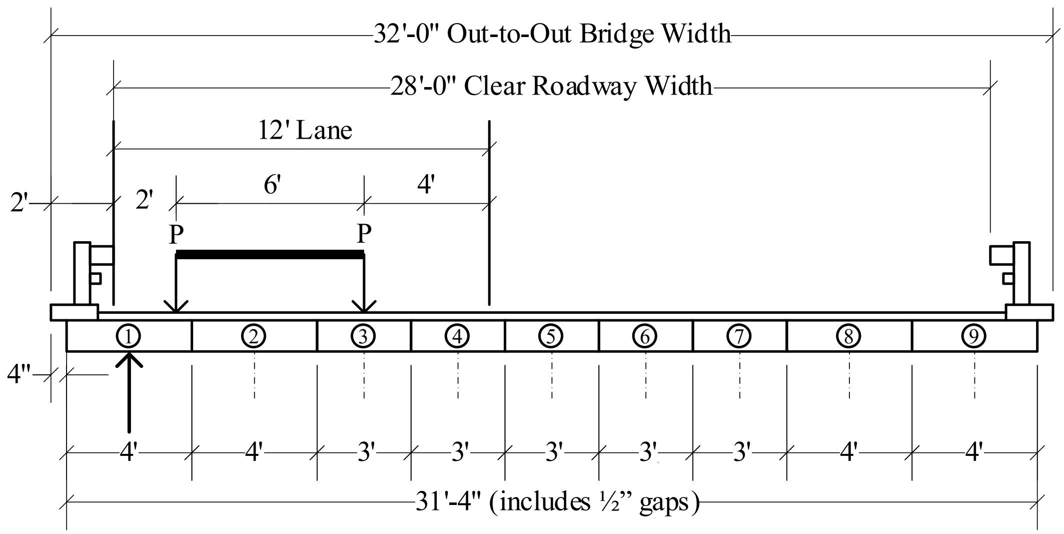
$D = (5.75 – 0.5N_L) + 0.7N_L(1 – 0.2C)^2$ per AASHTO Equation 3-12
$D = (5.75 – 0.5 \times 2) + 0.7 \times 2 \times (1 – 0.2 \times 0.594)^2 = 5.837$

$DF_M$ = S / D = 4.042 / 5.837 = 0.692 wheels x (1 axle/2 wheels) = 0.346 axles

The Range of Applicability checks per AASHTO 3.23.4.3:

| | |
|---|---|
| $\sqrt{(I/J)} \leq 5$ | $\sqrt{I/J} = 0.54$, OK |
| $\theta \leq 45°$ | $\theta = 34°$ ft, OK |

Lever Rule
In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously when the Lever Rule is used. For the one lane condition shown below, the reduction in load intensity factor is 1.00.




1 Lane Loaded, $DF_V$ = P[(4.042' – (2' + 2' – 2.333'))/4.042'] x 1.00 = 0.588 wheels / (2 wheels/axle) = 0.294 axles < 0.346 axles. Therefore,

$DF_V$ = <u>0.346 axles</u>

- Distribution Factor – Moment
Per Pub 238 IE 6B.6.3, the exterior beam distribution factor for moment and shear shall use the larger of the LFD Distribution Factor per IP 3.3.2.2 or Lever Rule per AASHTO 3.23.2.3. Based on the previous calculations, the exterior beam moment distribution factor for multi-beam bridges is $DF_M$ = <u>0.346 axles</u>.

- Distribution Factor – Deflection
Per AASHTO 10.6.4, the distribution factor for deflection is calculated as
$DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
Reduction Factor = 1.00 for 2 lanes per AASHTO 3.12.1
$DF_{Defl}$ = 2 lanes x 1.00 / 9 beams = <u>0.222 lanes/beam</u>

- Uniform Dead Loads – UDLF
UDLF = 0 ksf. There is not any uniform dead load applied to the beam.

- Dead Loads – DL1
The dead loads acting on the non-composite section include girder self-weight. Girder self-weight is calculated by PS3. Per the PS3 User's Manual Section 5.3, adjacent non-composite box beam loads due to curbs, parapets, railing, bituminous surface…etc, can be applied as DL1 loads.

<u>Asphalt Wearing Surface</u>
A 3 ½" asphalt wearing surface is present on the bridge and will be included in the rating analysis. Since there isn't a composite deck, the dead load will be applied based on the tributary width.
WS = 0.140 kcf x (3.5 in/12 in/ft) x (4 in/12 in/ft + 4.00 ft + ¼ in/12 in/ft – 1.5 ft) = 0.117 kips/ft

<u>Roadway Barriers</u>

Per Pub 238 IE 6B.6.1, the distribution of barrier dead load for the exterior beam shall be assumed to support 100% of the barrier dead load. The barrier dead load includes the concrete pad and guiderail accessories.

| | |
|---|---|
| Concrete pad = 0.5 ft x 1.5 ft x 0.150 kcf = | 0.113 kips/ft |
| Steel Post = 0.015 klf x 2.667 ft / 3.125 ft Spacing = | 0.013 kips/ft |
| Steel Spacer = 0.009 klf x 1.083 ft / 3.125 ft Spacing = | 0.003 kips/ft |
| Base Plate = 1 ft x 1 ft x 0.083 ft x 0.490 kcf / 3.125 ft Spacing = | 0.013 kips/ft |
| Guide Rail & Rub Rail = Say 0.008 klf + 0.005 klf = | 0.013 kips/ft |
| Total = | 0.155 kips/ft |

DL1 = 0.117 kips/ft + 0.155 kips/ft x 1.10 for connections = 0.288 kips/ft

- Future Wearing Surface – FWS
  Since there is an asphalt wearing surface present on the bridge, future wearing surface is not included in the rating analysis.
  FWS = 0 kips/ft or blank.

- Dead Loads – DL2
  Load due to barriers and wearing surfaces were entered as DL1 loads.
  DL 2 = 0 kips/ft

- P/S Loss = 0.08 for Stress-Relieved Strands

- Strand L or S = S for Stress-Relieved Strands

Span Lengths

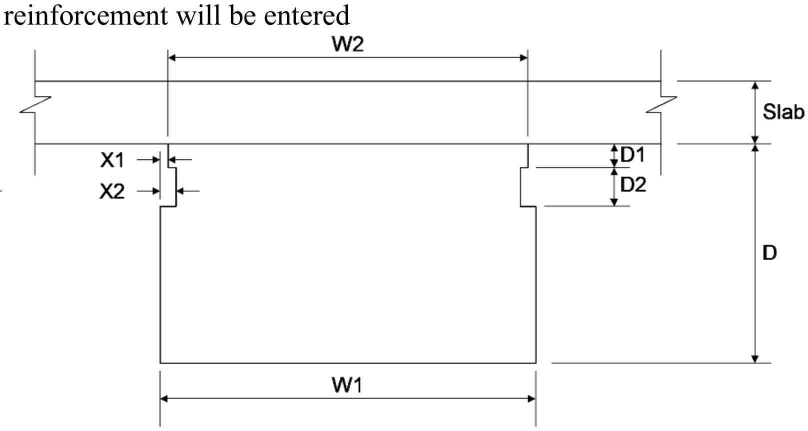
- Span 1 Length = 17.0 ft
- Beam Projection = 9.0 in

Prestress Criteria

- 28-Day Compressive Strength of Beam Concrete = 4.5 ksi
- 28-Day Compressive Strength of Slab Concrete = blank
- Compressive Strength of Beam Concrete at Initial Prestressing = 4.0 ksi
- Ultimate Strength of Prestressing Steel = 250 ksi
- Strand Diameter = 1/4” = 0.25 in
- Number of Strand Rows = 2
- Number of Debonded Lengths = 0
- Stirrup Details = Y indicating stirrup reinforcement will be entered

Prestressed Concrete Beam Dimensions

- Type = P for a plank beam
- Comp = N for a non-composite beam
- Design or D = 12.00 in beam depth

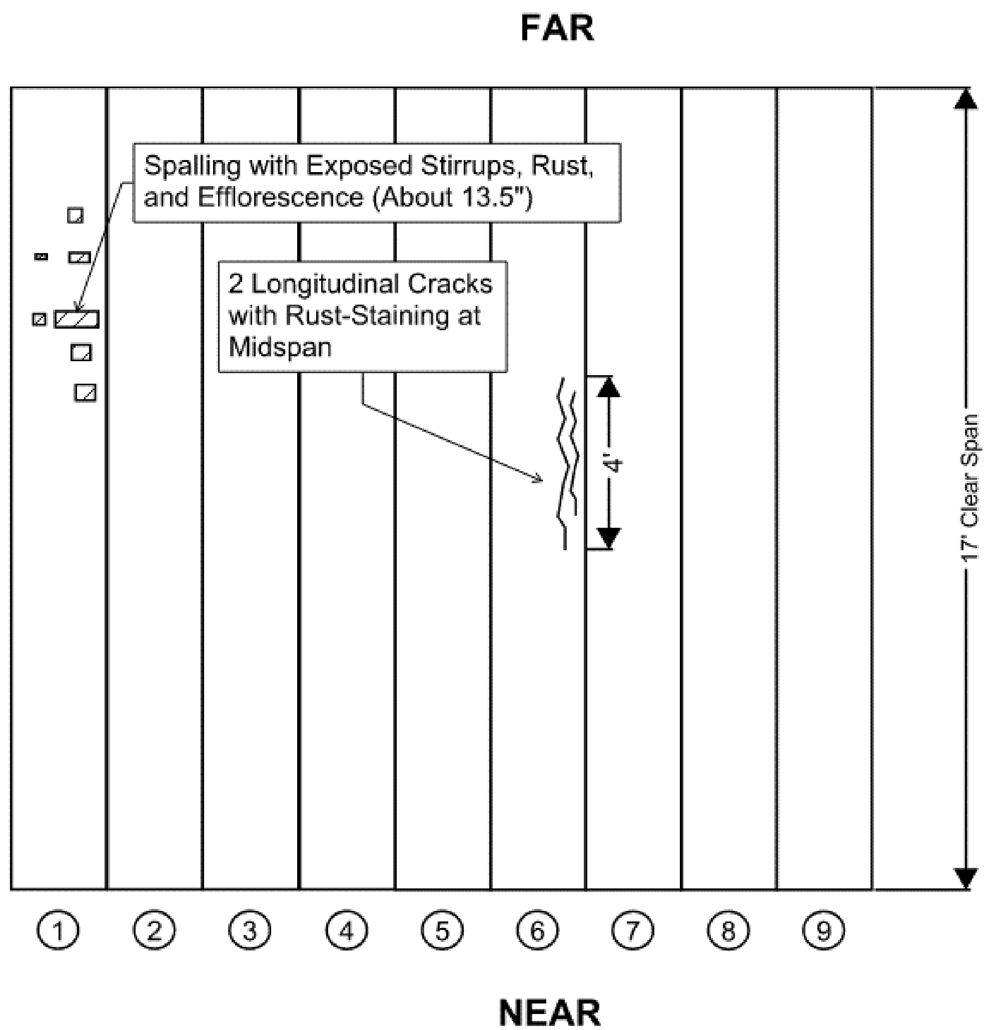


Plank Beam

- W1 = 48.0 in
- D1 = 3 in
- D2 = 4 in
- X1 = 0.375 in
- X2 = 0.75 in
- Slab Thickness = 0 in
- Haunch Thickness = 0 in

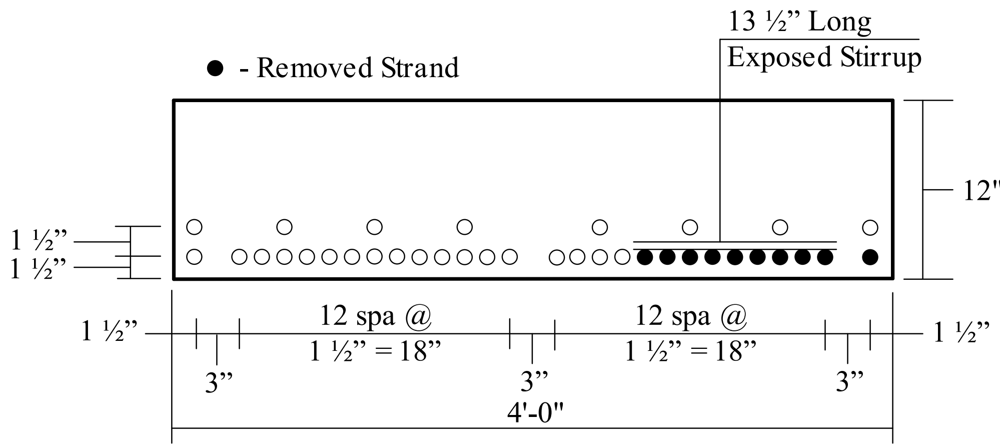
Strand Details

The following sketch was developed based on a field inspection. Beam 1 was noted as having exposed stirrups within the span.




FRAMING PLAN

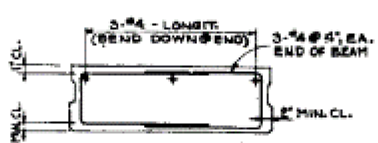
Per Pub 238 IE 6.1.5.3I – Method B – Deteriorated Concrete (3), for exposed shear reinforcement bars, disregard the full strength of strands located in the lower row directly above the exposed section of stirrups for capacity calculations. Due to the 13 ½” length of exposed stirrups measured along the beam width, the strands in this area will be removed. Also, remove the corner strand due to spalling. A total of 10 strands were removed in row 1 for analysis purposes.




- Strand Area = 0.036 in$^2$ for ¼” diameter strand
- G1 = 1.50 in
- G2 = 1.50 in
- R1 = 28 – 10 = 18
- R2 = 8

<u>Stirrup Details</u>

- Spec = “A” (Default)
- Stirrup Area = 0.20 in$^2$ for one leg of a #4 stirrup
- Yield Strength of Reinforcement Stirrups, $f_y$ = 40 ksi
- Location = 0.00 ft
- Spacing = 4.0 in
- Location = 0.67 ft (3 - #4 @ 4”)



REINFORCEMENT- 12" BEAM

**TITLE**

| | | | |
|---|---|---|---|
| Structure ID: | 02-7443-4256-2978 | | |
| Description: | EXTERIOR BEAM 1 | | |
| SLC Level: | | | |
| Live Load: | | | H20, HS20, ML80, TK527, EV2, and EV3 |
| Output: | 0 | | Normal output |
| Impact Factor: | | | |
| Gage Distance: | | ft | |
| Passing Distance: | | ft | |
| Roadway Width: | 28.00 | ft | Per Contract Drawings |
| DLF: | | | |
| LLF: | | | |
| I or F: | F | | Fascia beam |
| Principal: | | | |
| Design: | R | | Rating only problem |
| Skew Correction Factor: | 1.155 | | See Hand Calcs |
| IR Stress Level: | | | |
| AASHTO fc: | | | |

**Bridge Cross-Section and Loading**

| | | | |
|---|---|---|---|
| Beam Spacing: | 48.300 | in | See Hand Calcs |
| Distr Factor -Shear: | 0.346 | | See Hand Calcs |
| Distr Factor -Moment: | 0.346 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.222 | | See Hand Calcs |
| Unit Weight of Deck Concrete: | | | |
| UDLF: | 0.000 | | See Hand Calcs |
| DL1: | 0.288 | kip/ft | See Hand Calcs |
| FWS: | 0.000 | kip/ft | See Hand Calcs |
| DL2: | 0.000 | kip/ft | See Hand Calcs |
| Initial P/S Force: | | kips | |
| Midspan Eccentricity: | | in | |
| End Eccentricity: | | in | |
| P/S Loss %: | 0.08 | | Stress Relieved Strands |
| Drape Point: | -- | | Decimal part of the span |
| Strand L or S: | S | | Stress Relieved Strands |
| Rate FWS?: | | | |

**Span Lengths**

| | | | |
|---|---|---|---|
| Span Length 1: | 17.00 | ft | Per Contract Drawings |
| Beam Projection: | 9.000 | in | Per Contract Drawings |

**Prestress Criteria**

| | | | |
|---|---|---|---|
| Beam Conc $f'_{cb}$: | 4.5 | ksi | Per Contract Drawings |
| Slab Conc $f'_{cs}$: | --- | ksi | Per Contract Drawings |
| Conc Init $f'_{ci}$: | 4.0 | ksi | Per Contract Drawings |
| Strand Ult $f'_{s}$: | 250 | ksi | Per Contract Drawings |
| Strand Diameter: | 0.2500 | in | Per Contract Drawings |
| No. of Rows: | 2 | | Per Contract Drawings |
| No. Lx: | 0 | | Per Contract Drawings |
| St. Det: | Y | | Stirrup Details |

**Beam Dimensions**

| | | | |
|---|---|---|---|
| Beam Type: | P | | Plank Beam |
| Composite: | N | | Non-Composite |
| Designation or D: | 12 | | Per Contract Drawings |
| W1: | 36 | in | Per Contract Drawings |
| W2: | | in | |
| W3: | | in | |
| T1: | | in | |
| T2: | | in | |
| B1: | | in | |
| B2: | | in | |
| B3: | | in | |
| B4: | | in | |
| D1: | 3 | in | Per Contract Drawings/Standard Drawings |
| D2: | 4 | in | Per Contract Drawings/Standard Drawings |
| X1: | 0.375 | in | Per Contract Drawings/Standard Drawings |
| X2: | 0.75 | in | Per Contract Drawings/Standard Drawings |
| Slab Thickness: | 0.00 | in | Per Contract Drawings |
| Haunch: | 0 | in | |

**Strands**

| | | | |
|---|---|---|---|
| Strand Area: | 0.036 | $in^2$ | Per Contract Drawings/See Hand Calcs |
| G1: | 1.500 | in | Per Contract Drawings/See Hand Calcs |
| G2: | 1.500 | in | Per Contract Drawings/See Hand Calcs |
| R1: | 18 | | Per Contract Drawings/See Hand Calcs |
| R2: | 8 | | Per Contract Drawings/See Hand Calcs |
| R3: | | | |
| R4: | | | |

**Debonded**

| | | | |
|---|---|---|---|
| Debonded Length: | 1.229 | ft | See Hand Calcs |
| R1: | 1 | | |
| No. Strands: | 10 | | |
| R2: | 2 | | |
| No. Strands: | 6 | | |
| R3: | | | |
| No. Strands: | | | |
| R4: | | | |
| No. Strands: | | | |

**Stirrups**

| | | | |
|---|---|---|---|
| Spec: | A | | Shear values are to be computed per AASHTO 9.20 |
| Area: | 0.200 | $in^2$ | Per Contract Drawings |
| $f'_y$: | 40 | ksi | Per Contract Drawings |
| Location 1: | 0.00 | ft | Per Contract Drawings |
| Spacing 1: | 4.000 | in | Per Contract Drawings |
| Location 2: | 0.67 | ft | Per Contract Drawings |
| Spacing 2: | | in | |
| Location 3: | | ft | |

3.7.8.3.2 PS3 Output

```
****************************************************************************
*                                                                          *
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*                                                                          *
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LFD Prestressed Concrete Girder Design and Rating 330740

PROGRAM P4353050 06/17/2024 13:20
VERSION 3.6.0.5 LAST UPDATED 12/22/2023 DOCUMENTATION 12/2023

INPUT: Exterior PS Plank Beam PS3 Analysis.dat

STRUCTURE ID - 02744342562978 - EXTERIOR BEAM 6

| SLC LEVEL | LIVE LOAD | OUT-PUT | IMPACT FACTOR | GAGE DISTANCE | PASSING DISTANCE | ROADWAY WIDTH | LOAD FACTORS DLF | LLF | I OR F |
|---|---|---|---|---|---|---|---|---|---|
| | | 0 | 0.000 | 0.0 | 0.0 | 28.00 | 0.00 | 0.00 | F |

| PRINCIPAL STRESSES | DESIGN | SKEW CORRECTION FACTOR | IR STRESS LEVEL | AASHTO FC |
|---|---|---|---|---|
| | R | 1.155 | 0.000 | |

BRIDGE CROSS SECTION AND LOADING

| BEAM SPACING | DISTRIBUTION FACTORS SHEAR | MOMENT | DEFLECTION | UNIT WEIGHT OF DECK CONCRETE | DEAD LOADS UDLF | DL1 | FWS | DL2 | INITIAL P/S FORCE |
|---|---|---|---|---|---|---|---|---|---|
| 48.3 | 0.346 | 0.346 | 0.222 | 0.0000 | 0.0000 | 0.288 | 0.000 | 0.000 | 0.000 |

| ECCENTRICITY MIDSPAN | END | P/S LOSS % | XDRAPE | LEHIGH LOSS METHOD T0 | TS | TD | IC | MFG | IST | STRAND L or S | RATINGS w/ & w/o FWS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.000 | 0.000 | 0.08 | 0.0000 | 0 | 0 | 0 | 0 | 0 | 0 | S | |

SPAN LENGTHS (SIMPLE)

| SPAN # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | BEAM PROJ |
|---|---|---|---|---|---|---|---|---|---|
| LENGTH | 17.00 | | | | | | | | 9.000 |

PRESTRESS CRITERIA

| BEAM CONC F'CB | SLAB CONC F'CS | CONC INIT F'CI | STEEL INIT FSI | STEEL YIELD Fy | STEEL ULT F'S | INITIAL ALLOWABLE COMP FCI | TENS FTI | DRP/DBND FTFD |
|---|---|---|---|---|---|---|---|---|
| 4.500 | 0.000 | 4.000 | 0.0 | 0.0 | 250.0 | 0.000 | 0.000 | 0.000 |

| FINAL ALLOWABLE COMP FC | TENS FT | SLAB FCS | ALLOW SHEAR VHA | OR STRESS LEVEL | STEEL E | MODULAR RATIOS DES | ULT | CREEP FACTOR | EST. % LOSS | STRAND DIAMETER |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0 | 0.000 | 0.000 | 0.0 | 0.0 | 0.2500 |

| NUMBER OF ROWS | NUMBER OF Lx | STIRRUP DETAILS |
|---|---|---|
| 2 | 0 | Y |

PRESTRESSED CONCRETE BEAM DIMENSIONS

| TYPE | COMP | D | W1 | W2 | W3 | T1 | T2 |
|---|---|---|---|---|---|---|---|
| P | N | 12.000 | 48.000 | 0.000 | 0.000 | 0.000 | 0.000 |

| B1 | B2 | B3 | B4 | D1 | D2 | X1 | X2 | SLAB THICK | HAUNCH |
|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.00 | 0.00 | 0.00 | 3.00 | 4.00 | 0.375 | 0.750 | 0.00 | 0.00 |

STRAND DETAILS

| AREA | G1 | G2 | R1 | R2 | R3 | R4 | R5 | R6 | R7 | R8 | R9 | R10 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.036 | 1.50 | 1.500 | 18 | 8 | | | | | | | | |

STIRRUP DETAILS

| SPEC. FOR ANAL/RATE | STIRRUP AREA | FSY | LOCATION | SPACING | LOCATION | SPACING | LOCATION | SPACING |
|---|---|---|---|---|---|---|---|---|
| A | 0.200 | 40 | 0.00 | 4.000 | 0.67 | 0.000 | 0.00 | 0.000 |
| | | | 0.00 | 0.000 | 0.00 | 0.000 | 0.00 | 0.000 |

DEFAULT VALUES

| GAGE DIST | PASS DIST | DLF | LLF | UNIT WT DK CONC | ST INI FSI | ST YLD Fy | AASHTO FC |
|---|---|---|---|---|---|---|---|
| 6.0 | 4.0 | 1.30 | 2.17 | 0.150 | 175.0 | 212.5 | N |

| COMP FC | TENS FT | ALLOW SHR-VHA | OR STR LEVEL | IR STR LEVEL | STEEL E | CREEP FACTOR |
|---|---|---|---|---|---|---|
| 1.800 | 0.201 | 0.300 | 0.900 | 0.800 | 28000 | 1.6 |

```
*******************************************************************************
*                          RATING OF A FACIA BEAM                             *
*******************************************************************************
```

BASIC BEAM SECTION PROPERTIES

| DEPTH IN | AREA IN.2 | WEIGHT LBS/FT | M OF I IN.4 | N.A. TO TOP YT IN. | N.A. TO BOT YB IN | Z TOP IN.3 | Z BOT IN.3 |
|---|---|---|---|---|---|---|---|
| 12.00 | 567.8 | 591.41 | 6850.3 | 6.03 | 5.97 | 1136.3 | 1147.1 |

UNIFORM DEAD LOADS ACTING ON GIRDER (KIPS/FT)

| GIRDER WEIGHT | FORMWORK WEIGHT | INPUT DL1 | FUTURE WEARING SURFACE | INPUT DL2 | TOTAL DL1 | TOTAL DL2 |
|---|---|---|---|---|---|---|
| 0.5914 | 0.0000 | 0.2880 | 0.0000 | 0.0000 | 0.8794 | 0.0000 |

DEAD LOAD AND LIVE LOAD REACTIONS

| DL1 REACTION | DL2 REACTION | IMPACT FACTOR | LL+I H20 REACTION | LL+I HS20 REACTION | LL+I ML80 REACTION | LL+I TK527 REACTION |
|---|---|---|---|---|---|---|
| 7.5 | 0.0 | 1.300 | 17.4 T | 19.6 T | 24.6 T | 22.7 T |

| LL+I EV2 REACTION | LL+I EV3 REACTION |
|---|---|
| 18.9 T | 28.4 T |

PRESTRESSING FORCE (STRAND PATTERN KNOWN)

| INITIAL | LOSS % | EFFECTIVE | NO. OF STRANDS | ECCENTRICITY | C.G.S. |
|---|---|---|---|---|---|
| 163.800 | 20.00 | 131.040 | 26 | 4.010 | 1.962 |

```
******************************************************************************
*                                RATING SUMMARY                              *
******************************************************************************
```

| | | FLEXURAL RATINGS (BASED ON MOMENT) | | | | SHEAR RATINGS (AASHTO) | | |
|---|---|---|---|---|---|---|---|---|
| LOAD | | FACTOR | TONS | LOCATION FROM CL BRG | | FACTOR | TONS | LOCATION FROM CL BRG |
| H20 | IR | 0.872 | 17.43 | 8.500 | IR | 2.329 | 46.59 | 4.250 |
| | OR | 1.663 | 33.27 | 8.500 | OR | 3.888 | 77.76 | 4.250 |
| HS20 | IR | 0.872 | 31.38 | 8.500 | IR | 2.304 | 82.96 | 0.500 |
| | OR | 1.663 | 59.88 | 8.500 | OR | 3.847 | 138.48 | 0.500 |
| ML80 | IR | 0.658 | 24.10 | 8.500 | IR | 1.748 | 64.07 | 3.400 |
| | OR | 1.255 | 45.98 | 8.500 | OR | 2.919 | 106.94 | 3.400 |
| TK527 | IR | 0.767 | 30.69 | 8.500 | IR | 1.929 | 77.16 | 3.400 |
| | OR | 1.464 | 58.56 | 8.500 | OR | 3.220 | 128.80 | 3.400 |
| EV2 | IR | 0.833 | 23.94 | 8.500 | IR | 2.226 | 64.00 | 4.250 |
| | OR | 1.589 | 45.68 | 8.500 | OR | 3.716 | 106.84 | 4.250 |
| EV3 | IR | 0.582 | 25.01 | 7.650 | IR | 1.433 | 61.61 | 4.250 |
| | OR | 1.107 | 47.58 | 7.650 | OR | 2.392 | 102.85 | 4.250 |

```
******************************************************************************
*                             CONTROLLING RATINGS                            *
******************************************************************************
```

| VEHICLE TYPE | IR | OR |
|---|---|---|
| H20 LOADING (TONS) | 17.43 F | 33.27 F |
| HS20 LOADING (TONS) | 31.38 F | 59.88 F |
| ML80 LOADING (TONS) | 24.10 F | 45.98 F |
| TK527 LOADING (TONS) | 30.69 F | 58.56 F |
| EV2 LOADING (TONS) | 23.94 F | 45.68 F |
| EV3 LOADING (TONS) | 25.01 F | 47.58 F |

F = FLEXURAL RATING S = SHEAR RATING

## 3.8 NEXT BEAMS

This Section covers the rating of NEXT Beams.

### 3.8.1 Policies and Guidelines

Most NEXT beam bridges can be rated by analytical methods based on design plans and field measurements.

If plans are not available, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. Given that this is a newer bridge type, the material information should generally be available and the historical information in Pub. 238 IP 3.7.2 will not be applicable.

### 3.8.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating.

#### 3.8.2.1 LFR or ASR Method

Given that the majority of these bridges were built 2011 and after, LFD and ASD design methods would not have been used. Therefore, these bridges would need to be rated with LRFR.

#### 3.8.2.2 LRFR Method

For NEXT beam bridges designed with LRFD, load ratings should be performed using LRFD in PSLRFD.

### 3.8.3 Live Load and Dead Load Distribution

#### 3.8.3.1 LFR or ASR Method

Given that the majority of these bridges were built 2010 and after, LFD and ASD design methods would not have been used. Therefore, these bridges would need to be rated with LRFR.

#### 3.8.3.2 LRFR Method

For NEXT beam bridges designed with LRFD, distribution factors may be computed in PSLRFD. For analysis outside of the program, distribution factors should be computed as per the procedure outlined in the PSLRFD User’s Manual, Section 3.5.4.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO distribution factors. See DM-4 Table 4.6.2.2.3c-1 for the formulas for the skew correction factor.

### 3.8.4 Resources Available

There are no standard drawings available for NEXT Beam bridges.

### 3.8.5 Modeling Section Properties and Deterioration

See Section 2.5.1.2 for a discussion of modeling section loss.

See Pub. 238, Section IE 6.1.5.3I for guidance on removing strands from analysis based on various defects in prestressed beams. In addition, see Pub. 238, Appendix IE 04-D for an example of adjusting strand patterns based on this guidance.

### 3.8.6 Standard Practices

This Section is reserved for future use.

### 3.8.7 Common QA Findings

This Section is in development and will be provided in future editions of this manual.

### 3.8.8 Sample Load Rating

Sample load ratings are in development and will be provided in future editions of this manual.

## 3.9 STEEL GIRDER/MULTI-GIRDER

This Section covers the rating of steel girder/multi-girder bridges including: simple spans, continuous spans, and curved girders. The steel sections referenced may be built-up or rolled sections. There are situations for all bridge types covered in which the superstructure is composite with the deck and non-composite with the deck and these are both covered by this Section.

### 3.9.1 Policies and Guidelines

Most steel girder bridges can be rated by analytical methods based on design plans or field measurements.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B.5.2.1 for guidance on structural steel, MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.9.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating.

See Sections 2.2.2 and 2.2.3 for further discussion of the appropriate level of analysis (i.e., 1D, 2D, or 3D). This will dictate which software should be used.

See Section 2.6.2.1 for further discussion on applicable Pub. 238, AASHTO Std. Spec. and AASHTO MBE Sections. In addition, Sections 2.6.2.1 and 2.2.3 provides guidance and references for curved and skewed multi-girder bridges.

See Section 2.6.7.2 for discussion on analysis and load rating of bolted splice connections in NSTM members.

#### 3.9.2.1 LFR or ASR Method

PennDOT's BAR7 program can perform a 1D Line Girder analysis and rating for a variety of steel superstructure sections, including: Multigirder Steel Bridge (Bridge Type GGG), Girder-Floorbeam-Stringer Type Bridge ((Bridge Type GFS), Girder-Floorbeam Type Bridge (Bridge Type GFF), Floorbeam-Stringer Type Bridge (Bridge Type FSS), and Multigirder-Floorbeam Type Bridge (Bridge Type GGF).

The program AASHTOWare BrR is suitable for rating both straight and curved girders in LFR and ASR as needed.

#### 3.9.2.2 LRFR Method

For steel girder bridges designed with LRFD on or after 2011, load ratings should be performed using LRFD in STLRFD. See Section 2.6.2.1 for discussion on methods for analyzing curved girders.

When STLRFD cannot be utilized, the program AASHTOWare BrR is suitable for rating both straight and curved girders in LRFR.

### 3.9.3 Live Load and Dead Load Distribution

Distribution factors calculated as per the guidance of this Section are for 1D analysis.

#### 3.9.3.1 LFR or ASR Method

Pub. 238 Section 3.3 provides guidance on the distribution of live load to longitudinal girders. Typically, the live load distribution factor is in accordance with the AASHTO Standard Specification.

| | Exterior | | Interior | |
|---|---|---|---|---|
| | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.3.1.2) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.2, Table 3.23.1) |
| Longitudinal Girders and Stringers | Lever Rule* | Lever Rule*# | Lever Rule* | Varies depending on deck type (See AASHTO Table 3.23.1) |
| Floorbeams | Per AASHTO Std. Spec. 3.23.3.1, no transverse distribution of wheel load is assumed when calculating bending moments.<br><br>See AASHTO Std. Spec. 3.23.3.2 when longitudinal stringers are omitted and the floor is supported directly by the floorbeams.<br><br>When utilizing the BAR7 Program, the distribution of live load to the floorbeams is computed by the program. | | | |

Note: In all steel girder configurations, Exterior girder distribution factors shall be computed using Lever Rule for both Moment and Shear.

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

# AASHTO Std. Spec. 3.23.2.3.1.5 is intending to be a minimum distribution factor for design purposes of an exterior beam. For rating purposes, the provision can be ignored and the lever rule utilized.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2.2.3 and Pub. 238 TP 3.3.3.1 for additional discussion of skew.

Dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all girders (AASHTO Standard Specification 3.23.2.3.1.1).

#### 3.9.3.2 LRFR Method

For steel girder bridges designed with LRFD, distribution factors may be computed in STLRFD or AASHTOWare BrR. For analysis outside of these programs, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.2.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO/PennDOT distribution factors. See DM-4 Table 4.6.2.2.3c-1 for the formulas for the skew correction factor.

### 3.9.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document (Name is Link) | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1931-1940 Volume 2 | I Beam Bridges | 12/1/1937 through 10/25/1938 | S-1045 through S-1048 |
| Standards for Old Bridges 1961-1965 Volume 4 | Standard Steel I Beam Bridges | 8/2/1963 | S-2730 through S-2736 |
| Standard for Old Bridges 1965-1972 Vol. 5 | Standard Steel I Beam Bridges | 10/1/1968 through 12/17/1969 | ST-101, ST-111, ST-112 |
| BD-100 Series, 1970 Edition, Change 2 | Allowable Fatigue Stresses in Steel Beams & Girders | 11/1/1972 | BD-106 |
| Online document not available | Steel Girder Bridges Lateral Bracing Criteria and Details | 4/15/2004 | BD-620 |
| Online document not available | Steel Girder Bridges Lateral Bracing Criteria and Details | 9/20/2010 | BD-620 |
| Standards for Bridge Design September 2010 Edition Change 3 | Steel Girder Bridges Lateral Bracing Criteria and Details | 11/21/2014 | BD-620 |
| Standards for Bridge Design April 2016 Edition Change 7 | Steel Girder Bridges Lateral Bracing Criteria and Details | 4/29/2016 (Latest Revision 10/7/2024) | BD-620 |

### 3.9.5 Modeling Section Properties and Deterioration

See Section 2.6.1.2 for a discussion of modeling section loss.

### 3.9.6 Standard Practices

If plans and/or details are unavailable to verify the design, assume that the beams are non-composite with the deck slab.

When using BAR7 for LFR/ASR ratings and there are areas of localized section loss, the remaining section thicknesses should be averaged for the length of the section loss.

When beam ends are encased in concrete and considered stiffened, the web buckling criteria does not apply.

In some situations for Girder-Floorbeam-Stringer bridges there will need to be a more in-depth evaluation of the stringers. When using bridge type 'GFS' for an LFR/ASR rating, BAR7 does not have the ability to evaluate intermittent section loss in the stringers since they must have a uniform cross-section throughout. In order to evaluate the effect of section loss, the stringers must be checked in a multi-girder run (Bridge Type 'GGG' in BAR7) in order to accurately account for section loss in the stringers.

Follow AASHTO MBE 6A.6.9.3 for bracing of the compression flange when the girder does not act compositely with the deck (See statement below in italics taken directly from MBE) This criterion shall be assumed to be applicable to LFR and ASR ratings in addition to LRFR. Diaphragms and cross-frames also provide bracing of girder flanges.

*Compression flanges of sections where the deck is not connected to the steel section by shear connectors in positive flexure may be assumed to be adequately braced by the concrete deck, and the compression flange bracing requirements need not be checked where the top flange of the girder is fully in contact with the deck and no sign of cracking, rust, or separation along the steel-concrete interface is evident.*

The load rating engineer shall use their best judgment in determining the effectiveness of timber decks, open grid steel deck, corrugated deck, and other non-concrete decks in providing bracing of the top flange of a girder. The amount and condition of the connections between the girders and deck shall be considered. Visual observation in the field can also be utilized to help determine if the top flange is braced under heavy vehicle live loads.

K-frame legs are considered part of the superstructure and must be considered as part of the load rating. A PennDOT approved analysis program will need to be used to accurately determine loads acting on the legs.

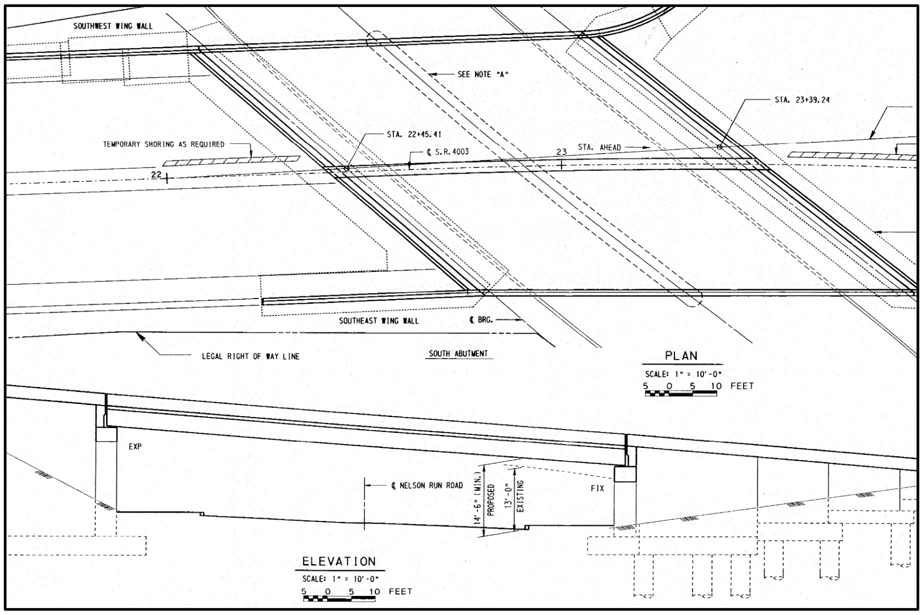
### 3.9.7 Common QA Findings

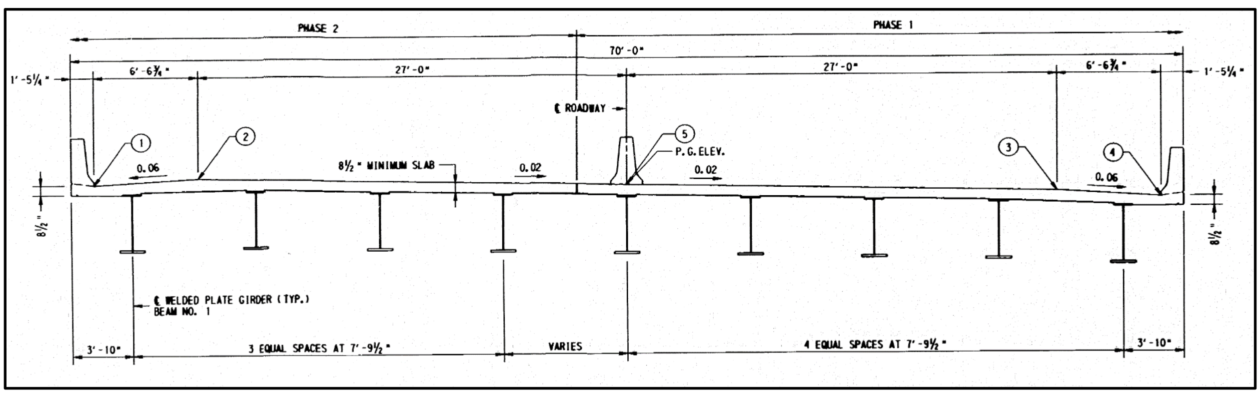
This section is in development and will be provided in future editions of this manual.

### 3.9.8 Sample Load Rating – Steel Multi-Girder Bridge

This Section contains sample load rating calculations for an interior and exterior girder of a single span composite steel girder structure shown below. In addition, an interior girder with section loss will also be rated. The analysis will be performed using PennDOT's Bridge Analysis and Rating (BAR7) Program, Version 7.15.0.0 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part-IV (DM-4).

The single span superstructure consists of an 8 ½" reinforced concrete deck supported on nine (9) welded plate girders. The steel girders were fabricated from structural steel conforming to AASHTO M270 (ASTM A709) Grade 50 designation and are composite with the concrete deck. Details from the Contract Drawings are shown below.


SOUTHWEST WING WALL
SEE NOTE "A"
STA. 23+39.24
TEMPORARY SHORING AS REQUIRED
STA. 22+45.41
℄ S.R. 4003
STA. AHEAD
SOUTHEAST WING WALL
℄ BRG.
LEGAL RIGHT OF WAY LINE
SOUTH ABUTMENT
PLAN
SCALE: 1" = 10'-0"
5 0 5 10 FEET
EXP
℄ NELSON RUN ROAD
14'-6" (MIN.) PROPOSED
13'-0" EXISTING
FIX
ELEVATION
SCALE: 1" = 10'-0"
5 0 5 10 FEET


PHASE 2
PHASE 1
70'-0"
1'-5¼"
6'-6¾"
27'-0"
27'-0"
6'-6¾"
1'-5¼"
℄ ROADWAY
P.G. ELEV.
0.06
8½" MINIMUM SLAB
0.02
0.02
0.06
8½"
8½"
℄ WELDED PLATE GIRDER (TYP.)
BEAM NO. 1
3'-10"
3 EQUAL SPACES AT 7'-9½"
VARIES
4 EQUAL SPACES AT 7'-9½"
3'-10"

## GENERAL NOTES

- DESIGN SPECIFICATIONS:

- PROVIDE MATERIALS AND WORKMANSHIP IN ACCORDANCE WITH THE PENNSYLVANIA DEPARTMENT OF TRANSPORTATION SPECIFICATIONS PUBLICATION 408/94, ANSI/AASHTO/AWS D1.5-88 BRIDGE WELDING CODE AND CONTRACT SPECIAL PROVISIONS.
- DESIGN SPECIFICATIONS: AASHTO 1992 "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES " ( INCLUDING 1995 INTERIM SPECIFICATIONS) , AND AS SUPPLEMENTED BY THE DESIGN MANUAL, PART 4, STRUCTURES.
- DESIGN IS IN ACCORDANCE WITH THE STRENGTH DESIGN METHOD (LOAD FACTOR DESIGN).
- LIVE LOADS: HS25 LOADING OR 125% OF THE ALTERNATE MILITARY LOADING, OR P-82 (204K PERMIT LOAD) WITH THE AASHTO GROUP I B LOADING (AT OPERATING LEVEL).
- LIVE LOAD DISTRIBUTION TO GIRDERS IS BASED UPON AASHTO METHOD.
- DEAD LOADS: INCLUDES 30 LBS. PER SQ. FT. FOR FUTURE WEARING SURFACE ON THE DECK SLAB AND 15 LBS. (WT. OF FORMS + WT. OF CONCRETE IN VALLEYS OF FORMS) PER SQ. FT. FORMED USING PERMANENT METAL DECK FORMS.

GENERAL:

- USE CLASS AAA CEMENT CONCRETE IN THE DECK SLAB.
- USE CLASS AA CEMENT CONCRETE IN DECK PARAPETS, WINGWALL PARAPETS, MEDIAN BARRIER, CHEEKWALLS, ABUTMENT BACKWALLS, AND ABUTMENT CAPS.
- USE CLASS A CEMENT CONCRETE IN WINGWALLS AND ALL FOOTINGS.
- PROVIDE GRADE 60 REINFORCING STEEL BARS THAT MEET THE REQUIREMENTS OF ASTM A615 FOR BILLET- STEEL, ASTM A616 FOR RAIL STEEL OR ASTM A617 FOR AXLE STEEL. DO NOT USE RAIL STEEL (A616) IN BRIDGE ABUTMENTS, BEAMS AND WHERE BENDING OR WELDING OF THE REINFORCEMENT BARS IS INDICATED.
- PROVIDE 2" CONCRETE COVER ON REINFORCEMENT BARS EXCEPT AS NOTED.
- PROVIDE MINIMUM LAP AND EMBEDMENT LENGTH OF REINFORCEMENT IN ACCORDANCE WITH 1992 AASHTO OR 30 DIAMETERS WHICHEVER IS GREATER. LAP LENGTH NOT SHOWN CORRESPOND TO THE FOLLOWING:

| REBAR NO. | LAP LENGTH (INCHES) |
|---|---|
| 4 | 15 |
| 5 | 19 |
| 6 | 23 |

- WELDING OF REINFORCEMENT BARS DURING FABRICATION OR CONSTRUCTION WILL NOT BE PERMITTED UNLESS SPECIFIED. OR PERMITTED BY THE BRIDGE ENGINEER.
- RAKE FINISH ALL HORIZONTAL CONSTRUCTION JOINTS EXCEPT AS INDICATED.
- CHAMFER EXPOSED CONCRETE EDGES 1" X 1" EXCEPT AS NOTED.
- USE EITHER PERMANENT METAL DECK FORMS OR REMOVABLE FORMS TO CONSTRUCT THE DECK SLAB.
- DECK SLAB THICKNESS INCLUDES ½ INCH INTEGRAL WEARING SURFACE.
- SUPERSTRUCTURE DIMENSIONS SHOWN ARE FOR A NORMAL TEMPERATURE OF 68-DEGREES F.
- ABUTMENT BACKWALLS MAY BE PLACED UP TO A CONSTRUCTION JOINT BELOW THE LEVEL OF THE BOTTOM OF THE DECK SLAB PRIOR TO CONSTRUCTION OF THE SUPERSTRUCTURE.
- EPOXY-COAT ALL REINFORCING BARS IN THE DECK SLAB, PARAPETS, ABUTMENT BACKWALLS AND ABUTMENT SEAT BARS. ALSO EPOXY-COAT SUBSTRUCTURE REINFORCEMENT BARS AS INDICATED.
- VERIFY ALL DIMENSIONS AND GEOMETRY OF THE EXISTING STRUCTURE IN THE FIELD AS NECESSARY FOR PROPER FIT OF THE PROPOSED CONSTRUCTION.
- APPLY PROTECTIVE COATING FOR REINFORCED CONCRETE SURFACES TO THE ENTIRE BRIDGE DECK AND PARAPETS AND PENETRATING SEALER TO THE SUBSTRUCTURE AS SHOWN OR DIRECTED BY THE ENGINEER.

WELDED PLATE GIRDERS

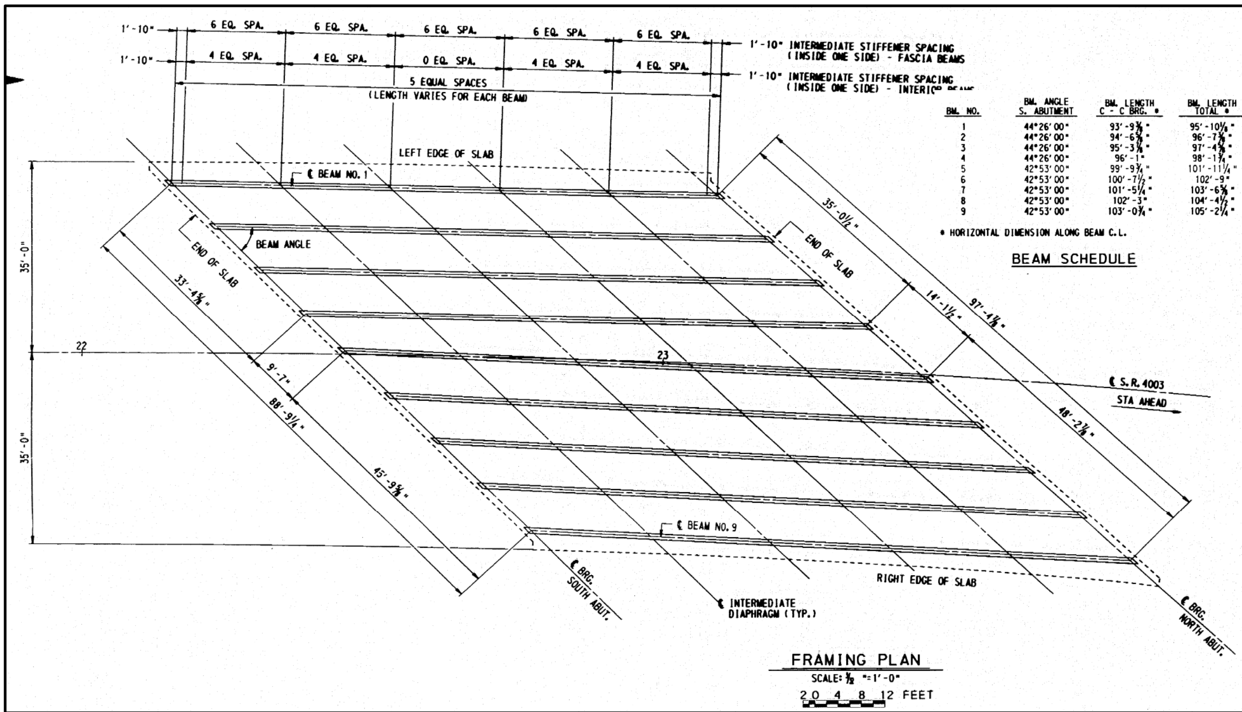
- PROVIDE STRUCTURAL STEEL CONFORMING TO AASHTO M270 (ASTM A709) GRADE 50 DESIGNATION.
- SUPPORT DECK SLAB OVERHANG FORMS FROM THE BOTTOM FLANGE OF THE FASCIA GIRDER.
- THE CONTRACTOR IS REQUIRED TO CHECK THE NEED FOR TEMPORARY BRACING BETWEEN THE EXTERIOR GIRDER AND THE ADJACENT INTERIOR GIRDER TO PREVENT GIRDER ROTATION DURING DECK PLACEMENT. SUBMIT THESE CALCULATIONS ALONG WITH THE DRAWINGS AND CALCULATIONS OF THE TEMPORARY BRACING SCHEME TO THE DEPARTMENT FOR APPROVAL.
- FATIGUE ANALYSIS IS BASED ON THE FOLLOWING NUMBER OF LOADING CYCLES: TRUCK LOADING 500,000 CYCLES LANE LOADING 100,000 CYCLES
- IF BEAMS (GIRDERS) CANNOT BE SHIPPED IN THE LENGTHS SHOWN ON THE PLANS FIELD SPLICES WILL BE PERMITTED AT THE REQUEST OF THE CONTRACTOR, BUT NO COMPENSATION WILL BE ALLOWED.
- REAMING OF FIELD SPLICES IS REQUIRED IN THE FABRICATION SHOP.
- DO NOT USE FORM SUPPORT SYSTEMS WHICH WILL CAUSE UNACCEPTABLE OVERSTRESS OR DEFORMATION TO PERMANENT BRIDGE MEMBERS.
- ALL FASTENERS ARE ⅞" DIAMETER MECHANICALLY GALVANIZED H.S. BOLTS EXCEPT AS NOTED.
- DO NOT MAKE WELDS BY MANUAL SHIELDED METAL ARC PROCESS.
- PAINT STRUCTURAL STEEL IN ACCORDANCE WITH SECTION 1060 PUBLICATION 408/94. ALL AREAS OF MATING SURFACES TO BE PRIMED ONLY IN SHOP. PAINTED IN FIELD AFTER INSTALLATION.
- BRACE STEEL FRAMEWORK IN LONGITUDINAL AND LATERAL DIRECTIONS UNTIL MEMBERS ARE IN STABLE (FINAL BRACED) CONDITION. SUBMIT PLAN FOR APPROVAL.
- ALL STRUCTURAL STEEL REQUIRES CHARPY V-NOTCH (CVN) TESTING. PROVIDE STEEL CONFORMING TO THE SUPPLEMENTAL IMPACT PROPERTIES FOR ZONE 2 AS GIVEN IN THE AASHTO MATERIAL SPECIFICATIONS FOR THESE MEMBERS.
- BEARING AREAS SHALL BE PREPARED AS SPECIFIED IN SECTION 1001.3 (K)8 OF PUB. 408/94.
- SET ANCHOR BOLTS IN PREFORMED HOLES. FILL THE PREFORMED HOLES WITH NON-SHRINK GROUT. FILL THE CLEARANCE BETWEEN ANCHOR BOLTS AND HOLES IN MASONRY PLATES WITH APPROVED NON-HARDENING CAULKING COMPOUND CONFORMING TO SECTION 705.8.
- PROVIDE END WELDED STUD SHEAR CONNECTORS MANUFACTURED FROM STEEL CONFORMING TO ASTM A108.
- PROVIDE EPOXY BONDING COMPOUND BETWEEN NEW AND EXISTING CONCRETE IN ALL LOCATIONS, IN ACCORDANCE WITH SECTION 1001.3 (m).
- THE CONTRACTOR ASSUMES FULL RESPONSIBILITY FOR SECURING A HAULING PERMIT.

EXISTING PLANS:

- COUNTY DRAWINGS #19941, SHEETS 1 THRU 21, DATED 1944.
- DO NOT CONSIDER ANY OF THE DATA ON THE EXISTING STRUCTURE SUPPLIED IN THE ORIGINAL DESIGN DRAWINGS OR MADE AVAILABLE TO YOU BY THE DEPARTMENT OR IT'S AUTHORIZED AGENTS AS POSITIVE REPRESENTATIONS OF ANY OF THE CONDITIONS THAT YOU WILL ENCOUNTER IN THE FIELD.
- THE INFORMATION SHOWN ON THE PLANS FOR THE EXISTING BRIDGE IS NOT PART OF THE PROPOSAL, OR CONTRACT AND IS NOT TO BE CONSIDERED A BASIS FOR COMPUTATION OF THE UNIT PRICES USED FOR BIDDING PURPOSES. PROPOSAL, OR CONTRACT AND IS NOT TO BE CONSIDERED A BASIS FOR COMPUTATION OF THE UNIT PRICES USED FOR BIDDING PURPOSES. THERE IS NO EXPRESSED OR IMPLIED AGREEMENT THAT INFORMATION IS CORRECTLY SHOWN. THE BIDDER IS NOT TO RELY ON THIS INFORMATION, BUT IS TO ASSUME THE POSSIBILITY THAT CONDITIONS AFFECTING COST AND/OR QUANTITIES OF WORK TO BE PERFORMED MAY DIFFER FROM THOSE INDICATED.
- SUBMIT DECK PLACEMENT SCHEDULE TO THE PROJECT ENGINEER FOR APPROVAL. MAINTAIN TRAFFIC IN BOTH DIRECTIONS AT ALL TIMES DURING CONSTRUCTION (ONE LANE EACH DIRECTION).

UTILITY NOTES:

- COORDINATE, LOCATE, AND CONDUCT ALL WORK RELATED TO PUBLIC AND PRIVATE UTILITIES IN ACCORDANCE WITH PUBLICATION 408, SECTION 105.06 AND 107.12.



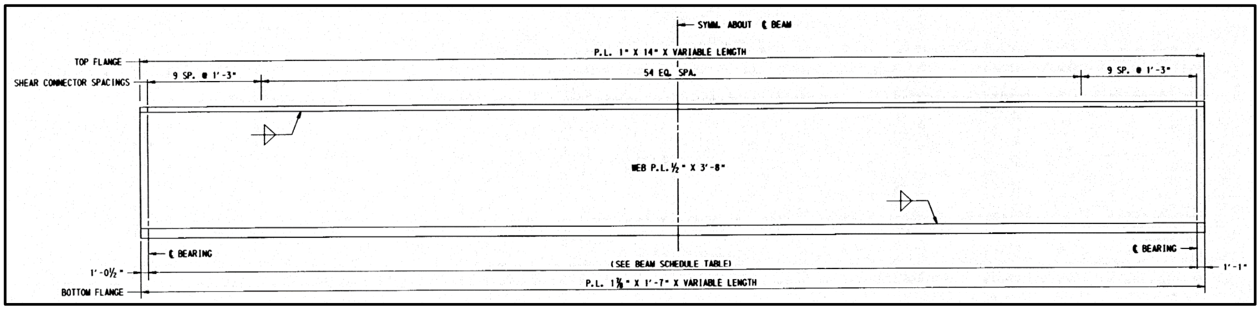

| BM. NO. | BM. ANGLE S. ABUTMENT | BM. LENGTH C - C BRG. * | BM. LENGTH TOTAL * |
|---|---|---|---|
| 1 | 44°26'00" | 93'-9⅜" | 95'-10⅛" |
| 2 | 44°26'00" | 94'-6⅝" | 96'-7⅜" |
| 3 | 44°26'00" | 95'-3⅞" | 97'-4⅝" |
| 4 | 44°26'00" | 96'-1" | 98'-1¾" |
| 5 | 42°53'00" | 99'-9¾" | 101'-11¼" |
| 6 | 42°53'00" | 100'-7½" | 102'-9" |
| 7 | 42°53'00" | 101'-5¼" | 103'-6⅝" |
| 8 | 42°53'00" | 102'-3" | 104'-4½" |
| 9 | 42°53'00" | 103'-0¾" | 105'-2¼" |

* HORIZONTAL DIMENSION ALONG BEAM C.L.

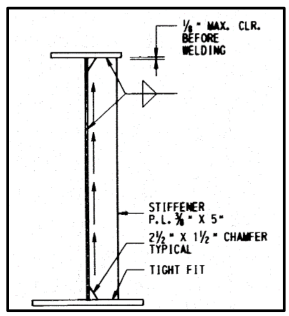
BEAM SCHEDULE

FRAMING PLAN

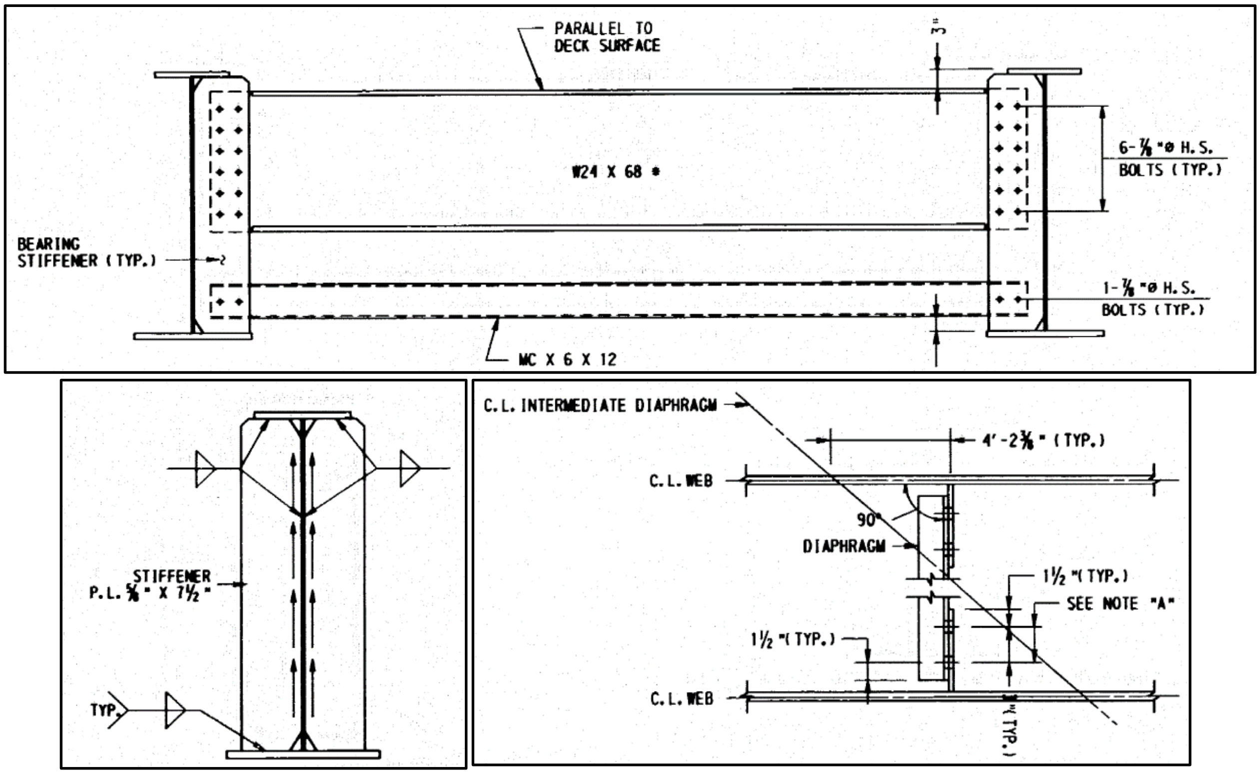
Framing Plan



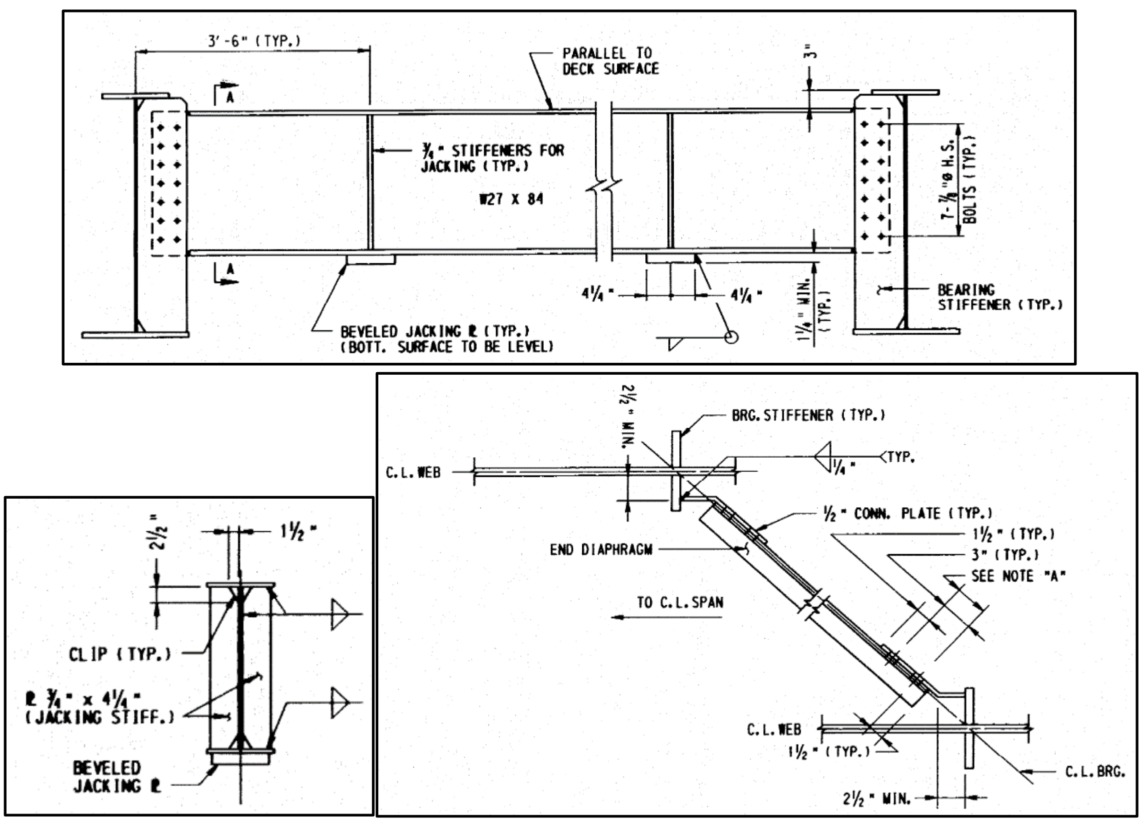

Girder Elevation




Intermediate Stiffener Detail




Intermediate Diaphragm Details




End Diaphragm Details

### 3.9.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

Done By: Date:

Checked By: Date:

| | |
|---|---|
| Structure ID (5A01): | 02-4003-0010-1709 |
| Inspection Date (7A01): | 5/21/2020 |
| Facility Carried (5A08): | McKnight Road |
| Feature Intersected (5A07): | Nelson Run Road |
| Structure Type (6A26 - 6A29): | Single Span Steel Girder Bridge |
| Spans / Members Analyzed: | Span 1/Interior Girder 8 including Section Loss and Exterior Girder 9 |
| Analysis Method: | LFD |
| PennDOT Program / Version: | BAR7 Version 7.15.0.0 |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 2.41 | 48.2 | 4.02 | 80.3 | 4.02 | 80.3 | * | * | M | M |
| HS20 | 2.00 | 72.0 | 3.33 | 120.0 | 3.33 | 120.0 | * | * | M | M |
| ML80 | 1.78 | 65.3 | 2.97 | 108.9 | 2.97 | 108.9 | * | * | M | M |
| TK527 | 1.75 | 70.2 | 2.92 | 117.0 | 2.92 | 117.0 | * | * | M | M |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 2.42 | 69.5 | 4.03 | 115.8 | 4.03 | 115.8 | * | * | M | M |
| EV3 | 1.60 | 68.6 | 2.66 | 114.4 | 2.66 | 114.4 | * | * | M | M |

Comments/Assumptions*:

Superstructure and substructure condition ratings are 5 and 6, respectively with isolated section loss. Therefore, for an ADTT < 500, the SLC Factor equals 1.0 per PennDOT Publication 238, Table IP 4.3.2-1.
Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.
BAR7 analysis includes section loss in the web and bottom flange.
* Controlling Member is Girder 8 in Span 1.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

3.9.8.2 Interior Girder Load Rating Analysis with Section Loss

3.9.8.2.1 BAR7 Input Parameters

Interior girder 8 with web and flange section loss was selected and will be rated using BAR7. Many of the BAR7 input parameters can be left blank. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

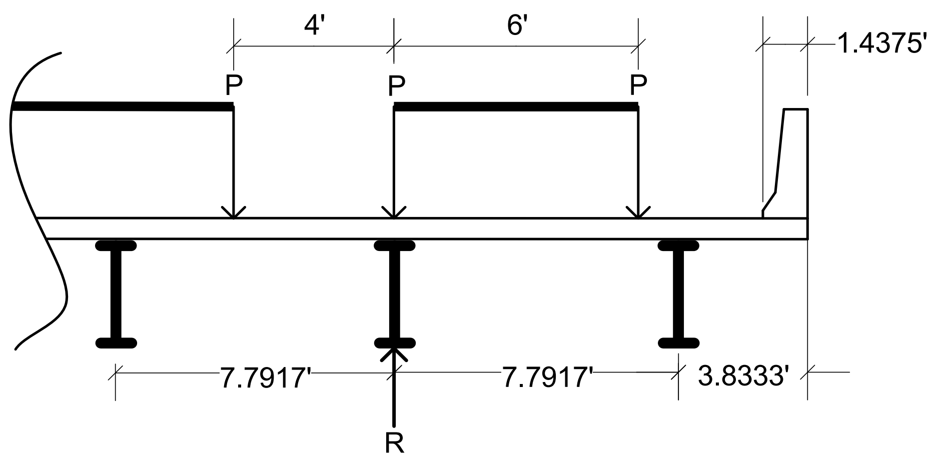
Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 02400300101709
- Description = INTERIOR BEAM 8
- Bridge Type = GGG
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 3 for Load Factor Rating since the structure was designed by the Load Factor Method.
- Concrete Deck = Y

Bridge Cross Section and Loading

- Beam Spacing, S = 7'-9 ½" = 7.7917 ft per Contract Drawings
- Distribution Factor – Shear

  Per AASHTO 3.23.1, the interior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the two-lane condition shown below, the reduction in load intensity factor is 1.0.




$DF_V$ = P[1 + (7.79167' - 6')/7.7917' + (7.7917' – 4')/7.7917'] x 1.0 = 1.716 wheels
$DF_V$ = 1.716 wheels / (2 wheels/axle) = 0.858 axles

- Distribution Factor – Moment

  Per AASHTO Table 3.23.1, the interior beam distribution factor for moment is calculated as S/5.5 for bridges with two or more traffic lanes. In accordance with AASHTO 3.12.2, the reduction in load intensity is not applicable when Table 2.23 if used for moment in longitudinal beams.

$DF_M$ = S / 5.5 = 7.7917 ft / 5.5 = 1.4167 wheels x (1 axle/2 wheels) = 0.708 axles

- Distribution Factor – Deflection
  Per AASHTO 10.6.4, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12’ Lane Width)
  Roadway Width = 70 ft – 2 x 1.4375 ft = 67.125 ft (Conservatively Neglects Median Barrier)
  $N_L$ = 67.125 ft / 12 ft = 5.59 ==> Use 5 lanes
  Reduction Factor = 0.75 for 5 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 5 lanes x 0.75 / 9 beams = 0.417 lanes/beam

- Deck Thickness = 8.5 in

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight, deck slab weight, SIP forms, haunch concrete, intermediate diaphragms, and stiffeners. Girder self-weight and deck slab weight are calculated by BAR7. The remaining loads are calculated below.

  <u>SIP Form Weight</u>
  In accordance with DM-4 3.3.3, an additional dead load of 15 psf of deck area must be included if stay-in-place forms are present. For this example, the form weight is applied over the entire girder spacing.
  SIP Form Weight = 0.015 ksf x 7.7917 ft = 0.117 kips/ft

  <u>Haunch Weight</u>
  Referencing the provided deck drawings, the depth of the deck slab over the centerline of the girder is 9 3/8 in. The depth of the concrete haunch at the centerline of the girder is equal to 9.375 in - 8.5 in = 0.875 in. The top flange width of the girder is 14 in. It is worth noting that the haunch defined on the plans can be used to calculate weight but the haunch height must be field measured to use in the section properties.
  Haunch Weight = 0.875 in x 14 in x 0.150 kcf / 144 $in^2/ft^2$ = 0.013 kips/ft

  <u>Intermediate Stiffeners</u>
  The transverse intermediate stiffeners are 3/8 in x 5 in x 44 in.
  The total number of stiffeners is 4 + 3 + 3 + 4 = 14.
  Stiffeners: 14 x 0.375 in x 5 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ / 102.25 ft = 0.003 kips/ft

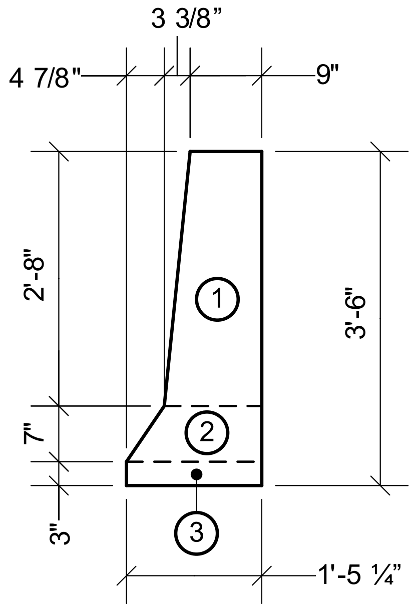
  Small individual plates like the intermediate stiffeners are not always calculated, and frequently a percentage of the girder weight is used (5% to 10%). For this example, these weights were calculated for comparison.

  Total Dead Load 1 = 0.117 + 0.013 + 0.003 = <u>0.133 kips/ft</u>

  For this example, dead loads due to intermediate diaphragms and end diaphragms are entered as concentrated (patch) loads and will be discussed later.

- Dead Loads – DL2
  The dead loads acting on the composite section include outside barriers and median barrier. Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

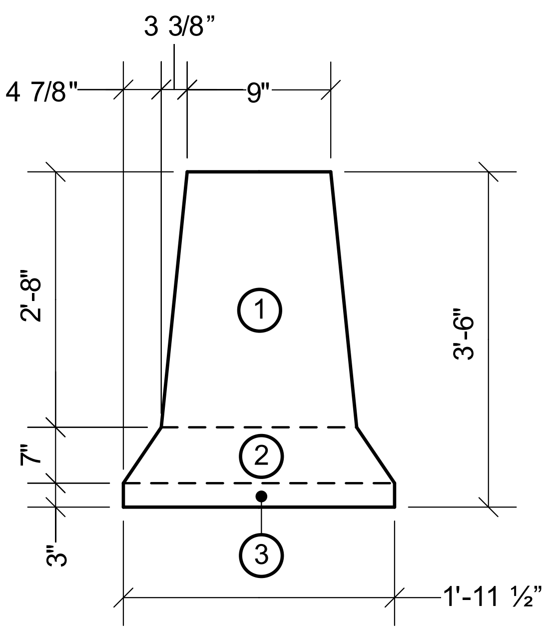
Outside Roadway Barriers




Area 1 = (9 in + 12.375 in)/2 x 32 in = 342.0 $in^2$
Area 2 = (12.375 in + 17.25 in)/2 x 7 in = 103.7 $in^2$
Area 3 = 17.25 in x 3 in = 51.8 $in^2$
Total Area = 497.5 $in^2$

Outside Barrier Weight = 497.5 $in^2$ / 144 $in^2/ft^2$ x 0.150 kcf
Outside Barrier Weight = 0.518 kips/ft

Median Barrier




Area 1 = (9 in + 15.75 in)/2 x 32 in = 396.0 $in^2$
Area 2 = (15.75 in + 23.5 in)/2 x 7 in =137.4 $in^2$
Area 3 = 23.5 in x 3 in = 70.5 $in^2$
Total Area = 603.9 $in^2$

Median Barrier Weight = 603.4 $in^2$ / 144 $in^2/ft^2$ x 0.150 kcf
Median Barrier Weight = 0.629 kips/ft

Total Dead Load 2 = (2 x 0.518 kips/ft + 0.629 kips/ft) / 9 beams = 0.185 kips/ft

- Concrete Compressive Strength, $f'_c$ = 4 ksi for Class AAA Concrete per Drawings and DM-4 8.2 since the bridge was built after 8/16/1989.
- Modular Ratio, n = 8 for Class AAA Concrete per DM-4 8.2

Span Lengths

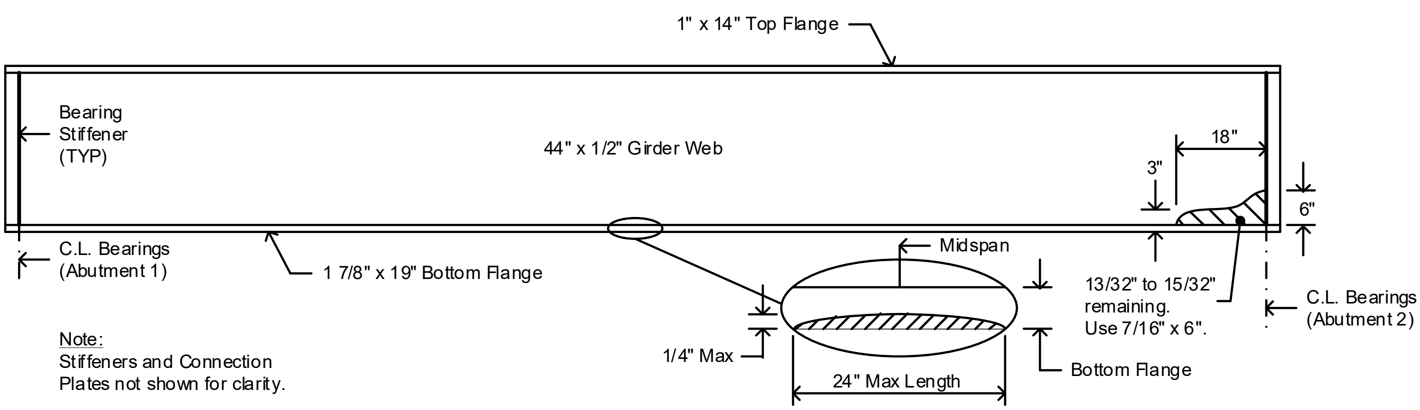
- Continuity = S for Simple Span
- Span 1 Length = 102'-3" = 102.25 ft per Contract Drawings for Beam 8

Concrete Member Properties

- Type = O for Steel Bridge
- Reinforcement Yield Strength, $f_y$ = 60 ksi per Contract Drawings and DM-4 8.2

Steel Member Properties

During the inspection of the structure, section loss was noted on the girder's bottom flange and web as shown below. At midspan, a maximum section loss thickness of ¼" occurs over the entire bottom of the bottom flange. The deterioration extends for a length of 2'-0". A bottom flange loss of ¼" thick x 24" length will be used for analysis at midspan. At Abutment 2, the web has section loss ranging in height from 3" to 6" near the centerline of bearing over a length of 18" from the centerline of bearing. The remaining web section varies between 13/32" and 15/32" in thickness. At the abutment 2 end of the girder, a web section loss acting over a 6" height x 18" length will be used with an average remaining web thickness of 7/16".




**GIRDER 8 ELEVATION**

Girder Section from 0 ft to 50.125 ft

- GFS Type = G for Girder
- Span Number = 1
- Range = 102.25 ft / 2 – 2 ft / 2 = 50.125 ft
- Section Type = P for Plate Girder
- Beam Depth = 44 in
- Web Thickness = 0.5 in

- Top Plate Width = 14 in
- Top Plate Thickness = 1 in
- Bottom Plate Width = 19 in
- Bottom Plate Thickness = 1.875 in
- Composite = Y for a composite superstructure
- Yield Strength of Girder = 50 ksi per Contract Drawings

<u>Girder Section from 50.125 ft to 52.125 ft</u>

- GFS Type = G for Girder
- Span Number = 1
- Range = 102.25 ft / 2 + 2 ft / 2 = 52.125 ft
- Section Type = P for Plate Girder
- Beam Depth = 44 in
- Web Thickness = 0.5 in
- Top Plate Width = 14 in
- Top Plate Thickness = 1 in
- Bottom Plate Width = 19 in
- Bottom Plate Thickness = 1.875 in – 0.25 in = 1.625 in
- Composite = Y for a composite superstructure
- Yield Strength of Girder = 50 ksi per Contract Drawings

<u>Girder Section from 52.125 ft to 100.75 ft</u>

- GFS Type = G for Girder
- Span Number = 1
- Range = 102.25 ft – 1.5 ft = 100.75 ft
- Section Type = P for Plate Girder
- Beam Depth = 44 in
- Web Thickness = 0.5 in
- Top Plate Width = 14 in
- Top Plate Thickness = 1 in
- Bottom Plate Width = 19 in
- Bottom Plate Thickness = 1.875 in
- Composite = Y for a composite superstructure
- Yield Strength of Girder = 50 ksi per Contract Drawings

<u>Girder Section from 100.75 ft to 102.25 ft</u>

- GFS Type = G for Girder
- Span Number = 1
- Range = 102.25 ft
- Section Type = P for Plate Girder
- Beam Depth = 44 in
- Web Thickness = [0.5 in x (44 in – 6 in) + 0.4375 in x 6 in] / 44 in = 0.4915 in average
- Top Plate Width = 14 in
- Top Plate Thickness = 1 in

- Bottom Plate Width = 19 in
- Bottom Plate Thickness = 1.875 in
- Composite = Y for a composite superstructure
- Yield Strength of Girder = 50 ksi per Contract Drawings

Steel Member Properties

- GFS Type = G for Girder
- Span Number = 1
- Range = 102.25 ft
- Section Type = P for Plate Girder
- Beam Depth = 44 in
- Web Thickness = 0.5 in
- Top Plate Width = 14 in
- Top Plate Thickness = 1 in
- Bottom Plate Width = 19 in
- Bottom Plate Thickness = 1.875 in
- Composite = Y for a composite superstructure
- Yield Strength of Girder = 50 ksi per Contract Drawings

Lateral Brace Points

Lateral brace points for a girder should be entered based on which flange is in compression. For a simple span bridge, the top flange is in compression for the entire span length while the bottom flange is in tension.

- Bracing or Stiffener Code = BG for brace points of a girder
- Span Number = 1
- Bracing Point or Stiffener Code = C for continuously braced top flange
- Number of Spaces = 1, Spacing = 102.25 ft
- Bracing or Stiffener Code = SG for stiffener spacings in a girder
- Span Number = 1
- Bracing Point or Stiffener Code = T for transversely stiffened girder web. Transverse stiffeners are equally spaced between intermediate diaphragms. Diaphragm spacing = 102.25 ft / 5 spaces = 20.45 ft.
- Number of Spaces = 1, Spacing = 1’-10” = 1.83 ft
- Number of Spaces = 4, Spacing = (20.45 ft – 1.83 ft) / 4 spaces = 4.66 ft
- Number of Spaces = 4, Spacing = 20.45 ft / 4 spaces = 5.11 ft
- Bracing Point or Stiffener Code = N for unstiffened girder web
- Number of Spaces = 1, Spacing = 20.45 ft
- Bracing Point or Stiffener Code = T for transversely stiffened girder web
- Number of Spaces = 4, Spacing = 20.45 ft / 4 spaces = 5.11 ft
- Number of Spaces = 4, Spacing = (20.45 ft – 1.83 ft) / 4 spaces = 4.66 ft
- Number of Spaces = 1, Spacing = 1’-10” = 1.83 ft

Concentrated Patch Loads

Applying intermediate and end diaphragm weights as patch loads typically isn't required unless a more accurate rating is desired. For this example, these loads are applied as patch loads. The addition of the end diaphragm weight should not change the ratings, however, these loads were included so a more accurate bearing reaction could be calculated by the program.

Intermediate Diaphragms

The intermediate diaphragms consist of a W24x68 beam, MC6x12 channel, connection plates, and bolts placed perpendicular to the girder. The weight of the intermediate diaphragms is applied as point loads along the span at the diaphragm locations.

W24x68: 4 x 0.068 kips/ft x 7.7917 ft = 2.12 kips
MC6x12: 4 x 0.012 kips/ft x 7.7917 ft = 0.37 kips
Connection Plates: 8 x 0.625 in x 7.5 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.47 kips
Total Intermediate Diaphragm Weight = 2.12 + 0.37 + 0.47 = 2.96 kips

Diaphragm Spacing = 102.25 ft / 5 spaces = 20.45 ft
Diaphragm Locations = 20.45 ft, 40.90 ft, 61.35 ft, 81.80 ft

End Diaphragms

The end diaphragms consist of a W27x84 beam, jacking stiffeners, jacking plates, connections plates, and bearing stiffeners.

Jacking Stiffeners: 4 x 0.75 in x 4.25 in x (26.69 in – 2 x 0.636 in) x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.092 kips
Jacking Plates: 4 x 1.25 in x 8.5 in x 8.5 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.102 kips
Connection Plates: 2 x 0.5 in x 10 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.125 kips
Bearing Stiffeners: 2 x 1 in x 9 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.225 kips

End Diaphragm Length at South Abutment = 45.80 ft / 4 = 11.45 ft (Abutment 1)
W27x84 Diaphragm: 0.084 kips/ft x 11.45 ft = 0.962 kips (Abutment 1)
End Diaphragm Weight at South Abutment = 0.092 + 0.102 + 0.125 + 0.225 + 0.962 = 1.51 kips

End Diaphragm Length at North Abutment = 48.24 ft / 4 = 12.06 ft (Abutment 2)
W27x84 Diaphragm: 0.084 kips/ft x 12.06 ft = 1.013 kips (Abutment 2)
End Diaphragm Weight at North Abutment = 0.092 + 0.102 + 0.125 + 0.225 + 1.013 = 1.56 kips

For information, the intermediate and end diaphragm weight per foot length of girder would be equal to (2.96 kips + 1.51 kips + 1.56 kips) / 102.25 ft = 0.059 kips/ft. If the patch loads were removed and the diaphragm weight was included in the bridge dead load 1, the rating factors would decrease by about 0.03 to 0.04. Since the rating is controlled by flexure and the end diaphragms do not contribute any flexural moment to the beam, removal of the end diaphragm weight results in an intermediate diaphragm weight of 2.96 kips / 102.25 ft = 0.029 kips/ft. For this loading, the rating factors are very similar but slightly less to those with the original patch loading.

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 02-4003-0010-1709 | | |
| Description: | | | |
| Bridge Type: | GGG | | Multigirder |
| SLC Level: | | | Super = 5 & Sub = 6; ADTT = 23 therefore SLC = 1.0 |
| Lanes: | | | N/A for CTB |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 3 | | Load Factor Ratings |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | Y | | |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | | | Leave blank and calculate DF |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | | | |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for GGG |
| Overhang or Spacing: | 7.79 | ft | |
| CL of Girder: | -- | ft | Leave blank for GGG |
| Roadway Width: | -- | ft | Leave blank for GGG |
| Distr Factor -Shear: | 0.858 | | See Hand Calcs |
| Distr Factor -Moment: | 0.708 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.417 | | See Hand Calcs |
| Slab Thickness: | 8.50 | in | Per plans |
| Haunch: | | in | |
| Bridge DL1: | 0.133 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.185 | kip/ft | See Hand Calcs |
| F'c: | 4.0 | ksi | Per plans |
| N: | 8 | | Class AAA per DM-4 8.2 |
| Symmetry: | | | |
| LL Location: | | | Only applies to truss |
| Number of Panels: | | | Only applies to truss |
| End Connections: | | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | | | Only applies to truss |
| Stringer DL: | | kip/ft | N/A for CTB |
| Floorbeam DL: | | kip/ft | N/A for CTB |
| Unit weight of Concrete: | | lb/cf | N/A for steel grid |
| Gusset Plate Analysis: | | | N/A for CTB |
| Patch Load Analysis: | | | N/A for CTB |
| Unsymmetrical Pier Support: | | | N/A for CTB |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 102.25 | ft | Per plans |

| Steel Member Properties | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| GFS | G | G | G | G | | | | |
| Span Number | 1 | 1 | 1 | 1 | | | | |
| Range | 50.12 | 52.13 | 100.75 | 102.25 | | | | |
| Section Type | P | P | P | P | | | | |
| WF Beam M of I | | | | | | | | |
| WF Beam A | | | | | | | | |
| Flange Thickness | | | | | | | | |
| Flange Width | | | | | | | | |
| Varies | | | | | | | | |
| WF Beam Depth | 44.00 | 44.00 | 44.00 | 44.00 | | | | |
| Web Thickness | 0.5 | 0.5 | 0.5 | 0.4915 | | | | |
| Top Plate Width | 14 | 14 | 14 | 14 | | | | |
| Top Plate Thickness | 1 | 1 | 1 | 1 | | | | |
| Bottom Plate Width | 19 | 19 | 19 | 19 | | | | |
| Bottom Plate Thickness | 1.875 | 1.625 | 1.875 | 1.875 | | | | |
| Composite | Y | Y | Y | Y | | | | |
| Yield Strength | 50 | 50 | 50 | 50 | | | | |
| Hybrid fy Top | | | | | | | | |
| Hybrid fy Bottom | | | | | | | | |

| Lateral Bracing | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Bracing or Stiff Code | BG | SG | | | | | | |
| Span Number | 1 | 1 | | | | | | |
| Code 1 | C | T | | | | | | |
| Num of Spaces | 1 | 1 | | | | | | |
| Spacing 1 | 102.25 | 1.83 | | | | | | |
| Code 2 | | T | | | | | | |
| Num of Spaces | | 4 | | | | | | |
| Spacing 2 | | 4.66 | | | | | | |
| Code 3 | | T | | | | | | |
| Num of Spaces | | 4 | | | | | | |
| Spacing 3 | | 5.11 | | | | | | |
| Code 4 | | N | | | | | | |
| Num of Spaces | | 1 | | | | | | |
| Spacing 4 | | 20.45 | | | | | | |
| Code 5 | | T | | | | | | |
| Num of Spaces | | 4 | | | | | | |
| Spacing 5 | | 5.11 | | | | | | |
| Code 6 | | T | | | | | | |
| Num of Spaces | | 4 | | | | | | |
| Spacing 6 | | 4.66 | | | | | | |
| Code 7 | | T | | | | | | |
| Num of Spaces 7 | | 1 | | | | | | |
| Spacing 7 | | 1.83 | | | | | | |
| Code 8 | | | | | | | | |
| Num of Spaces 8 | | | | | | | | |
| Spacing 8 | | | | | | | | |

### 3.9.8.2.2 BAR7 Output

```
*****************************************************************************
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                    BRIDGE ANALYSIS AND RATING (BAR7)                 330427

PROGRAM P4353000                                               05/01/2024 12:47
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: ... l Girder BAR7 Analysis - With Section Loss.dat




                            PROJECT IDENTIFICATION


STRUCTURE ID - 02400300101709 - INT BEAM NO. 8 WITH LOSS

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 GGG                3   0.00  0.0  0.0        Y                  0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     0.000        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    7.79       0.00        0.00   0.858  0.708  0.417

   SLAB               DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   8.50      0.00   0.133   0.185   4.000    8.

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00          Y


                         SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH    102.25


            CONCRETE MEMBER PROPERTIES

                                             FY
TYPE  DEPTH    B     D    AS     D'   A'S   REINF
 O     0.00   0.00  0.00  0.00  0.00  0.00   60.


     ALLOWABLE FS    ST                          INTEGRAL
       IR    OR      DET   AV   SPECS  ALPHA  WEARING SURFACE
       0.0   0.0          0.00    0      0.         0.0


                            STEEL MEMBER PROPERTIES
```

```
  S         T  WF BM    WF BM   FLANGE            WF BM
G P         Y  M OF I    AREA     OR           V  OR WEB
F A         P  OR VRT   OR HRZ  ANGLE  FLANGE  A   PLATE    WEB
S N  RANGE E   LEG      LEG    THICK   WIDTH  R   DEPTH   THICK  SHAPE
G 1  50.12 P     0.00     0.00  0.0000  0.000       44.00  0.5000
               TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             14.00 1.0000 19.00 1.8750    Y  50.0    0.0    0.0   0.000  0.000

     RANGE                          THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1  52.13 P     0.00     0.00  0.0000  0.000       44.00  0.5000
               TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             14.00 1.0000 19.00 1.6250    Y  50.0    0.0    0.0   0.000  0.000

     RANGE                          THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1 100.75 P     0.00     0.00  0.0000  0.000       44.00  0.5000
               TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             14.00 1.0000 19.00 1.8750    Y  50.0    0.0    0.0   0.000  0.000

     RANGE                          THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1 102.25 P     0.00     0.00  0.0000  0.000       44.00  0.4915
               TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             14.00 1.0000 19.00 1.8750    Y  50.0    0.0    0.0   0.000  0.000


                LATERAL BRACE POINTS AND STIFFENER SPACINGS

             C               C                C               C
B OR S       O  NO.          O  NO.           O  NO.          O  NO.
G OR F       D  OF           D  OF            D  OF           D  OF
 CODE  SPAN  E SPCS SPACING  E SPCS SPACING  E SPCS SPACING  E SPCS SPACING
  BG     1   C   1  102.25      0    0.00       0    0.00       0    0.00
                 0    0.00      0    0.00       0    0.00       0    0.00
  SG     1   T   1    1.83   T  4    4.66    T  4    5.11    N  1   20.45
             T   4    5.11   T  4    4.66    T  1    1.83       0    0.00


                                         CONCENTRATED PATCH LOADS

MEM LOAD SPAN                | LOAD SPAN                 | LOAD SPAN                 |
TYP TYPE  NO    DIST     MAG | TYPE  NO    DIST      MAG | TYPE  NO    DIST      MAG |
 G  DC1S   1   20.45    2.96 | DC1S   1   40.90     2.96 | DC1S   1   61.35     2.96 |

MEM LOAD SPAN                |
TYP TYPE  NO    DIST     MAG |
 G  DC1S   1   81.80    2.96 |

MEM LOAD SPAN                | LOAD SPAN                 | LOAD SPAN                 |
TYP TYPE  NO    DIST     MAG | TYPE  NO    DIST      MAG | TYPE  NO    DIST      MAG |
 G  DC1S   1    0.00    1.51 | DC1S   1  102.25     1.56 |        0    0.00      0.00 |


                                          DEFAULT VALUES

                               UNIT                                INTEGRAL
 SLC     GAGE    PASSING    WEIGHT    FY     ALLOWABLE FS   WEARING   SKEW CORR
LEVEL  DISTANCE  DISTANCE    DECK    REINF    IR       OR    SURFACE    FACTOR
  I       6.0       4.0      150.0    ---    24.0     36.0     0.5       1.000


                ++++++++++++++++++++++++++++++++++++++++++++
                +                                          +
```

```
                +       G I R D E R   A N A L Y S I S       +
                +                                            +
                ++++++++++++++++++++++++++++++++++++++++++++++
```

DEAD LOADS ACTING ON GIRDER

| INPUT DL1 | GIRDER WEIGHT | SLAB WEIGHT | FL BEAM WEIGHT | STRINGER WEIGHT | FL BEAM DL1 | STRINGER DL1 | TOTAL DL1 | TOTAL DL2 |
|---|---|---|---|---|---|---|---|---|
| 0.133 | 0.243 | 0.828 | 0.000 | 0.000 | 0.000 | 0.000 | 1.204 | 0.185 |

NOTE: IF A SECTION DOES NOT MEET FLANGE OR WEB BUCKLING CRITERIA OF CURRENT AASHTO SPECIFICATIONS FOR LOAD FACTOR METHOD, THE RATING FACTORS ARE REPRINTED AS 888.88. THIS INDICATES THAT THERE IS A POTENTIAL FATIGUE PROBLEM.

GIRDER SECTION PROPERTIES (NON-COMPOSITE)

SPAN 1
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT |
|---|---|---|---|---|---|---|
| 0.00 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 10.22 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 20.45 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 30.67 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 40.90 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 50.12L | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 50.12R | 46.62 | 66.88 | 24445.80 | 17.80 | 848.17 | 1373.12 |
| 51.12 | 46.62 | 66.88 | 24445.80 | 17.80 | 848.17 | 1373.12 |
| 52.13L | 46.62 | 66.88 | 24445.80 | 17.80 | 848.17 | 1373.12 |
| 52.13R | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 61.35 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 71.58 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 81.80 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 92.02 | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 100.75L | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 |
| 100.75R | 46.88 | 71.25 | 25792.50 | 16.83 | 858.39 | 1532.77 |
| 102.25 | 46.88 | 71.25 | 25792.50 | 16.83 | 858.39 | 1532.77 |

GIRDER SECTION PROPERTIES (COMPOSITE, N = 8)

SPAN 1 - EFFECTIVE SLAB WIDTH: 93.48 THICKNESS: 8.00
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S CONC |
|---|---|---|---|---|---|---|---|
| 0.00 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 10.22 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 20.45 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 30.67 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 40.90 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 50.12L | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 50.12R | 54.62 | 160.36 | 66942.11 | 36.94 | 6909.70 | 1812.34 | 3784.58 |
| 51.12 | 54.62 | 160.36 | 66942.11 | 36.94 | 6909.70 | 1812.34 | 3784.58 |
| 52.13L | 54.62 | 160.36 | 66942.11 | 36.94 | 6909.70 | 1812.34 | 3784.58 |
| 52.13R | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 61.35 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 71.58 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 81.80 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 92.02 | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 100.75L | 54.88 | 165.11 | 73278.94 | 36.12 | 6813.86 | 2028.73 | 3907.30 |
| 100.75R | 54.88 | 164.73 | 73162.39 | 36.15 | 6820.66 | 2023.94 | 3906.87 |
| 102.25 | 54.88 | 164.73 | 73162.39 | 36.15 | 6820.66 | 2023.94 | 3906.87 |

GIRDER SECTION PROPERTIES (COMPOSITE, N = 24)

SPAN 1 - EFFECTIVE SLAB WIDTH: 93.48 THICKNESS: 8.00
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S CONC |
|---|---|---|---|---|---|---|---|
| 0.00 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 10.22 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 20.45 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 30.67 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 40.90 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 50.12L | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 50.12R | 54.62 | 98.03 | 47510.44 | 28.24 | 2583.55 | 1682.65 | 1800.35 |
| 51.12 | 54.62 | 98.03 | 47510.44 | 28.24 | 2583.55 | 1682.65 | 1800.35 |
| 52.13L | 54.62 | 98.03 | 47510.44 | 28.24 | 2583.55 | 1682.65 | 1800.35 |
| 52.13R | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 61.35 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 71.58 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 81.80 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 92.02 | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 100.75L | 54.88 | 102.78 | 51154.40 | 27.17 | 2596.65 | 1882.42 | 1846.72 |
| 100.75R | 54.88 | 102.41 | 51089.98 | 27.19 | 2594.96 | 1879.22 | 1845.19 |
| 102.25 | 54.88 | 102.41 | 51089.98 | 27.19 | 2594.96 | 1879.22 | 1845.19 |

GIRDER SECTION PROPERTIES (COMPOSITE, NEGATIVE MOMENT)

SPAN 1 - EFFECTIVE SLAB WIDTH: 93.48 THICKNESS: 8.00
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S REINF |
|---|---|---|---|---|---|---|---|
| 0.00 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 10.22 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 20.45 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 30.67 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 40.90 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 50.12L | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 50.12R | 54.62 | 66.88 | 24445.80 | 17.80 | 848.17 | 1373.12 | N/A |
| 51.12 | 54.62 | 66.88 | 24445.80 | 17.80 | 848.17 | 1373.12 | N/A |
| 52.13L | 54.62 | 66.88 | 24445.80 | 17.80 | 848.17 | 1373.12 | N/A |
| 52.13R | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 61.35 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 71.58 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 81.80 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 92.02 | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 100.75L | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |
| 100.75R | 54.88 | 71.25 | 25792.50 | 16.83 | 858.39 | 1532.77 | N/A |
| 102.25 | 54.88 | 71.25 | 25792.50 | 16.83 | 858.39 | 1532.77 | N/A |

```
**************
* DEAD LOADS *
**************


    *******
    * DL1 *
    *******


===============================================================
DL1 DEAD LOAD INCLUDING USER-ENTERED DL1 AND BAR7-COMPUTED DL1.
===============================================================
```

```
          DISTANCE            INTENSITY
       BEGIN     END       BEGIN      END


       0.000   102.250     1.204     1.204


 ========================================================================
 TOTAL DL1 DEAD LOAD EFFECT INCLUDING USER-ENTERED DL1, BAR7-COMPUTED DL1,
 AND DL1 PATCH LOADS
 ========================================================================


SUPPORT NO    REACTION

     1          68.988
     2          69.038

 SPAN SECTION
  NO     NO   DISTANCE      DM1        DS1      DEF1

   1      1     0.00       0.000    67.478   0.000
   1      2    10.22     627.023    55.167   1.389
   1      3    20.45    1128.158    42.855   2.632
   1      4    30.67    1473.141    27.583   3.610
   1      5    40.90    1692.238    15.272   4.238
   1      6    50.12    1754.573     1.210   4.454
   1      7    51.12    1755.181    -0.000   4.457
   1      8    52.13    1754.573    -1.210   4.454
   1      9    61.35    1692.238   -15.272   4.238
   1     10    71.58    1473.141   -27.583   3.610
   1     11    81.80    1128.158   -42.855   2.632
   1     12    92.02     627.023   -55.167   1.389
   1     13   100.75      99.863   -65.672   0.213
   1     14   102.25       0.000   -67.478   0.000

 SUPPORT NO    END SHEAR

      1         -67.478

                                  *******
                                  * DL2 *
                                  *******


 ========================================
 DL2 DEAD LOAD INCLUDING USER-ENTERED DL2
 ========================================



          DISTANCE            INTENSITY
       BEGIN     END       BEGIN      END


       0.000   102.250     0.185     0.185

 ========================================================================
 TOTAL DL2 DEAD LOAD EFFECT INCLUDING USER-ENTERED DL2 AND DL2 PATCH LOADS
 ========================================================================
```

| SUPPORT NO | REACTION |
|---|---|
| 1 | 9.458 |
| 2 | 9.458 |

| SPAN NO | SECTION NO | DISTANCE | DM2 | DS2 | DEF2 |
|---|---|---|---|---|---|
| 1 | 1 | 0.00 | 0.000 | 9.458 | 0.000 |
| 1 | 2 | 10.22 | 87.038 | 7.566 | 0.097 |
| 1 | 3 | 20.45 | 154.735 | 5.675 | 0.184 |
| 1 | 4 | 30.67 | 203.090 | 3.783 | 0.252 |
| 1 | 5 | 40.90 | 232.102 | 1.892 | 0.296 |
| 1 | 6 | 50.12 | 241.680 | 0.186 | 0.312 |
| 1 | 7 | 51.12 | 241.773 | -0.000 | 0.312 |
| 1 | 8 | 52.13 | 241.680 | -0.186 | 0.312 |
| 1 | 9 | 61.35 | 232.102 | -1.892 | 0.296 |
| 1 | 10 | 71.58 | 203.090 | -3.783 | 0.252 |
| 1 | 11 | 81.80 | 154.735 | -5.675 | 0.184 |
| 1 | 12 | 92.02 | 87.038 | -7.566 | 0.097 |
| 1 | 13 | 100.75 | 13.979 | -9.181 | 0.015 |
| 1 | 14 | 102.25 | 0.000 | -9.458 | 0.000 |

| SUPPORT NO | END SHEAR |
|---|---|
| 1 | -9.458 |

```
********************************
*  GIRDER  -  LIVE LOAD  H20   *
********************************
```

MAXIMUM REACTIONS

| | | | | | REACTIONS | | MOMENTS | |
|---|---|---|---|---|---|---|---|---|
| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | +I.F. | -I.F. | +I.F. | -I.F. |
| 1 | 69.0 | 9.5 | 55.5 L | 0.0 | 1.22 | | | |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 67.5 | 9.5 | 55.5L | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 10.22 | 627.0 | 87.0 | 403.2L | 0.0 | | 55.2 | 7.6 | 43.5L | -2.8 | 1.23 |
| | SIMULT | SHEAR | 36.9 | 0.0 | | SIMULT | MOM | 440.8 | 254.4 | |
| 20.45 | 1128.2 | 154.7 | 716.7L | 0.0 | | 42.9 | 5.7 | 36.7L | -6.1 | 1.24 |
| | SIMULT | SHEAR | 29.9 | 0.0 | | SIMULT | MOM | 737.3 | 487.9 | |
| 30.67 | 1473.1 | 203.1 | 940.7L | 0.0 | | 27.6 | 3.8 | 30.4L | -9.7 | 1.25 |
| | SIMULT | SHEAR | 22.8 | 0.0 | | SIMULT | MOM | 907.0 | 674.2 | |
| 40.90 | 1692.2 | 232.1 | 1075.1L | 0.0 | | 15.3 | 1.9 | 24.6L | -14.0L | 1.27 |
| | SIMULT | SHEAR | 15.6 | 0.0 | | SIMULT | MOM | 967.3 | 828.6 | |
| 50.12 | 1754.6 | 241.7 | 1119.5L | 0.0 | | 1.2 | 0.2 | 19.8L | -18.7L | 1.28 |

```
            SIMULT SHEAR    8.9       0.0       SIMULT MOM   942.1   927.9
   51.12  1755.2   241.8 1119.9L     0.0       -0.0   -0.0   19.3L  -19.3L 1.28
            SIMULT SHEAR   -8.2       0.0       SIMULT MOM   935.3   935.3
   52.13  1754.6   241.7 1119.5L     0.0       -1.2   -0.2   18.7L  -19.8L 1.28
            SIMULT SHEAR   -8.9       0.0       SIMULT MOM   927.9   942.1
   61.35  1692.2   232.1 1075.1L     0.0      -15.3   -1.9   14.0L  -24.6L 1.27
            SIMULT SHEAR  -15.6       0.0       SIMULT MOM   828.6   967.3
   71.58  1473.1   203.1  940.7L     0.0      -27.6   -3.8    9.7   -30.4L 1.25
            SIMULT SHEAR  -22.8       0.0       SIMULT MOM   674.2   907.0
   81.80  1128.2   154.7  716.7L     0.0      -42.9   -5.7    6.1   -36.7L 1.24
            SIMULT SHEAR  -29.9       0.0       SIMULT MOM   487.9   737.3
   92.02   627.0    87.0  403.2L     0.0      -55.2   -7.6    2.8   -43.5L 1.23
            SIMULT SHEAR  -36.9       0.0       SIMULT MOM   254.4   440.8
  100.75    99.9    14.0   64.8L     0.0      -65.7   -9.2    0.4   -49.6L 1.22
            SIMULT SHEAR  -42.8       0.0       SIMULT MOM    40.9    74.4
  102.25     0.0     0.0    0.0      0.0      -67.5   -9.5    0.0   -55.5L 1.22
            SIMULT SHEAR    0.0       0.0       SIMULT MOM     0.0     0.0
```

```
NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR
      UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER
      SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.


                     STRENGTHS AND LOAD FACTOR RATINGS

SPAN  1
=======
        NON-COMP OVERLOAD          NON-COMPACT     COMPACT      COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS   MOMENT   RATING FACTORS
   X    STRENGTH STRENGTH STRENGTH   IR       OR    STRENGTH    IR       OR
  0.00   8453.0 B  8030.4   638.0   4.48 V   7.46 V   9314.6   4.48 V   7.46 V
 10.22   8453.0 B  8030.4   599.0   5.49 V   9.16 V   9314.6   5.49 V   9.16 V
 20.45   8453.0 B  8030.4   599.0   3.88 I   6.46 I   9314.6   3.88 I   6.46 I
 30.67   8453.0 B  8030.4   587.4   2.77 B   4.61 B   9314.6   3.50     5.83
 40.90   8453.0 B  8030.4   587.4   2.24 B   3.73 B   9314.6   2.92     4.87
 50.12   7551.4 B  7173.8   370.7   1.73 B   2.89 B   8443.7   2.41     4.02
 51.12   7551.4 B  7173.8   370.7   1.73 B   2.89 B   8443.7   2.41     4.02
 52.13   7551.4 B  7173.8   370.7   1.73 B   2.89 B   8443.7   2.41     4.02
 61.35   8453.0 B  8030.4   370.7   2.24 B   3.73 B   9314.6   2.92     4.87
 71.58   8453.0 B  8030.4   587.4   2.77 B   4.61 B   9314.6   3.50     5.83
 81.80   8453.0 B  8030.4   587.4   3.85 I   6.41 I   9314.6   3.85 I   6.41 I
 92.02   8453.0 B  8030.4   599.0   5.49 V   9.16 V   9314.6   5.49 V   9.16 V
100.75   8433.1 B  8011.4   627.2   4.93 V   8.21 V   9284.8   4.93 V   8.21 V
102.25   8433.1 B  8011.4   627.2   4.39 V   7.31 V   9284.8   4.39 V   7.31 V


                      *******************************
                      *  GIRDER  -  LIVE LOAD  HS20  *
                      *******************************

                                 MAXIMUM REACTIONS

                                                   REACTIONS     MOMENTS
    SUPPORT    DL1       DL2    +(LL+I)     -(LL+I)  +I.F. -I.F.  +I.F. -I.F.
       1      69.0       9.5      62.4         0.0    1.22


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
      ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
      SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.
```

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 67.5 | 9.5 | 62.4 | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 10.22 | 627.0 | 87.0 | 514.3 | 0.0 | | 55.2 | 7.6 | 50.7 | -2.8 | 1.23 |
| | SIMULT | SHEAR | 50.7 | 0.0 | | SIMULT | MOM | 514.3 | 254.4 | |
| 20.45 | 1128.2 | 154.7 | 901.4 | 0.0 | | 42.9 | 5.7 | 44.9 | -7.4 | 1.24 |
| | SIMULT | SHEAR | 44.9 | 0.0 | | SIMULT | MOM | 901.4 | 594.8 | |
| 30.67 | 1473.1 | 203.1 | 1161.3 | 0.0 | | 27.6 | 3.8 | 38.9 | -13.3 | 1.25 |
| | SIMULT | SHEAR | 38.9 | 0.0 | | SIMULT | MOM | 1161.3 | 929.1 | |
| 40.90 | 1692.2 | 232.1 | 1313.4 | 0.0 | | 15.3 | 1.9 | 32.9 | -20.0 | 1.27 |
| | SIMULT | SHEAR | 28.7 | 0.0 | | SIMULT | MOM | 1294.0 | 1177.9 | |
| 50.12 | 1754.6 | 241.7 | 1350.2 | 0.0 | | 1.2 | 0.2 | 27.4 | -26.1 | 1.28 |
| | SIMULT | SHEAR | 23.1 | 0.0 | | SIMULT | MOM | 1304.6 | 1293.2 | |
| 51.12 | 1755.2 | 241.8 | 1347.9 | 0.0 | | -0.0 | -0.0 | 26.7 | -26.7 | 1.28 |
| | SIMULT | SHEAR | -22.5 | 0.0 | | SIMULT | MOM | 1299.6 | 1299.6 | |
| 52.13 | 1754.6 | 241.7 | 1350.2 | 0.0 | | -1.2 | -0.2 | 26.1 | -27.4 | 1.28 |
| | SIMULT | SHEAR | -23.1 | 0.0 | | SIMULT | MOM | 1293.2 | 1304.6 | |
| 61.35 | 1692.2 | 232.1 | 1313.4 | 0.0 | | -15.3 | -1.9 | 20.0 | -32.9 | 1.27 |
| | SIMULT | SHEAR | -28.7 | 0.0 | | SIMULT | MOM | 1177.9 | 1294.0 | |
| 71.58 | 1473.1 | 203.1 | 1161.3 | 0.0 | | -27.6 | -3.8 | 13.3 | -38.9 | 1.25 |
| | SIMULT | SHEAR | -38.9 | 0.0 | | SIMULT | MOM | 929.1 | 1161.3 | |
| 81.80 | 1128.2 | 154.7 | 901.4 | 0.0 | | -42.9 | -5.7 | 7.4 | -44.9 | 1.24 |
| | SIMULT | SHEAR | -44.9 | 0.0 | | SIMULT | MOM | 594.8 | 901.4 | |
| 92.02 | 627.0 | 87.0 | 514.3 | 0.0 | | -55.2 | -7.6 | 2.8 | -50.7 | 1.23 |
| | SIMULT | SHEAR | -50.7 | 0.0 | | SIMULT | MOM | 254.4 | 514.3 | |
| 100.75 | 99.9 | 14.0 | 83.4 | 0.0 | | -65.7 | -9.2 | 0.4 | -55.7 | 1.22 |
| | SIMULT | SHEAR | -55.7 | 0.0 | | SIMULT | MOM | 40.9 | 83.4 | |
| 102.25 | 0.0 | 0.0 | 0.0 | 0.0 | | -67.5 | -9.5 | 0.0 | -62.4 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 8453.0 B | 8030.4 | 638.0 | 3.98 V | 6.64 V | 9314.6 | 3.98 V | 6.64 V |
| 10.22 | 8453.0 B | 8030.4 | 599.0 | 4.71 V | 7.85 V | 9314.6 | 4.71 V | 7.85 V |
| 20.45 | 8453.0 B | 8030.4 | 599.0 | 3.17 I | 5.28 I | 9314.6 | 3.17 I | 5.28 I |
| 30.67 | 8453.0 B | 8030.4 | 587.4 | 2.24 B | 3.73 B | 9314.6 | 2.84 | 4.73 |
| 40.90 | 8453.0 B | 8030.4 | 587.4 | 1.83 B | 3.06 B | 9314.6 | 2.39 | 3.99 |
| 50.12 | 7551.4 B | 7173.8 | 370.7 | 1.44 B | 2.39 B | 8443.7 | 2.00 | 3.33 |
| 51.12 | 7551.4 B | 7173.8 | 370.7 | 1.44 B | 2.40 B | 8443.7 | 2.00 | 3.34 |
| 52.13 | 7551.4 B | 7173.8 | 370.7 | 1.44 B | 2.39 B | 8443.7 | 2.00 | 3.33 |
| 61.35 | 8453.0 B | 8030.4 | 370.7 | 1.83 B | 3.06 B | 9314.6 | 2.39 | 3.99 |
| 71.58 | 8453.0 B | 8030.4 | 587.4 | 2.24 B | 3.73 B | 9314.6 | 2.84 | 4.73 |
| 81.80 | 8453.0 B | 8030.4 | 587.4 | 3.15 I | 5.25 I | 9314.6 | 3.15 I | 5.25 I |
| 92.02 | 8453.0 B | 8030.4 | 599.0 | 4.71 V | 7.85 V | 9314.6 | 4.71 V | 7.85 V |
| 100.75 | 8433.1 B | 8011.4 | 627.2 | 4.39 V | 7.32 V | 9284.8 | 4.39 V | 7.32 V |
| 102.25 | 8433.1 B | 8011.4 | 627.2 | 3.90 V | 6.50 V | 9284.8 | 3.90 V | 6.50 V |

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                    ********************************
                    *  GIRDER  -  LIVE LOAD TK527  *
                    ********************************

                             MAXIMUM REACTIONS

                                                       REACTIONS     MOMENTS
    SUPPORT     DL1       DL2    +(LL+I)     -(LL+I)  +I.F. -I.F.  +I.F. -I.F.
       1       69.0       9.5     68.0         0.0    1.22


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
      ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
      SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.


                      UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======
          DL1     DL2   +(LL+I)  -(LL+I)  I   DL1    DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT  M   SHEAR  SHEAR  SHEAR   SHEAR  I.F.
   0.00     0.0     0.0    0.0      0.0        67.5    9.5   68.0     0.0  1.22
          SIMULT SHEAR     0.0      0.0        SIMULT MOM     0.0     0.0
  10.22   627.0    87.0  583.8      0.0        55.2    7.6   57.6    -3.0  1.23
          SIMULT SHEAR    57.6      0.0        SIMULT MOM   583.8   277.7
  20.45  1128.2   154.7 1022.1      0.0        42.9    5.7   50.9    -8.4  1.24
          SIMULT SHEAR    50.9      0.0        SIMULT MOM  1022.1   672.2
  30.67  1473.1   203.1 1329.0      0.0        27.6    3.8   44.1   -15.0  1.25
          SIMULT SHEAR    28.6      0.0        SIMULT MOM  1314.8  1046.6
  40.90  1692.2   232.1 1504.6      0.0        15.3    1.9   37.2   -22.4  1.27
          SIMULT SHEAR    21.6      0.0        SIMULT MOM  1461.9  1319.6
  50.12  1754.6   241.7 1538.2      0.0         1.2    0.2   30.8   -29.4  1.28
          SIMULT SHEAR    15.0      0.0        SIMULT MOM  1469.8  1455.8
  51.12  1755.2   241.8 1534.7      0.0        -0.0   -0.0   30.1   -30.1  1.28
          SIMULT SHEAR    -9.0      0.0        SIMULT MOM  1463.5  1463.5
  52.13  1754.6   241.7 1538.2      0.0        -1.2   -0.2   29.4   -30.8  1.28
          SIMULT SHEAR   -15.0      0.0        SIMULT MOM  1455.8  1469.8
  61.35  1692.2   232.1 1504.6      0.0       -15.3   -1.9   22.4   -37.2  1.27
          SIMULT SHEAR   -21.6      0.0        SIMULT MOM  1319.6  1461.9
  71.58  1473.1   203.1 1329.0      0.0       -27.6   -3.8   15.0   -44.1  1.25
          SIMULT SHEAR   -28.6      0.0        SIMULT MOM  1046.6  1314.8
  81.80  1128.2   154.7 1022.1      0.0       -42.9   -5.7    8.4   -50.9  1.24
          SIMULT SHEAR   -50.9      0.0        SIMULT MOM   672.2  1022.1
  92.02   627.0    87.0  583.8      0.0       -55.2   -7.6    3.0   -57.6  1.23
          SIMULT SHEAR   -57.6      0.0        SIMULT MOM   277.7   583.8
 100.75    99.9    14.0   94.8      0.0       -65.7   -9.2    0.3   -63.2  1.22
          SIMULT SHEAR   -63.2      0.0        SIMULT MOM    26.3    94.8
 102.25     0.0     0.0    0.0      0.0       -67.5   -9.5    0.0   -68.0  1.22
          SIMULT SHEAR     0.0      0.0        SIMULT MOM     0.0     0.0


NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR
      UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER
      SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.


                     STRENGTHS AND LOAD FACTOR RATINGS

SPAN  1
=======
        NON-COMP OVERLOAD            NON-COMPACT     COMPACT      COMPACT
```

| X | MOMENT STRENGTH | MOMENT STRENGTH | SHEAR STRENGTH | RATING FACTORS IR | RATING FACTORS OR | MOMENT STRENGTH | RATING FACTORS IR | RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 8453.0 B | 8030.4 | 638.0 | 3.65 V | 6.09 V | 9314.6 | 3.65 V | 6.09 V |
| 10.22 | 8453.0 B | 8030.4 | 599.0 | 4.15 V | 6.91 V | 9314.6 | 4.15 V | 6.91 V |
| 20.45 | 8453.0 B | 8030.4 | 599.0 | 2.80 I | 4.66 I | 9314.6 | 2.80 I | 4.66 I |
| 30.67 | 8453.0 B | 8030.4 | 587.4 | 1.96 B | 3.26 B | 9314.6 | 2.48 | 4.13 |
| 40.90 | 8453.0 B | 8030.4 | 587.4 | 1.60 B | 2.67 B | 9314.6 | 2.09 | 3.48 |
| 50.12 | 7551.4 B | 7173.8 | 370.7 | 1.26 B | 2.10 B | 8443.7 | 1.75 | 2.92 |
| 51.12 | 7551.4 B | 7173.8 | 370.7 | 1.26 B | 2.11 B | 8443.7 | 1.76 | 2.93 |
| 52.13 | 7551.4 B | 7173.8 | 370.7 | 1.26 B | 2.10 B | 8443.7 | 1.75 | 2.92 |
| 61.35 | 8453.0 B | 8030.4 | 370.7 | 1.60 B | 2.67 B | 9314.6 | 2.09 | 3.48 |
| 71.58 | 8453.0 B | 8030.4 | 587.4 | 1.96 B | 3.26 B | 9314.6 | 2.48 | 4.13 |
| 81.80 | 8453.0 B | 8030.4 | 587.4 | 2.78 I | 4.63 I | 9314.6 | 2.78 I | 4.63 I |
| 92.02 | 8453.0 B | 8030.4 | 599.0 | 4.15 V | 6.91 V | 9314.6 | 4.15 V | 6.91 V |
| 100.75 | 8433.1 B | 8011.4 | 627.2 | 3.87 V | 6.44 V | 9284.8 | 3.87 V | 6.44 V |
| 102.25 | 8433.1 B | 8011.4 | 627.2 | 3.58 V | 5.96 V | 9284.8 | 3.58 V | 5.96 V |

```
********************************
*  GIRDER  -  LIVE LOAD  ML80  *
********************************
```

MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 69.0 | 9.5 | 64.8 | 0.0 | 1.22 | | | |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 67.5 | 9.5 | 64.8 | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 10.22 | 627.0 | 87.0 | 557.4 | 0.0 | | 55.2 | 7.6 | 55.0 | -3.3 | 1.23 |
| | SIMULT | SHEAR | 55.0 | 0.0 | | SIMULT | MOM | 557.4 | 299.1 | |
| 20.45 | 1128.2 | 154.7 | 981.4 | 0.0 | | 42.9 | 5.7 | 48.8 | -9.0 | 1.24 |
| | SIMULT | SHEAR | 48.8 | 0.0 | | SIMULT | MOM | 981.4 | 725.7 | |
| 30.67 | 1473.1 | 203.1 | 1279.2 | 0.0 | | 27.6 | 3.8 | 42.6 | -15.8 | 1.25 |
| | SIMULT | SHEAR | 27.0 | 0.0 | | SIMULT | MOM | 1272.1 | 1101.6 | |
| 40.90 | 1692.2 | 232.1 | 1462.6 | 0.0 | | 15.3 | 1.9 | 36.3 | -22.8 | 1.27 |
| | SIMULT | SHEAR | 20.5 | 0.0 | | SIMULT | MOM | 1429.5 | 1344.2 | |
| 50.12 | 1754.6 | 241.7 | 1513.7 | 0.0 | | 1.2 | 0.2 | 30.6 | -29.2 | 1.28 |
| | SIMULT | SHEAR | 14.5 | 0.0 | | SIMULT | MOM | 1457.1 | 1448.7 | |
| 51.12 | 1755.2 | 241.8 | 1512.7 | 0.0 | | -0.0 | -0.0 | 29.9 | -29.9 | 1.28 |
| | SIMULT | SHEAR | 13.9 | 0.0 | | SIMULT | MOM | 1453.5 | 1453.5 | |
| 52.13 | 1754.6 | 241.7 | 1513.7 | 0.0 | | -1.2 | -0.2 | 29.2 | -30.6 | 1.28 |
| | SIMULT | SHEAR | -14.5 | 0.0 | | SIMULT | MOM | 1448.7 | 1457.1 | |
| 61.35 | 1692.2 | 232.1 | 1462.6 | 0.0 | | -15.3 | -1.9 | 22.8 | -36.3 | 1.27 |
| | SIMULT | SHEAR | -20.5 | 0.0 | | SIMULT | MOM | 1344.2 | 1429.5 | |
| 71.58 | 1473.1 | 203.1 | 1279.2 | 0.0 | | -27.6 | -3.8 | 15.8 | -42.6 | 1.25 |
| | SIMULT | SHEAR | -27.0 | 0.0 | | SIMULT | MOM | 1101.6 | 1272.1 | |
| 81.80 | 1128.2 | 154.7 | 981.4 | 0.0 | | -42.9 | -5.7 | 9.0 | -48.8 | 1.24 |
| | SIMULT | SHEAR | -48.8 | 0.0 | | SIMULT | MOM | 725.7 | 981.4 | |
| 92.02 | 627.0 | 87.0 | 557.4 | 0.0 | | -55.2 | -7.6 | 3.3 | -55.0 | 1.23 |

```
          SIMULT SHEAR  -55.0      0.0      SIMULT MOM   299.1   557.4
  100.75    99.9    14.0   90.1      0.0     -65.7   -9.2    0.3   -60.1  1.22
          SIMULT SHEAR  -60.1      0.0      SIMULT MOM    26.3    90.1
  102.25     0.0     0.0    0.0      0.0     -67.5   -9.5    0.0   -64.8  1.22
          SIMULT SHEAR    0.0      0.0      SIMULT MOM     0.0     0.0
```

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 8453.0 B | 8030.4 | 638.0 | 3.83 V | 6.39 V | 9314.6 | 3.83 V | 6.39 V |
| 10.22 | 8453.0 B | 8030.4 | 599.0 | 4.34 V | 7.24 V | 9314.6 | 4.34 V | 7.24 V |
| 20.45 | 8453.0 B | 8030.4 | 599.0 | 2.91 I | 4.85 I | 9314.6 | 2.91 I | 4.85 I |
| 30.67 | 8453.0 B | 8030.4 | 587.4 | 2.03 B | 3.39 B | 9314.6 | 2.57 | 4.29 |
| 40.90 | 8453.0 B | 8030.4 | 587.4 | 1.65 B | 2.74 B | 9314.6 | 2.15 | 3.58 |
| 50.12 | 7551.4 B | 7173.8 | 370.7 | 1.28 B | 2.14 B | 8443.7 | 1.78 | 2.97 |
| 51.12 | 7551.4 B | 7173.8 | 370.7 | 1.28 B | 2.14 B | 8443.7 | 1.78 | 2.97 |
| 52.13 | 7551.4 B | 7173.8 | 370.7 | 1.28 B | 2.14 B | 8443.7 | 1.78 | 2.97 |
| 61.35 | 8453.0 B | 8030.4 | 370.7 | 1.65 B | 2.74 B | 9314.6 | 2.15 | 3.58 |
| 71.58 | 8453.0 B | 8030.4 | 587.4 | 2.03 B | 3.39 B | 9314.6 | 2.57 | 4.29 |
| 81.80 | 8453.0 B | 8030.4 | 587.4 | 2.89 I | 4.82 I | 9314.6 | 2.89 I | 4.82 I |
| 92.02 | 8453.0 B | 8030.4 | 599.0 | 4.34 V | 7.24 V | 9314.6 | 4.34 V | 7.24 V |
| 100.75 | 8433.1 B | 8011.4 | 627.2 | 4.07 V | 6.78 V | 9284.8 | 4.07 V | 6.78 V |
| 102.25 | 8433.1 B | 8011.4 | 627.2 | 3.75 V | 6.26 V | 9284.8 | 3.75 V | 6.26 V |

```
+++++++++++++++++++++++++++++++++++++++++++++
+                                           +
+        R A T I N G   S U M M A R Y        +
+                                           +
+++++++++++++++++++++++++++++++++++++++++++++
```

MEMBER: GIRDER

LOAD FACTOR RATING

| LOAD | | FACTOR | TONS | X | SPAN |
|---|---|---|---|---|---|
| H20 | IR (CRITICAL) | 2.41 | 48.2 | 51.12 | 1 |
| | OR (CRITICAL) | 4.02 | 80.3 | 51.12 | 1 |
| | IR ( POS MOM) | 2.41 | 48.2 | 51.12 | 1 |
| | OR ( POS MOM) | 4.02 | 80.3 | 51.12 | 1 |
| HS20 | IR (CRITICAL) | 2.00 | 72.0 | 52.13 | 1 |
| | OR (CRITICAL) | 3.33 | 120.0 | 52.13 | 1 |
| | IR ( POS MOM) | 2.00 | 72.0 | 52.13 | 1 |
| | OR ( POS MOM) | 3.33 | 120.0 | 52.13 | 1 |
| TK527 | IR (CRITICAL) | 1.75 | 70.2 | 52.13 | 1 |
| | OR (CRITICAL) | 2.92 | 117.0 | 52.13 | 1 |
| | IR ( POS MOM) | 1.75 | 70.2 | 52.13 | 1 |
| | OR ( POS MOM) | 2.92 | 117.0 | 52.13 | 1 |
| ML80 | IR (CRITICAL) | 1.78 | 65.3 | 52.13 | 1 |
| | OR (CRITICAL) | 2.97 | 108.9 | 52.13 | 1 |
| | IR ( POS MOM) | 1.78 | 65.3 | 52.13 | 1 |
| | OR ( POS MOM) | 2.97 | 108.9 | 52.13 | 1 |

NOTE: THIS RATING SUMMARY OF GIRDERS DOES FOLLOW THE ALL-OR-NONE COMPACT REQUIREMENTS.

```
        THIS GIRDER DOES HAVE ALL SECTIONS QUALIFYING AS COMPACT
        SECTIONS.  THEREFORE, THE RATING PRINTED IN THE SUMMARY
        IS THAT BASED ON COMPACT SECTIONS

      RATING FACTOR CODES: FIRST CHARACTER AFTER THE RATING FACTOR
      T - TOP STEEL STRESS/STRENGTH GOVERNS
      B - BOTTOM STEEL STRESS/STRENGTH GOVERNS
      C - CONCRETE STRESS/STRENGTH GOVERNS
      R - REINFORCEMENT STRESS/STRENGTH GOVERNS
      V - SHEAR STRESS/STRENGTH GOVERNS
      blank - COMPACT MOMENT STRENGTH GOVERNS
      O - OVERLOAD PROVISIONS GOVERN
      I - MOMENT-SHEAR INTERACTION GOVERNS

      RATING FACTOR CODES FOR FLANGE/WEB BUCKLING: SECOND CHARACTER AFTER THE RATING
FACTOR
      blank - SECTION DOES MEET FLANGE/WEB BUCKLING CRITERIA
          AND WILL NOT BUCKLE.
      F - SECTION DOES NOT MEET FLANGE PROJECTION/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE.
      W - SECTION DOES NOT MEET WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN WEB.
      Z - SECTION DOES NOT MEET BOTH FLANGE PROJECTION/THICKNESS RATIO
          CRITERIA AND WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE OR WEB.

      NON-COMPACT MOMENT STRENGTH CODES:
      B - SECTION IS BRACED
      U - SECTION IS UNBRACED

  NOTE: ALL RATINGS ARE BASED ON THE NUMBER OF DESIGN LANES OR THE ACTUAL
        TRAFFIC LANES AS DEFINED BY "D" OR "L" ENTERED FOR LANES IN THE
        PROJECT IDENTIFICATION.

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

### 3.9.8.3 Exterior Girder Load Rating Analysis

#### 3.9.8.3.1 BAR7 Input Parameters

Exterior girder 9 was selected and will be rated using BAR7. Many of the BAR7 input parameters can be left blank. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

<u>Project Identification</u>

- Project Identification = "=BRRAT"
- Structure ID = 02400300101709
- Description = EXTERIOR BEAM 9
- Bridge Type = GGG
- Live Load = Blank for H20, HS20, ML80, TK527. Enter L for EV2 and EV3 vehicles.
- Output = 3 for Load Factor Rating since the structure was designed by the Load Factor Method.
- Concrete Deck = Y
- Skew Correction Factor = 1.206 (See below)
  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

$$\text{SCF} = 1.0 + 0.2\left(\frac{12Lt_s^3}{K_g}\right)^{0.3} \tan\ \theta$$

where,

L = Span Length = 103.0625 ft
$t_s$ = Deck Thickness = 8.5 in
θ = AASHTO Skew Angle = (90° – 42° 53' 00") = 47.117° (skew angle at obtuse corner of bridge)
$e_g$ = Distance between the c.g. of the deck and c.g. of the beam = 30.015 in + 4.25 in = 34.265 in
  Beam c.g. from Top of Beam = 46.875 in – 16.86 in (calc'd by BAR7) = 30.015 in
  Deck c.g. from Top of Beam = 8.5 in / 2 = 4.25 in (Neglect haunch per 1993 DM-4 10.5.1.1 since it was not field measured)
n = Modular Ratio Between the Beam and Deck = 29,000 ksi / 3605 ksi = 8.04
  Steel Modulus of Elasticity, Es = 29,000 ksi
  Concrete Compressive Strength, $f'_c$ = 4 ksi (AAA Concrete per Contract Drawings)
  Concrete Modulus of Elasticty, Ec = 57,000(f'c)0.5 = 57,000(4000 psi)0.5/1000 = 3605 ksi

I = Moment of Inertia of the Beam = 25871.32 $in^4$ (non-composite beam, calculated by BAR7)

A = Beam Area = 71.625 $in^2$ (Use non-composite beam area per AASHTO 4.6.2.2.1, calculated by BAR7)

$K_g$ = Longitudinal Stiffness Parameter = $n(I+Ae_g^2)$ = 8.04[25871.32 $in^4$ + 71.625 $in^2$ x (34.265 in)$^2$] = 884,123 $in^4$

The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

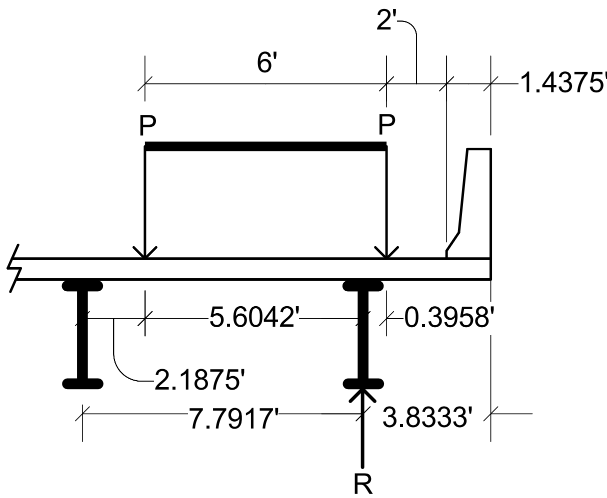
| | |
|---|---|
| 0° ≤ θ ≤ 60° | θ = 47.117°, OK |
| 3.5 ft ≤ S ≤ 16 ft | S = 7.792 ft, OK |
| 20 ft ≤ L ≤ 240 ft | L = 103.06 ft, OK |
| $N_b$ ≥ 4 | $N_b$ = 9, OK |

Bridge Cross Section and Loading

- Spacing = 7.7917 ft / 2 + 3.8333 ft = 7.73 ft per Contract Drawings

- Distribution Factor – Shear

  Per AASHTO 3.23.1, the exterior beam distribution factor for shear is based on the Lever Rule. In accordance with AASHTO 3.12, a reduction in load intensity is applied when multiple traffic lanes are loaded simultaneously. For the one-lane condition shown below, the reduction in load intensity factor is 1.0.




  $DF_V$ = P[(7.7917’ + 0.3958’)/7.7917’ + (7.7917’ – 5.6042’)/7.7917’] x 1.0 = 1.293 wheels

  $DF_V$ = 1.293 wheels / (2 wheels/axle) = 0.647 axles

- Distribution Factor – Moment

  Per AASHTO 3.23.2.3.1.2, the exterior beam distribution factor for moment is based on the Lever Rule. The moment distribution factor is the same as calculated for shear.

  $DF_M$ = 0.647 axles

- Distribution Factor – Deflection
  Per AASHTO 10.6.4, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12’ Lane Width)
  Roadway Width = 70 ft – 2 x 1.4375 ft = 67.125 ft (Neglects Median Barrier)
  $N_L$ = 67.125 ft / 12 ft = 5.59 ==> Use 5 lanes
  Reduction Factor = 0.75 for 5 lanes per AASHTO 3.12.1
  $DF_{Defl}$ = 5 lanes x 0.75 / 9 beams = 0.417 lanes/beam

- Deck Thickness = 8.5 in

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight, deck slab weight, SIP forms, haunch concrete, intermediate diaphragms, and stiffeners. Girder self-weight and deck slab weight are calculated by BAR7. The remaining loads are calculated below.

  SIP Form Weight
  In accordance with DM-4 3.3.3, an additional dead load of 15 psf of deck area must be included if stay-in-place forms are present. For this example, the form weight is applied over the entire girder spacing.
  SIP Form Weight = 0.015 ksf x 7.7917 ft / 2 = 0.058 kips/ft

  Haunch and Overhang Weight
  Referencing the provided deck drawings, the depth of the deck slab over the centerline of the girder is 9 3/8 in. The depth of the concrete haunch at the centerline of the girder is equal to 9.375 in - 8.5 in = 0.875 in. The top flange width of the girder is 14 in.
  Haunch Weight = 0.875 in x 14 in x 0.150 kcf / 144 $in^2/ft^2$ = 0.013 kips/ft

  Overhang Weight = 0.5 x 0.06 x $(3.833 \text{ ft} – 1.4375 \text{ ft} – 14 \text{ in}/2/12)^2$ x 0.150 kcf = 0.015 kips/ft

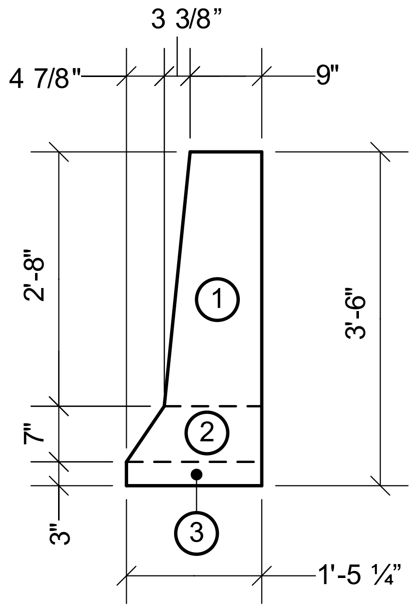
  Intermediate Stiffeners
  The transverse intermediate stiffeners are 3/8 in x 5 in x 44 in.
  The total number of stiffeners is 6 + 5 + 5 + 5 + 6 = 27.
  Stiffeners: 27 x 0.375 in x 5 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ / 103.06 ft = 0.006 kips/ft

  Total Dead Load 1 = 0.058 + 0.013 + 0.015 + 0.006 = 0.092 kips/ft

  Dead loads due to intermediate diaphragms and end diaphragms are entered as concentrated (patch) loads and will be discussed later.

- Dead Loads – DL2
  The dead loads acting on the composite section include outside barriers and median barrier. Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

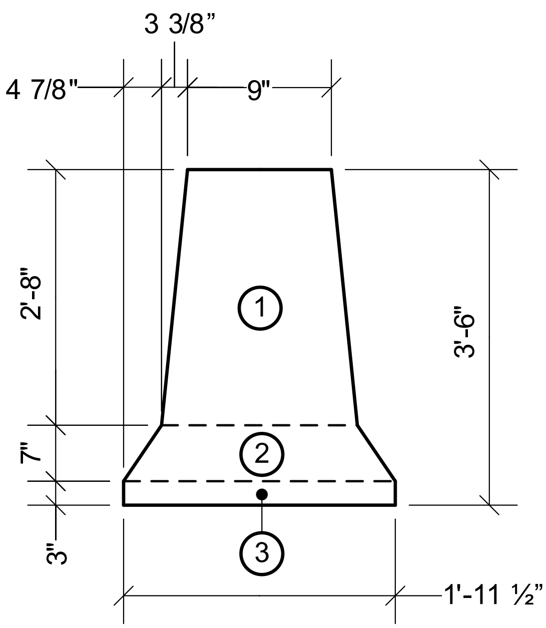
Outside Roadway Barriers




Area 1 = (9 in + 12.375 in)/2 x 32 in = 342.0 in$^2$
Area 2 = (12.375 in + 17.25 in)/2 x 7 in = 103.7 in$^2$
<u>Area 3 = 17.25 in x 3 in = 51.8 in$^2$</u>
Total Area = 497.5 in$^2$

Outside Barrier Weight = 497.5 in$^2$ / 144 in$^2$/ft$^2$ x 0.150 kcf
Outside Barrier Weight = 0.518 kips/ft

Median Barrier




Area 1 = (9 in + 15.75 in)/2 x 32 in = 396.0 in$^2$
Area 2 = (15.75 in + 23.5 in)/2 x 7 in =137.4 in$^2$
<u>Area 3 = 23.5 in x 3 in = 70.5 in$^2$</u>
Total Area = 603.9 in$^2$

Median Barrier Weight = 603.4 in$^2$ / 144 in$^2$/ft$^2$ x 0.150 kcf
Median Barrier Weight = 0.629 kips/ft

Total Dead Load 2 = (2 x 0.518 kips/ft + 0.629 kips/ft) / 9 beams = <u>0.185 kips/ft</u>

- Concrete Compressive Strength, $f'_c$ = 4 ksi for Class AAA Concrete per Drawings and DM-4 8.2 since the bridge was built after 8/16/1989.

- Modular Ratio, n = 8 for Class AAA Concrete per DM-4 8.2

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = 103'-0 ¾" = 103.06 ft per Contract Drawings for Beam 9

Concrete Member Properties

- Type = O for Steel Bridge
- Reinforcement Yield Strength, $f_y$ = 60 ksi per Contract Drawings and DM-4 8.2

Steel Member Properties

- GFS Type = G for Girder
- Span Number = 1
- Range = 103.06 ft
- Section Type = P for Plate Girder
- Beam Depth = 44 in
- Web Thickness = 0.5 in
- Top Plate Width = 14 in
- Top Plate Thickness = 1 in
- Bottom Plate Width = 19 in
- Bottom Plate Thickness = 1.875 in
- Composite = Y for a composite superstructure
- Yield Strength of Girder = 50 ksi per Contract Drawings

Lateral Brace Points

Lateral brace points for a girder should be entered based on which flange is in compression. For a simple span bridge, the top flange is in compression for the entire span length while the bottom flange is in tension.

- Bracing or Stiffener Code = BG for brace points of a girder
- Span Number = 1
- Bracing Point or Stiffener Code = C for continuously braced top flange
- Number of Spaces = 1
- Spacing = 103.06 ft
- Bracing or Stiffener Code = SG for stiffener spacings in a girder
- Span Number = 1
- Bracing Point or Stiffener Code = T for transversely stiffened girder web. Transverse stiffeners are equally spaced between intermediate diaphragms. Diaphragm spacing = 103.06 ft / 5 spaces = 20.61 ft.
- Number of Spaces = 1, Spacing = 1'-10" = 1.83 ft
- Number of Spaces = 6, Spacing = (20.61 ft – 1.83 ft) / 6 spaces = 3.13 ft
- Number of Spaces = 18, Spacing = 3 x 20.61 ft / 18 spaces = 3.44 ft
- Number of Spaces = 6, Spacing = (20.61 ft – 1.83 ft) / 6 spaces = 3.13 ft
- Number of Spaces = 1, Spacing = 1'-10" = 1.83 ft

Concentrated Patch Loads

Applying intermediate and end diaphragm weights as patch loads typically isn't required unless a more accurate rating is desired. For this example, these loads are applied as patch loads. The addition of the end diaphragm weight should not change the ratings, however, these loads were included so a more accurate bearing reaction could be calculated by the program.

Intermediate Diaphragms

The intermediate diaphragms consist of a W24x68 beam, MC6x12 channel, connection plates, and bolts placed perpendicular to the girder. The weight of the intermediate diaphragms is applied as point loads along the span at the diaphragm locations.

W24x68: 4 x 0.068 kips/ft x 7.7917 ft / 2 = 1.06 kips
MC6x12: 4 x 0.012 kips/ft x 7.7917 ft / 2 = 0.19 kips
Connection Plates: 4 x 0.625 in x 7.5 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.23 kips
Intermediate Diaphragm Weight = 1.06 + 0.19 + 0.23 = 1.48 kips

Diaphragm Spacing = 103.06 ft / 5 spaces = 20.61 ft
Diaphragm Locations = 20.61 ft, 41.23 ft, 61.84 ft, 82.45 ft

End Diaphragms

The end diaphragms consist of a W27x84 beam, jacking stiffeners, jacking plates, connections plates, and bearing stiffeners.

Jacking Stiffeners: 4 x 0.75 in x 4.25 in x (26.69 in – 2 x 0.636 in) x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.092 kips
Jacking Plates: 4 x 1.25 in x 8.5 in x 8.5 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.102 kips
Connection Plates: 2 x 0.5 in x 10 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.125 kips
Bearing Stiffeners: 2 x 1 in x 9 in x 44 in x 0.490 kcf / $12^3$ $in^3/ft^3$ = 0.225 kips

End Diaphragm Length at South Abutment = 45.80 ft / 4 = 11.45 ft (Abutment 1)
W27x84 Diaphragm: 0.084 kips/ft x 11.45 ft = 0.962 kips (Abutment 1)
End Diaphragm Weight at South Abutment = (0.092 + 0.102 + 0.125 + 0.225 + 0.962) / 2 = 0.75 kips

End Diaphragm Length at North Abutment = 48.24 ft / 4 = 12.06 ft (Abutment 2)
W27x84 Diaphragm: 0.084 kips/ft x 12.06 ft = 1.013 kips (Abutment 2)
End Diaphragm Weight at South Abutment = (0.092 + 0.102 + 0.125 + 0.225 + 1.013) / 2 = 0.78 kips

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 02-4003-0010-1709 | | |
| Description: | | | |
| Bridge Type: | GGG | | Multigirder |
| SLC Level: | | | Super = 5 & Sub = 6; ADTT = 23 therefore SLC = 1.0 |
| Lanes: | | | N/A for CTB |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 3 | | Load Factor Ratings |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | Y | | |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | | | Leave blank and calculate DF |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | 1.206 | | See Hand Calcs |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for GGG |
| Overhang or Spacing: | 7.73 | ft | See Hand Calcs |
| CL of Girder: | -- | ft | Leave blank for GGG |
| Roadway Width: | -- | ft | Leave blank for GGG |
| Distr Factor -Shear: | 0.647 | | See Hand Calcs |
| Distr Factor -Moment: | 0.647 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.417 | | See Hand Calcs |
| Slab Thickness: | 8.50 | in | Per plans |
| Haunch: | | in | |
| Bridge DL1: | 0.092 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.185 | kip/ft | See Hand Calcs |
| F'c: | 4.0 | ksi | Per plans |
| N: | 8 | | Class AAA per DM-4 8.2 |
| Symmetry: | | | |
| LL Location: | | | Only applies to truss |
| Number of Panels: | | | Only applies to truss |
| End Connections: | | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | | | Only applies to truss |
| Stringer DL: | | kip/ft | N/A for CTB |
| Floorbeam DL: | | kip/ft | N/A for CTB |
| Unit weight of Concrete: | | lb/cf | N/A for steel grid |
| Gusset Plate Analysis: | | | N/A for CTB |
| Patch Load Analysis: | | | N/A for CTB |
| Unsymmetrical Pier Support: | | | N/A for CTB |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 130.06 | ft | Per plans |

| Steel Member Properties | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| GFS | G | | | | | | | |
| Span Number | 1 | | | | | | | |
| Range | 103.06 | | | | | | | |
| Section Type | P | | | | | | | |
| WF Beam M of I | | | | | | | | |
| WF Beam A | | | | | | | | |
| Flange Thickness | | | | | | | | |
| Flange Width | | | | | | | | |
| Varies | | | | | | | | |
| WF Beam Depth | 44.00 | | | | | | | |
| Web Thickness | 0.5 | | | | | | | |
| Top Plate Width | 14 | | | | | | | |
| Top Plate Thickness | 1 | | | | | | | |
| Bottom Plate Width | 19 | | | | | | | |
| Bottom Plate Thickness | 1.875 | | | | | | | |
| Composite | Y | | | | | | | |
| Yield Strength | 50 | | | | | | | |
| Hybrid fy Top | | | | | | | | |
| Hybrid fy Bottom | | | | | | | | |

| Lateral Bracing | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Bracing or Stiff Code | BG | SG | | | | | | |
| Span Number | 1 | 1 | | | | | | |
| Code 1 | C | T | | | | | | |
| Num of Spaces | 1 | 1 | | | | | | |
| Spacing 1 | 103.06 | 1.83 | | | | | | |
| Code 2 | | T | | | | | | |
| Num of Spaces | | 6 | | | | | | |
| Spacing 2 | | 3.13 | | | | | | |
| Code 3 | | T | | | | | | |
| Num of Spaces | | 18 | | | | | | |
| Spacing 3 | | 3.44 | | | | | | |
| Code 4 | | T | | | | | | |
| Num of Spaces | | 6 | | | | | | |
| Spacing 4 | | 3.13 | | | | | | |
| Code 5 | | T | | | | | | |
| Num of Spaces | | 1 | | | | | | |
| Spacing 5 | | 1.83 | | | | | | |
| Code 6 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 6 | | | | | | | | |
| Code 7 | | | | | | | | |
| Num of Spaces 7 | | | | | | | | |
| Spacing 7 | | | | | | | | |
| Code 8 | | | | | | | | |
| Num of Spaces 8 | | | | | | | | |
| Spacing 8 | | | | | | | | |

### 3.9.8.3.2 BAR7 Output

```
******************************************************************************
*                                                                            *
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*                                                                            *
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```

BRIDGE ANALYSIS AND RATING (BAR7) 330427

PROGRAM P4353000 05/01/2024 12:57
VERSION 7.15.0.0 LAST UPDATED 02/15/2018 DOCUMENTATION 02/2018

INPUT: Exterior Steel Girder BAR7 Analysis.dat

PROJECT IDENTIFICATION

STRUCTURE ID - 02400300101709 - EXTERIOR BEAM NO. 9

| BRG TYPE | SLC LEV | LANES | LIVE LOAD | OUT-PUT | IMP FACT | GAGE DIST | PASS DIST | FAT-IGUE | CONC DECK | SPEC | RE-DIST | DIR | S OVER FACTOR | END PAN |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| GGG | | | | 3 | 0.00 | 0.0 | 0.0 | | Y | | | | 0.00 | |

| HYB | SKEW CORR FACTOR | PONY TRUSS | PDF | COMPACT |
|---|---|---|---|---|
| | 1.206 | | Y | |

BRIDGE CROSS SECTION AND LOADING

| DECK WIDTH | OVERHANG OR SPACING | CL OF GIRDER OR TRUSS TO CURB | ROADWAY WIDTH | DISTRIBUTION FACTORS SHEAR | MOMENT | DEFLECT |
|---|---|---|---|---|---|---|
| 0.00 | 7.73 | 0.00 | 0.00 | 0.647 | 0.647 | 0.417 |

| SLAB THICKNESS | HAUNCH | DEAD LOADS DL1 | DL2 | F'C | N | SYMMETRY |
|---|---|---|---|---|---|---|
| 8.50 | 0.00 | 0.092 | 0.185 | 4.000 | 8. | |

| STRINGER DL1 | FLOORBEAM DL1 | UNIT WEIGHT DECK CONCRETE | PATCH LOAD ANALYSIS | UNSYMM PIER SUPPORT |
|---|---|---|---|---|
| 0.000 | 0.000 | 0.00 | Y | |

SPAN LENGTHS (SIMPLE)

| SPAN # | 1 |
|---|---|
| LENGTH | 103.06 |

CONCRETE MEMBER PROPERTIES

| TYPE | DEPTH | B | D | AS | D' | A'S | FY REINF |
|---|---|---|---|---|---|---|---|
| O | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 60. |

| ALLOWABLE FS IR | OR | ST DET | AV | SPECS | ALPHA | INTEGRAL WEARING SURFACE |
|---|---|---|---|---|---|---|
| 0.0 | 0.0 | | 0.00 | 0 | 0. | 0.0 |

STEEL MEMBER PROPERTIES

```
  S         T  WF BM    WF BM   FLANGE            WF BM
G P         Y  M OF I    AREA     OR          V  OR WEB
F A         P  OR VRT   OR HRZ  ANGLE  FLANGE  A   PLATE    WEB
S N  RANGE E   LEG      LEG    THICK   WIDTH  R   DEPTH   THICK  SHAPE
G 1 103.06 P     0.00     0.00  0.0000  0.000       44.00  0.5000
               TPW   TPT    BPW   BPT   COMP  FY  FY TOP FY BOT CG TOP CG BOT
             14.00 1.0000 19.00 1.8750    Y  50.0   0.0    0.0   0.000  0.000
```

```
                LATERAL BRACE POINTS AND STIFFENER SPACINGS

             C               C               C               C
B OR S       O  NO.          O  NO.          O  NO.          O  NO.
G OR F       D  OF           D  OF           D  OF           D  OF
 CODE  SPAN  E SPCS SPACING  E SPCS SPACING  E SPCS SPACING  E SPCS SPACING
  BG     1   C   1  103.06      0    0.00       0    0.00       0    0.00
                 0    0.00      0    0.00       0    0.00       0    0.00
  SG     1   T   1    1.83   T  6    3.13    T 18    3.44    T  6    3.13
             T   1    1.83      0    0.00       0    0.00       0    0.00
```

```
                                     CONCENTRATED PATCH LOADS

MEM LOAD SPAN                | LOAD SPAN                 | LOAD SPAN                 |
TYP TYPE  NO    DIST     MAG | TYPE  NO    DIST     MAG  | TYPE  NO    DIST     MAG  |
 G  DC1S   1   20.61    1.48 | DC1S   1   41.23    1.48  | DC1S   1   61.84    1.48  |

MEM LOAD SPAN                |
TYP TYPE  NO    DIST     MAG |
 G  DC1S   1   82.45    1.48 |

MEM LOAD SPAN                | LOAD SPAN                 | LOAD SPAN                 |
TYP TYPE  NO    DIST     MAG | TYPE  NO    DIST     MAG  | TYPE  NO    DIST     MAG  |
 G  DC1S   1    0.00    0.75 | DC1S   1  103.06    0.78  |        0    0.00    0.00  |
```

```
                                      DEFAULT VALUES

                                 UNIT                            INTEGRAL
 SLC     GAGE    PASSING    WEIGHT    FY    ALLOWABLE FS  WEARING   SKEW CORR
LEVEL  DISTANCE  DISTANCE    DECK    REINF   IR      OR    SURFACE    FACTOR
  I       6.0       4.0      150.0    ---   24.0    36.0     0.5      -----
```

```
                ++++++++++++++++++++++++++++++++++++++++++++
                +                                          +
                +       G I R D E R   A N A L Y S I S      +
                +                                          +
                ++++++++++++++++++++++++++++++++++++++++++++
```

```
                          DEAD LOADS ACTING ON GIRDER

 INPUT  GIRDER   SLAB   FL BEAM  STRINGER  FL BEAM STRINGER   TOTAL   TOTAL
  DL1   WEIGHT  WEIGHT   WEIGHT   WEIGHT     DL1      DL1       DL1     DL2
 0.092   0.244   0.821   0.000    0.000    0.000    0.000     1.157   0.185

NOTE: IF A SECTION DOES NOT MEET FLANGE OR WEB BUCKLING CRITERIA
      OF CURRENT AASHTO SPECIFICATIONS FOR LOAD FACTOR METHOD, THE
      RATING FACTORS ARE REPRINTED AS 888.88.  THIS INDICATES
      THAT THERE IS A POTENTIAL FATIGUE PROBLEM.
```

GIRDER SECTION PROPERTIES

SPAN 1 - EFFECTIVE SLAB WIDTH: 92.76 THICKNESS: 8.00
=======

| | DEPTH | GROSS AREA | MOMENT OF INERTIA | C BOTTOM | SECTION MODULUS TOP | SECTION MODULUS BOTTOM | CONC OR NEG REINF |
|---|---|---|---|---|---|---|---|
| NON-COMPOSITE | 46.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | |
| COMPOSITE (N= 8) | 54.88 | 164.38 | 73117.68 | 36.06 | 6758.26 | 2027.89 | 3885.31 |
| COMPOSITE (N=24) | 54.88 | 102.55 | 51018.00 | 27.12 | 2582.45 | 1881.24 | 1838.11 |
| COMPOSITE (NEG M) | 54.88 | 71.62 | 25871.32 | 16.86 | 862.07 | 1534.10 | N/A |

**************
* DEAD LOADS *
**************

*******
* DL1 *
*******

=================================================================
DL1 DEAD LOAD INCLUDING USER-ENTERED DL1 AND BAR7-COMPUTED DL1.
=================================================================

| DISTANCE BEGIN | DISTANCE END | INTENSITY BEGIN | INTENSITY END |
|---|---|---|---|
| 0.000 | 103.060 | 1.157 | 1.157 |

===========================================================================
TOTAL DL1 DEAD LOAD EFFECT INCLUDING USER-ENTERED DL1, BAR7-COMPUTED DL1, AND DL1 PATCH LOADS
===========================================================================

| SUPPORT NO | REACTION |
|---|---|
| 1 | 63.332 |
| 2 | 63.362 |

| SPAN NO | SECTION NO | DISTANCE | DM1 | DS1 | DEF1 |
|---|---|---|---|---|---|
| 1 | 1 | 0.00 | 0.000 | 62.582 | 0.000 |
| 1 | 2 | 10.31 | 583.523 | 50.658 | 1.291 |
| 1 | 3 | 20.61 | 1044.150 | 37.253 | 2.444 |
| 1 | 4 | 30.92 | 1366.634 | 25.329 | 3.348 |
| 1 | 5 | 41.22 | 1566.225 | 13.404 | 3.922 |
| 1 | 6 | 51.53 | 1627.679 | -0.000 | 4.118 |
| 1 | 7 | 61.84 | 1566.231 | -11.925 | 3.922 |
| 1 | 8 | 72.14 | 1366.643 | -25.329 | 3.348 |
| 1 | 9 | 82.45 | 1044.156 | -37.253 | 2.444 |
| 1 | 10 | 92.75 | 583.526 | -50.658 | 1.291 |

```
    1     11   103.06       0.000   -62.582   0.000

 SUPPORT NO    END SHEAR

      1         -62.582

                                   *******
                                   * DL2 *
                                   *******


  =========================================
  DL2 DEAD LOAD INCLUDING USER-ENTERED DL2
  =========================================



         DISTANCE            INTENSITY
      BEGIN     END       BEGIN      END


      0.000   103.060     0.185     0.185

  ==========================================================================
  TOTAL DL2 DEAD LOAD EFFECT INCLUDING USER-ENTERED DL2 AND DL2 PATCH LOADS
  ==========================================================================


SUPPORT NO    REACTION

     1           9.533
     2           9.533

 SPAN SECTION
  NO     NO   DISTANCE       DM2        DS2     DEF2

   1      1     0.00       0.000      9.533   0.000
   1      2    10.31      88.423      7.626   0.099
   1      3    20.61     157.196      5.720   0.187
   1      4    30.92     206.320      3.813   0.256
   1      5    41.22     235.794      1.907   0.300
   1      6    51.53     245.619     -0.000   0.315
   1      7    61.84     235.794     -1.907   0.300
   1      8    72.14     206.320     -3.813   0.256
   1      9    82.45     157.196     -5.720   0.187
   1     10    92.75      88.423     -7.626   0.099
   1     11   103.06       0.000     -9.533   0.000

 SUPPORT NO    END SHEAR

      1          -9.533


                      ********************************
                      *  GIRDER  -  LIVE LOAD  H20   *
                      ********************************

                               MAXIMUM REACTIONS

                                                       REACTIONS     MOMENTS
  SUPPORT    DL1       DL2    +(LL+I)     -(LL+I)  +I.F. -I.F.  +I.F. -I.F.
     1      63.3       9.5     46.5 L       0.0     1.22
```

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 62.6 | 9.5 | 56.1L | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 10.31 | 583.5 | 88.4 | 373.0L | 0.0 | | 50.7 | 7.6 | 48.1L | -3.1 | 1.23 |
| | SIMULT | SHEAR | 40.9 | 0.0 | | SIMULT | MOM | 407.4 | 234.1 | |
| 20.61 | 1044.1 | 157.2 | 663.1L | 0.0 | | 37.3 | 5.7 | 40.6L | -6.7 | 1.24 |
| | SIMULT | SHEAR | 33.1 | 0.0 | | SIMULT | MOM | 681.4 | 449.6 | |
| 30.92 | 1366.6 | 206.3 | 870.4L | 0.0 | | 25.3 | 3.8 | 33.6L | -10.7 | 1.25 |
| | SIMULT | SHEAR | 25.2 | 0.0 | | SIMULT | MOM | 838.0 | 621.1 | |
| 41.22 | 1566.2 | 235.8 | 994.7L | 0.0 | | 13.4 | 1.9 | 27.2L | -15.5L | 1.27 |
| | SIMULT | SHEAR | 17.2 | 0.0 | | SIMULT | MOM | 893.4 | 764.7 | |
| 51.53 | 1627.7 | 245.6 | 1036.1L | 0.0 | | -0.0 | -0.0 | 21.3L | -21.3L | 1.28 |
| | SIMULT | SHEAR | -9.0 | 0.0 | | SIMULT | MOM | 863.6 | 863.6 | |
| 61.84 | 1566.2 | 235.8 | 994.7L | 0.0 | | -11.9 | -1.9 | 15.5L | -27.2L | 1.27 |
| | SIMULT | SHEAR | -17.2 | 0.0 | | SIMULT | MOM | 764.7 | 893.4 | |
| 72.14 | 1366.6 | 206.3 | 870.4L | 0.0 | | -25.3 | -3.8 | 10.7 | -33.6L | 1.25 |
| | SIMULT | SHEAR | -25.2 | 0.0 | | SIMULT | MOM | 621.1 | 838.0 | |
| 82.45 | 1044.2 | 157.2 | 663.1L | 0.0 | | -37.3 | -5.7 | 6.7 | -40.6L | 1.24 |
| | SIMULT | SHEAR | -33.1 | 0.0 | | SIMULT | MOM | 449.6 | 681.4 | |
| 92.75 | 583.5 | 88.4 | 373.0L | 0.0 | | -50.7 | -7.6 | 3.1 | -48.1L | 1.23 |
| | SIMULT | SHEAR | -40.9 | 0.0 | | SIMULT | MOM | 234.1 | 407.4 | |
| 103.06 | 0.0 | 0.0 | 0.0 | 0.0 | | -62.6 | -9.5 | 0.0 | -56.1L | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR
UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER
SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 8449.6 B | 8027.1 | 638.0 | 4.48 V | 7.46 V | 9305.9 | 4.48 V | 7.46 V |
| 10.31 | 8449.6 B | 8027.1 | 638.0 | 5.40 V | 8.99 V | 9305.9 | 5.40 V | 8.99 V |
| 20.61 | 8449.6 B | 8027.1 | 637.2 | 4.15 I | 6.91 I | 9305.9 | 4.15 I | 6.91 I |
| 30.92 | 8449.6 B | 8027.1 | 637.2 | 3.08 B | 5.14 B | 9305.9 | 3.85 | 6.42 |
| 41.22 | 8449.6 B | 8027.1 | 637.2 | 2.52 B | 4.20 B | 9305.9 | 3.23 | 5.38 |
| 51.53 | 8449.6 B | 8027.1 | 637.2 | 2.36 B | 3.94 B | 9305.9 | 3.06 | 5.10 |
| 61.84 | 8449.6 B | 8027.1 | 637.2 | 2.52 B | 4.20 B | 9305.9 | 3.23 | 5.38 |
| 72.14 | 8449.6 B | 8027.1 | 637.2 | 3.08 B | 5.14 B | 9305.9 | 3.85 | 6.42 |
| 82.45 | 8449.6 B | 8027.1 | 637.2 | 4.15 I | 6.91 I | 9305.9 | 4.15 I | 6.91 I |
| 92.75 | 8449.6 B | 8027.1 | 638.0 | 5.40 V | 8.99 V | 9305.9 | 5.40 V | 8.99 V |
| 103.06 | 8449.6 B | 8027.1 | 638.0 | 4.48 V | 7.46 V | 9305.9 | 4.48 V | 7.46 V |

```
*******************************
*  GIRDER  -  LIVE LOAD  HS20  *
```

********************************

MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 63.3 | 9.5 | 51.7 | 0.0 | 1.22 | | | |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 62.6 | 9.5 | 62.3 | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 10.31 | 583.5 | 88.4 | 473.8 | 0.0 | | 50.7 | 7.6 | 55.9 | -3.1 | 1.23 |
| | SIMULT | SHEAR | 55.9 | 0.0 | | SIMULT | MOM | 473.8 | 234.1 | |
| 20.61 | 1044.1 | 157.2 | 830.5 | 0.0 | | 37.3 | 5.7 | 49.5 | -8.2 | 1.24 |
| | SIMULT | SHEAR | 49.5 | 0.0 | | SIMULT | MOM | 830.5 | 549.8 | |
| 30.92 | 1366.6 | 206.3 | 1070.2 | 0.0 | | 25.3 | 3.8 | 42.9 | -14.8 | 1.25 |
| | SIMULT | SHEAR | 42.9 | 0.0 | | SIMULT | MOM | 1070.2 | 858.2 | |
| 41.22 | 1566.2 | 235.8 | 1210.5 | 0.0 | | 13.4 | 1.9 | 36.3 | -22.0 | 1.27 |
| | SIMULT | SHEAR | 31.6 | 0.0 | | SIMULT | MOM | 1192.8 | 1086.8 | |
| 51.53 | 1627.7 | 245.6 | 1242.5 | 0.0 | | -0.0 | -0.0 | 29.5 | -29.5 | 1.28 |
| | SIMULT | SHEAR | -24.8 | 0.0 | | SIMULT | MOM | 1198.3 | 1198.3 | |
| 61.84 | 1566.2 | 235.8 | 1210.5 | 0.0 | | -11.9 | -1.9 | 22.0 | -36.3 | 1.27 |
| | SIMULT | SHEAR | -31.6 | 0.0 | | SIMULT | MOM | 1086.8 | 1192.8 | |
| 72.14 | 1366.6 | 206.3 | 1070.2 | 0.0 | | -25.3 | -3.8 | 14.8 | -42.9 | 1.25 |
| | SIMULT | SHEAR | -42.9 | 0.0 | | SIMULT | MOM | 858.2 | 1070.2 | |
| 82.45 | 1044.2 | 157.2 | 830.5 | 0.0 | | -37.3 | -5.7 | 8.2 | -49.5 | 1.24 |
| | SIMULT | SHEAR | -49.5 | 0.0 | | SIMULT | MOM | 549.8 | 830.5 | |
| 92.75 | 583.5 | 88.4 | 473.8 | 0.0 | | -50.7 | -7.6 | 3.1 | -55.9 | 1.23 |
| | SIMULT | SHEAR | -55.9 | 0.0 | | SIMULT | MOM | 234.1 | 473.8 | |
| 103.06 | 0.0 | 0.0 | 0.0 | 0.0 | | -62.6 | -9.5 | 0.0 | -62.3 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 8449.6 B | 8027.1 | 638.0 | 4.03 V | 6.72 V | 9305.9 | 4.03 V | 6.72 V |
| 10.31 | 8449.6 B | 8027.1 | 638.0 | 4.64 V | 7.73 V | 9305.9 | 4.64 V | 7.73 V |
| 20.61 | 8449.6 B | 8027.1 | 637.2 | 3.40 I | 5.67 I | 9305.9 | 3.40 I | 5.67 I |
| 30.92 | 8449.6 B | 8027.1 | 637.2 | 2.51 B | 4.18 B | 9305.9 | 3.13 | 5.22 |
| 41.22 | 8449.6 B | 8027.1 | 637.2 | 2.07 B | 3.45 B | 9305.9 | 2.66 | 4.43 |
| 51.53 | 8449.6 B | 8027.1 | 637.2 | 1.97 B | 3.29 B | 9305.9 | 2.55 | 4.25 |
| 61.84 | 8449.6 B | 8027.1 | 637.2 | 2.07 B | 3.45 B | 9305.9 | 2.66 | 4.43 |

```
 72.14    8449.6 B  8027.1   637.2   2.51 B   4.18 B   9305.9   3.13     5.22
 82.45    8449.6 B  8027.1   637.2   3.40 I   5.67 I   9305.9   3.40 I   5.67 I
 92.75    8449.6 B  8027.1   638.0   4.64 V   7.73 V   9305.9   4.64 V   7.73 V
103.06    8449.6 B  8027.1   638.0   4.03 V   6.72 V   9305.9   4.03 V   6.72 V


                       *******************************
                       *  GIRDER  -  LIVE LOAD TK527  *
                       *******************************

                                MAXIMUM REACTIONS

                                                         REACTIONS     MOMENTS
    SUPPORT    DL1       DL2   +(LL+I)     -(LL+I)  +I.F. -I.F.  +I.F. -I.F.
       1      63.3       9.5     58.7         0.0    1.22


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
      ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
      SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.


                         UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======
          DL1     DL2  +(LL+I)  -(LL+I)  I   DL1     DL2  +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT  M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
    0.00     0.0     0.0    0.0      0.0      62.6    9.5   70.8      0.0  1.22
          SIMULT SHEAR    0.0      0.0      SIMULT MOM     0.0      0.0
   10.31   583.5    88.4  537.9      0.0      50.7    7.6   63.5     -3.4  1.23
          SIMULT SHEAR   63.5      0.0      SIMULT MOM   537.9    256.4
   20.61  1044.1   157.2  941.8      0.0      37.3    5.7   56.1     -9.3  1.24
          SIMULT SHEAR   56.1      0.0      SIMULT MOM   941.8    621.5
   30.92  1366.6   206.3 1224.8      0.0      25.3    3.8   48.6    -16.6  1.25
          SIMULT SHEAR   31.6      0.0      SIMULT MOM  1211.8    965.8
   41.22  1566.2   235.8 1386.8      0.0      13.4    1.9   41.0    -24.7  1.27
          SIMULT SHEAR   23.8      0.0      SIMULT MOM  1347.8   1217.8
   51.53  1627.7   245.6 1414.7      0.0      -0.0   -0.0   33.2    -33.2  1.28
          SIMULT SHEAR   -9.9      0.0      SIMULT MOM  1349.7   1349.7
   61.84  1566.2   235.8 1386.8      0.0     -11.9   -1.9   24.7    -41.0  1.27
          SIMULT SHEAR  -23.8      0.0      SIMULT MOM  1217.8   1347.8
   72.14  1366.6   206.3 1224.8      0.0     -25.3   -3.8   16.6    -48.6  1.25
          SIMULT SHEAR  -31.6      0.0      SIMULT MOM   965.8   1211.8
   82.45  1044.2   157.2  941.8      0.0     -37.3   -5.7    9.3    -56.1  1.24
          SIMULT SHEAR  -56.1      0.0      SIMULT MOM   621.5    941.8
   92.75   583.5    88.4  537.9      0.0     -50.7   -7.6    3.4    -63.5  1.23
          SIMULT SHEAR  -63.5      0.0      SIMULT MOM   256.4    537.9
  103.06     0.0     0.0    0.0      0.0     -62.6   -9.5    0.0    -70.8  1.22
          SIMULT SHEAR    0.0      0.0      SIMULT MOM     0.0      0.0


NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR
      UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER
      SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.


                     STRENGTHS AND LOAD FACTOR RATINGS

SPAN  1
=======
        NON-COMP OVERLOAD          NON-COMPACT      COMPACT       COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS    MOMENT   RATING FACTORS
```

| X | STRENGTH | STRENGTH | STRENGTH | IR | OR | STRENGTH | IR | OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 8449.6 B | 8027.1 | 638.0 | 3.55 V | 5.91 V | 9305.9 | 3.55 V | 5.91 V |
| 10.31 | 8449.6 B | 8027.1 | 638.0 | 4.09 V | 6.81 V | 9305.9 | 4.09 V | 6.81 V |
| 20.61 | 8449.6 B | 8027.1 | 637.2 | 3.00 I | 5.00 I | 9305.9 | 3.00 I | 5.00 I |
| 30.92 | 8449.6 B | 8027.1 | 637.2 | 2.19 B | 3.65 B | 9305.9 | 2.74 | 4.56 |
| 41.22 | 8449.6 B | 8027.1 | 637.2 | 1.81 B | 3.01 B | 9305.9 | 2.32 | 3.86 |
| 51.53 | 8449.6 B | 8027.1 | 637.2 | 1.73 B | 2.89 B | 9305.9 | 2.24 | 3.74 |
| 61.84 | 8449.6 B | 8027.1 | 637.2 | 1.81 B | 3.01 B | 9305.9 | 2.32 | 3.86 |
| 72.14 | 8449.6 B | 8027.1 | 637.2 | 2.19 B | 3.65 B | 9305.9 | 2.74 | 4.56 |
| 82.45 | 8449.6 B | 8027.1 | 637.2 | 3.00 I | 5.00 I | 9305.9 | 3.00 I | 5.00 I |
| 92.75 | 8449.6 B | 8027.1 | 638.0 | 4.09 V | 6.81 V | 9305.9 | 4.09 V | 6.81 V |
| 103.06 | 8449.6 B | 8027.1 | 638.0 | 3.55 V | 5.91 V | 9305.9 | 3.55 V | 5.91 V |

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*******************************
*  GIRDER  -  LIVE LOAD  ML80  *
*******************************
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MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 63.3 | 9.5 | 55.8 | 0.0 | 1.22 | | | |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 62.6 | 9.5 | 67.3 | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 10.31 | 583.5 | 88.4 | 513.4 | 0.0 | | 50.7 | 7.6 | 60.6 | -3.6 | 1.23 |
| | SIMULT | SHEAR | 60.6 | 0.0 | | SIMULT | MOM | 513.4 | 276.7 | |
| 20.61 | 1044.1 | 157.2 | 904.0 | 0.0 | | 37.3 | 5.7 | 53.8 | -10.0 | 1.24 |
| | SIMULT | SHEAR | 53.8 | 0.0 | | SIMULT | MOM | 904.0 | 670.4 | |
| 30.92 | 1366.6 | 206.3 | 1178.3 | 0.0 | | 25.3 | 3.8 | 47.0 | -17.5 | 1.25 |
| | SIMULT | SHEAR | 29.7 | 0.0 | | SIMULT | MOM | 1171.9 | 1016.2 | |
| 41.22 | 1566.2 | 235.8 | 1347.3 | 0.0 | | 13.4 | 1.9 | 40.1 | -25.1 | 1.27 |
| | SIMULT | SHEAR | 22.6 | 0.0 | | SIMULT | MOM | 1317.0 | 1239.2 | |
| 51.53 | 1627.7 | 245.6 | 1393.6 | 0.0 | | -0.0 | -0.0 | 33.0 | -33.0 | 1.28 |
| | SIMULT | SHEAR | 15.3 | 0.0 | | SIMULT | MOM | 1339.5 | 1339.5 | |
| 61.84 | 1566.2 | 235.8 | 1347.3 | 0.0 | | -11.9 | -1.9 | 25.1 | -40.1 | 1.27 |
| | SIMULT | SHEAR | -22.6 | 0.0 | | SIMULT | MOM | 1239.2 | 1317.0 | |
| 72.14 | 1366.6 | 206.3 | 1178.3 | 0.0 | | -25.3 | -3.8 | 17.5 | -47.0 | 1.25 |
| | SIMULT | SHEAR | -29.7 | 0.0 | | SIMULT | MOM | 1016.2 | 1171.9 | |
| 82.45 | 1044.2 | 157.2 | 904.0 | 0.0 | | -37.3 | -5.7 | 10.0 | -53.8 | 1.24 |
| | SIMULT | SHEAR | -53.8 | 0.0 | | SIMULT | MOM | 670.4 | 904.0 | |
| 92.75 | 583.5 | 88.4 | 513.4 | 0.0 | | -50.7 | -7.6 | 3.6 | -60.6 | 1.23 |
| | SIMULT | SHEAR | -60.6 | 0.0 | | SIMULT | MOM | 276.7 | 513.4 | |
| 103.06 | 0.0 | 0.0 | 0.0 | 0.0 | | -62.6 | -9.5 | 0.0 | -67.3 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 8449.6 B | 8027.1 | 638.0 | 3.74 V | 6.23 V | 9305.9 | 3.74 V | 6.23 V |
| 10.31 | 8449.6 B | 8027.1 | 638.0 | 4.28 V | 7.14 V | 9305.9 | 4.28 V | 7.14 V |
| 20.61 | 8449.6 B | 8027.1 | 637.2 | 3.12 I | 5.21 I | 9305.9 | 3.12 I | 5.21 I |
| 30.92 | 8449.6 B | 8027.1 | 637.2 | 2.28 B | 3.79 B | 9305.9 | 2.84 | 4.74 |
| 41.22 | 8449.6 B | 8027.1 | 637.2 | 1.86 B | 3.10 B | 9305.9 | 2.39 | 3.98 |
| 51.53 | 8449.6 B | 8027.1 | 637.2 | 1.76 B | 2.93 B | 9305.9 | 2.28 | 3.79 |
| 61.84 | 8449.6 B | 8027.1 | 637.2 | 1.86 B | 3.10 B | 9305.9 | 2.39 | 3.98 |
| 72.14 | 8449.6 B | 8027.1 | 637.2 | 2.28 B | 3.79 B | 9305.9 | 2.84 | 4.74 |
| 82.45 | 8449.6 B | 8027.1 | 637.2 | 3.12 I | 5.21 I | 9305.9 | 3.12 I | 5.21 I |
| 92.75 | 8449.6 B | 8027.1 | 638.0 | 4.28 V | 7.14 V | 9305.9 | 4.28 V | 7.14 V |
| 103.06 | 8449.6 B | 8027.1 | 638.0 | 3.74 V | 6.23 V | 9305.9 | 3.74 V | 6.23 V |

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++++++++++++++++++++++++++++++++++++++++++++
+                                          +
+        R A T I N G   S U M M A R Y       +
+                                          +
++++++++++++++++++++++++++++++++++++++++++++
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MEMBER: GIRDER

| LOAD | | LOAD FACTOR RATING FACTOR | TONS | X | SPAN |
|---|---|---|---|---|---|
| H20 | IR (CRITICAL) | 3.06 | 61.2 | 51.53 | 1 |
| | OR (CRITICAL) | 5.10 | 102.0 | 51.53 | 1 |
| | IR ( POS MOM) | 3.06 | 61.2 | 51.53 | 1 |
| | OR ( POS MOM) | 5.10 | 102.0 | 51.53 | 1 |
| HS20 | IR (CRITICAL) | 2.55 | 91.9 | 51.53 | 1 |
| | OR (CRITICAL) | 4.25 | 153.1 | 51.53 | 1 |
| | IR ( POS MOM) | 2.55 | 91.9 | 51.53 | 1 |
| | OR ( POS MOM) | 4.25 | 153.1 | 51.53 | 1 |
| TK527 | IR (CRITICAL) | 2.24 | 89.7 | 51.53 | 1 |
| | OR (CRITICAL) | 3.74 | 149.4 | 51.53 | 1 |
| | IR ( POS MOM) | 2.24 | 89.7 | 51.53 | 1 |
| | OR ( POS MOM) | 3.74 | 149.4 | 51.53 | 1 |
| ML80 | IR (CRITICAL) | 2.28 | 83.4 | 51.53 | 1 |
| | OR (CRITICAL) | 3.79 | 139.0 | 51.53 | 1 |
| | IR ( POS MOM) | 2.28 | 83.4 | 51.53 | 1 |
| | OR ( POS MOM) | 3.79 | 139.0 | 51.53 | 1 |

NOTE: THIS RATING SUMMARY OF GIRDERS DOES FOLLOW THE ALL-OR-NONE COMPACT REQUIREMENTS.
THIS GIRDER DOES HAVE ALL SECTIONS QUALIFYING AS COMPACT SECTIONS. THEREFORE, THE RATING PRINTED IN THE SUMMARY IS THAT BASED ON COMPACT SECTIONS

RATING FACTOR CODES: FIRST CHARACTER AFTER THE RATING FACTOR
T - TOP STEEL STRESS/STRENGTH GOVERNS
B - BOTTOM STEEL STRESS/STRENGTH GOVERNS
C - CONCRETE STRESS/STRENGTH GOVERNS
R - REINFORCEMENT STRESS/STRENGTH GOVERNS
V - SHEAR STRESS/STRENGTH GOVERNS
blank - COMPACT MOMENT STRENGTH GOVERNS
O - OVERLOAD PROVISIONS GOVERN
I - MOMENT-SHEAR INTERACTION GOVERNS

```
      RATING FACTOR CODES FOR FLANGE/WEB BUCKLING: SECOND CHARACTER AFTER THE RATING
FACTOR
      blank - SECTION DOES MEET FLANGE/WEB BUCKLING CRITERIA
          AND WILL NOT BUCKLE.
      F - SECTION DOES NOT MEET FLANGE PROJECTION/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE.
      W - SECTION DOES NOT MEET WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN WEB.
      Z - SECTION DOES NOT MEET BOTH FLANGE PROJECTION/THICKNESS RATIO
          CRITERIA AND WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE OR WEB.

      NON-COMPACT MOMENT STRENGTH CODES:
      B - SECTION IS BRACED
      U - SECTION IS UNBRACED

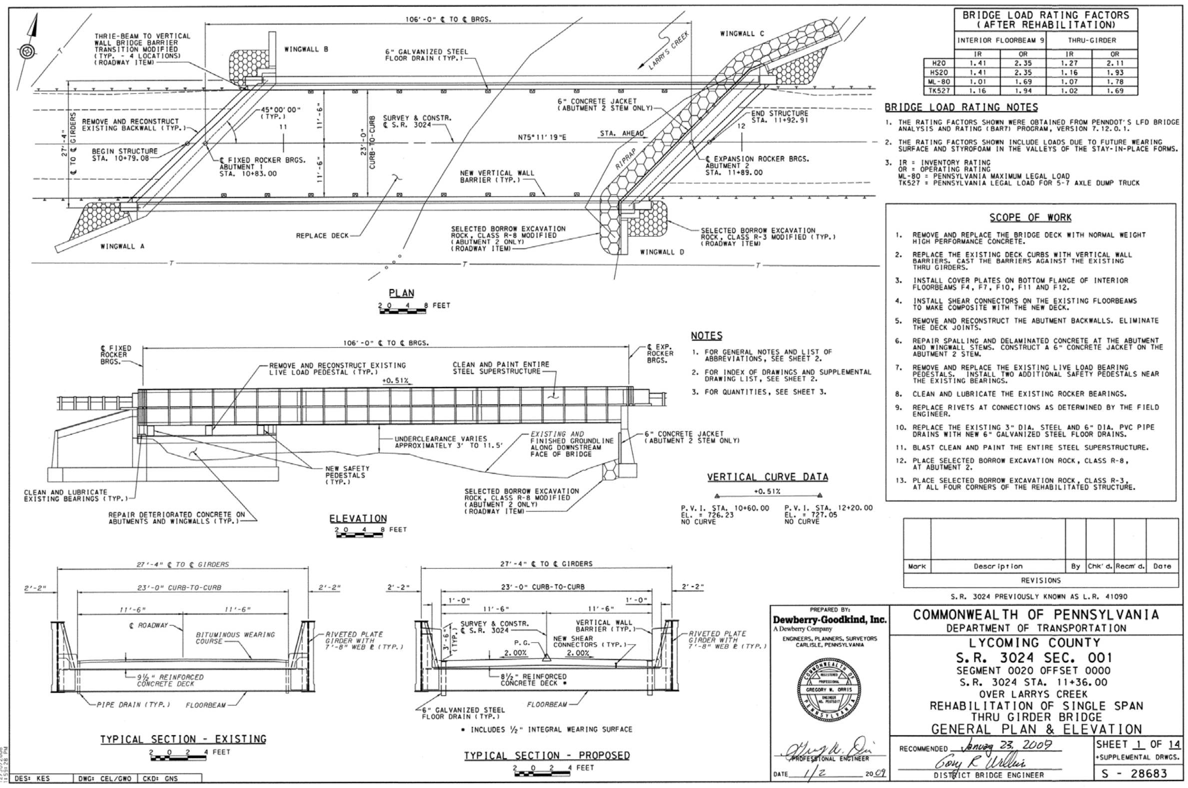
  NOTE: ALL RATINGS ARE BASED ON THE NUMBER OF DESIGN LANES OR THE ACTUAL
        TRAFFIC LANES AS DEFINED BY "D" OR "L" ENTERED FOR LANES IN THE
        PROJECT IDENTIFICATION.

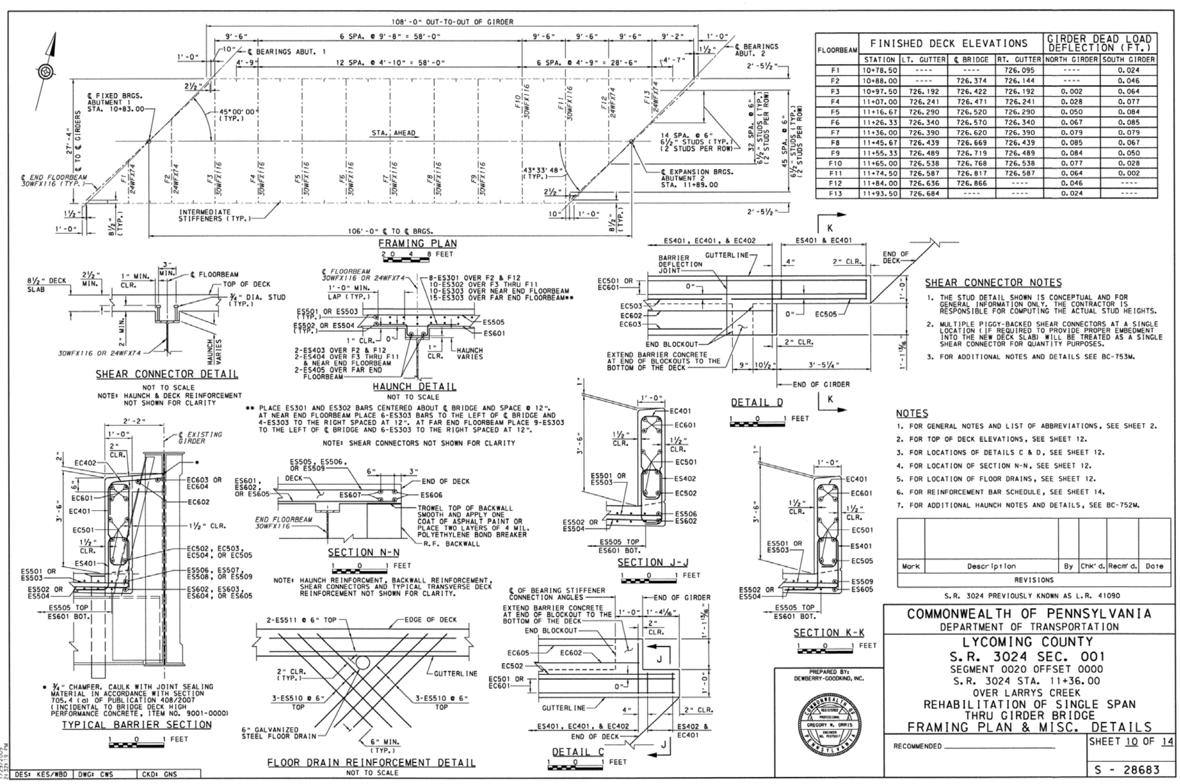
BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

### 3.9.9 Sample Load Rating – Steel Thru-Girder/Girder Floorbeam System

This Section contains sample load rating calculations for a typical floorbeam and exterior girder of a single span composite steel thru-girder structure shown below. Section loss on the exterior girder will be modeled. The analysis will be performed using PennDOT's Bridge Analysis and Rating (BAR7) Program, Version 7.15.0.0 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part-IV (DM-4).

The single span superstructure consists of an 8 ½" reinforced concrete deck composite with and supported on rolled beam floorbeams with cover plates. Details from the Contract Drawings are shown below.


BRIDGE LOAD RATING FACTORS (AFTER REHABILITATION)
BRIDGE LOAD RATING NOTES
SCOPE OF WORK
PLAN
ELEVATION
NOTES
VERTICAL CURVE DATA
TYPICAL SECTION - EXISTING
TYPICAL SECTION - PROPOSED
S.R. 3024 PREVIOUSLY KNOWN AS L.R. 41090
Dewberry-Goodkind, Inc.
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
LYCOMING COUNTY
S.R. 3024 SEC. 001
SEGMENT 0020 OFFSET 0000
S.R. 3024 STA. 11+36.00
OVER LARRYS CREEK
REHABILITATION OF SINGLE SPAN
THRU GIRDER BRIDGE
GENERAL PLAN & ELEVATION
SHEET 1 OF 14
S - 28683


FINISHED DECK ELEVATIONS
GIRDER DEAD LOAD DEFLECTION (FT.)
FRAMING PLAN
SHEAR CONNECTOR DETAIL
HAUNCH DETAIL
SHEAR CONNECTOR NOTES
DETAIL D
NOTES
SECTION N-N
SECTION J-J
SECTION K-K
TYPICAL BARRIER SECTION
FLOOR DRAIN REINFORCEMENT DETAIL
DETAIL C
S.R. 3024 PREVIOUSLY KNOWN AS L.R. 41090
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
LYCOMING COUNTY
S.R. 3024 SEC. 001
SEGMENT 0020 OFFSET 0000
S.R. 3024 STA. 11+36.00
OVER LARRYS CREEK
REHABILITATION OF SINGLE SPAN
THRU GIRDER BRIDGE
FRAMING PLAN & MISC. DETAILS
SHEET 10 OF 14
S - 28683

### 3.9.9.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

Done By: Date:

Checked By: Date:

Structure ID (5A01): Inspection Date (7A01):

Facility Carried (5A08): SR 3024 over Larrys Creek

Feature Intersected (5A07): Larrys Creek

Structure Type (6A26 - 6A29): Thru-Girder Structure

Spans / Members Analyzed: Girder #2 and Floorbeam #3

Analysis Method: ASD/LFD

PennDOT Program / Version: BAR7 Version 7.15.0.0

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 1.35 | 26.9 | 2.24 | 44.9 | 2.02 | 40.4 | * | * | V | V |
| HS20 | 1.21 | 43.7 | 2.02 | 72.8 | 1.82 | 65.5 | * | * | T | T |
| ML80 | 1.08 | 39.7 | 1.81 | 66.2 | 1.63 | 59.6 | * | * | T | T |
| TK527 | 1.07 | 42.6 | 1.78 | 71.0 | 1.60 | 63.9 | * | * | T | T |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 1.47 | 42.2 | 2.45 | 70.4 | 2.20 | 63.4 | * | * | T | T |
| EV3 | 0.97 | 41.7 | 1.62 | 69.5 | 1.46 | 62.6 | * | * | T | T |

Comments/Assumptions*:

Superstructure and substructure condition ratings are 6 and 4, respectively. Therefore, the SLC Factor equals 0.9 per PennDOT Publication 238, Table IP 4.3.2-1.
Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.
BAR7 analysis includes section loss.
* Controlling Member is Exterior Girder 2 in Span 1. T = Top Steel Strength Governs.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

#### 3.9.9.2 Exterior Girder and Typical Floorbeam Load Rating Analysis

##### 3.9.9.2.1 BAR7 Input Parameters

Exterior girder 2 was selected along with a typical floorbeam and will be rated using BAR7. Many of the BAR7 input parameters can be left blank. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 41302400200000
- Description = EXT BEAM 2 W/ FLOORBEAM
- Bridge Type = GFF
- Lanes = D for BAR7 to calculate based on the number of possible lanes
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 3 for Load Factor Rating since the structure was designed by the Load Factor Method.
- Concrete Deck = Y

Bridge Cross Section and Loading

- Deck Width = 25'-0" = 25.0 ft
- Deck Overhang = 0 ft
- CL Girder to Edge of Curb = 2'-2" = 2.167 ft per Contract Drawings
- Roadway Width = 23'-0" = 23.0 ft per Contract Drawings
- Slab Thickness = 8.5 in
- Haunch Thickness = 0 in
- Dead Loads – DL1
  The non-composite dead loads acting on the girder include girder self-weight, deck slab weight, SIP forms, haunch concrete, stiffeners, knee braces, and splice plates…etc. Girder self-weight and deck slab weight are calculated by BAR7. The weight of the SIP forms and haunch weight are entered as floorbeam dead loads and then applied to the girder. The remaining loads are calculated below.

  Web Stiffeners
  The web stiffeners are L6"x3.5"x3/8" with a length of 7.65 ft.
  The total number of web stiffeners is 33.
  Web Stiffeners: 33 x 0.0117 klf x 7.65 ft / 106.00 ft = 0.0279 kips/ft

  Floorbeam Stiffeners
  The floorbeam stiffeners are L7"x4"x1/2" with a length of 2.40 ft.
  The total number of floorbeam stiffeners is 11.
  Floorbeam Stiffeners: 11 x 0.0179 klf x 2.40 ft / 106.00 ft = 0.0045 kips/ft
  Stiffener for Floorbeam Connection ≈ 2 x stiffener weight = 2 x 0.0045 = 0.0089 kips/ft
  Total Floorbeam Stiffeners = 0.0045 kips/ft + 0.0089 kips/ft = 0.0134 kips/ft

Longitudinal Stiffeners
The longitudinal stiffeners are L4"x3"x1/2" with a length of 106.00 ft.
The total number of longitudinal web stiffeners is 1.
Longitudinal Stiffeners: 1 x 0.0111 klf x 106.00 ft / 106.00 ft = 0.0111 kips/ft

Bearing Stiffeners
The bearing stiffeners are L7"x4"x5/8" with a length of 7.65 ft.
The total number of bearing stiffeners is 8.
Bearing Stiffeners: 8 x 0.0221 klf x 7.65 ft / 106.00 ft = 0.0128 kips/ft

Splice Plates
The splice plates are 6.25 ft x 1.50 ft x 0.50 in.
The total number of splice plates is 4.
Splice Plates: 4 x 6.25 ft x 1.50 ft x 0.50 in x 0.490 kcf / 12 in/ft / 106.00 ft = 0.0072 kips/ft

Knee Braces
The knee braces are L3.5"x3.5"x3/8" with a length of 4.50 ft.
The total number of knee braces is 11.
Knee Braces: 11 x 0.0085 klf x 4.50 ft / 106.00 ft = 0.0040 kips/ft

Plates Connecting Knee Brace to Girder
The plates are 4.50 ft x 11 in x 0.375 in.
The total number of connection plates is 11.
Connection Plates: 11 x 4.50 ft x 11 in x 0.375 in x 0.490 kcf / 12 in/ft / 106.00 ft = 0.0066 kips/ft

Total Dead Load 1 = 0.0279 + 0.0134 + 0.0111 + 0.0128 + 0.0072 + 0.0040 + 0.0066 = 0.083 kips/ft

- Dead Loads – DL2
  The dead loads acting on the composite section include concrete barriers. Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  As shown on the Contract Drawings, the barrier consists a 3'-6" x 1'-0" wide vertical wall barrier with an additional piece that is 1.17 ft wide x 0.70 ft high.

  Barrier Cross-Sectional Area = 3.50 ft x 1 ft + 1.17 ft x 0.70 ft = 4.319 $ft^2$
  Barrier Weight = 4.319 $ft^2$ x 0.150 kcf = 0.648 kips/ft

  Total Dead Load 2 = 0.648 kips/ft

- Concrete Compressive Strength, $f'_c$ = 4 ksi for Class AAA Concrete per DM-4 8.2

- Modular Ratio, n = 8 for Class AAA Concrete per DM-4 8.2

- Floorbeam Dead Load – DL1
  The floorbeam dead load 1 includes the weight of the haunch, permanent formwork, bracing, stiffeners and other hardware attached to the floorbeam. Floorbeam self-weight and deck slab weight are calculated by BAR7.

  SIP Form Weight
  The form weight for this structure is less than the typical 0.015 ksf since the valleys of the floor are blocked out.
  SIP Form Density = 0.005 ksf
  Floorbeam Spacing = 9.667 ft
  Top Flange Width = 10.5 in / 12 = 0.875 ft
  SIP Form Weight = 0.005 ksf x (9.667 ft – 0.875 ft) = 0.044 kips/ft

  Haunch Weight
  The haunch depth varies along the floorbeam. Use 2.5 in as an average haunch depth.
  Haunch Weight = 2.50 in x 10.5 in x 0.150 kcf / 144 $in^2/ft^2$ = 0.027 kips/ft

  The Floorbeam Dead Load 1 = 0.044 kips/ft + 0.027 kips/ft = 0.071 kips/ft

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = 106’-0” = 106.00 ft per Contract Drawings

Stringer (Floorbeam) Span Lengths

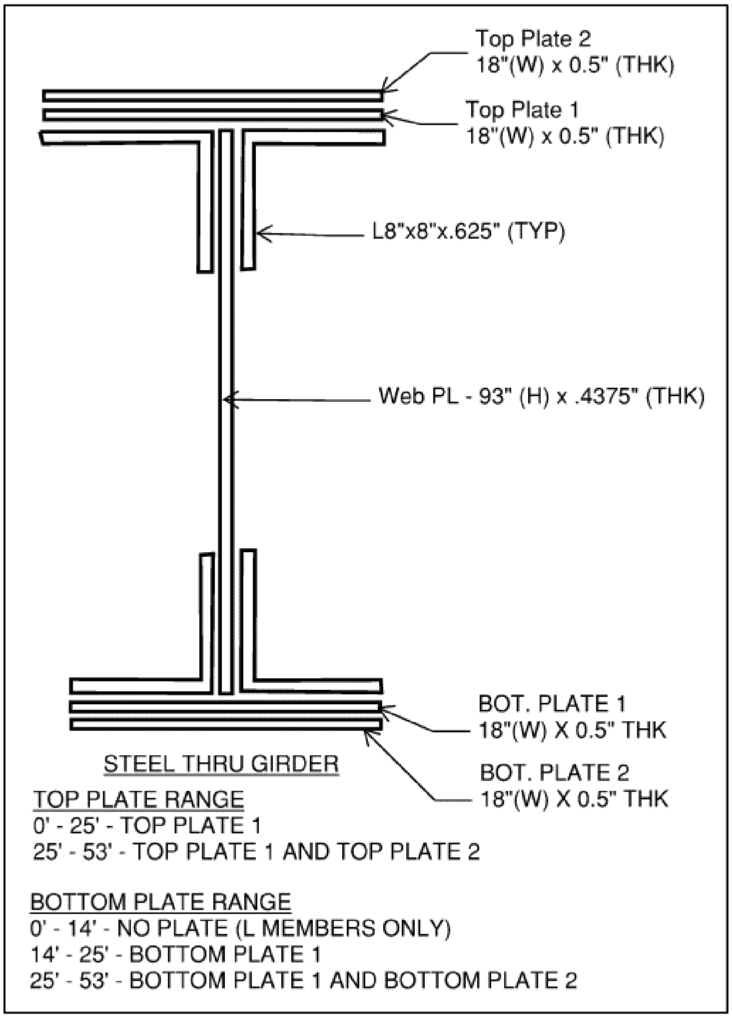
- Continuity = S for Simple Span
- Span Length 1 = 9.17 ft
- Span Length 2 = 9.50 ft
- Span Length 3 = 9.50 ft
- Span Length 4 = 9.50 ft
- Span Length 5 = 9.67 ft
- Span Length 6 = 9.67 ft
- Span Length 7 = 9.67 ft
- Span Length 8 = 9.67 ft
- Span Length 9 = 9.67 ft
- Span Length 10 = 9.67 ft
- Span Length 11 = 9.50 ft

Concrete Member Properties

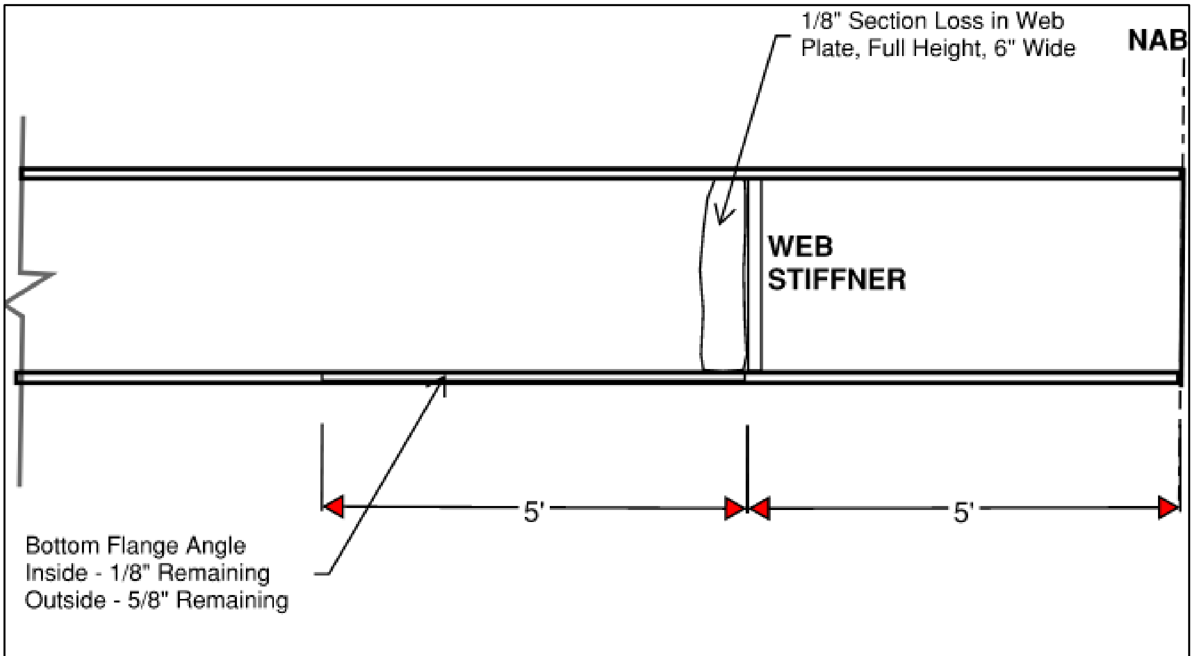
- Type = O for Steel Bridge
- Reinforcement Yield Strength, $f_y$ = 60 ksi per DM-4 8.2

Steel Member Properties

The steel member sketches and section properties are shown on the following pages.

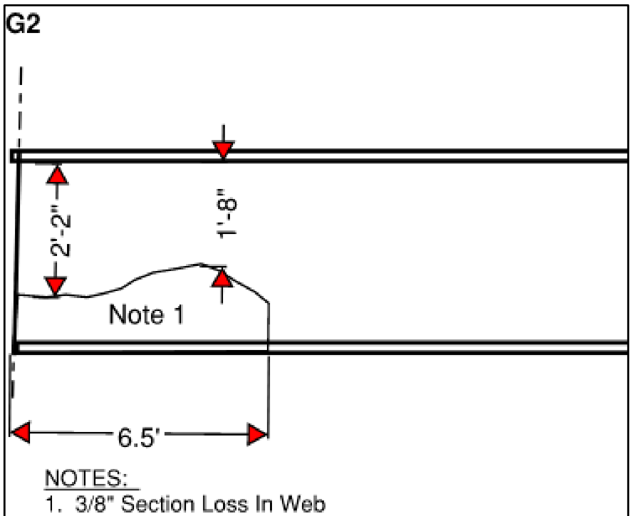


Girder G2 has section loss in the web and bottom flange as shown in the sketch below.
Remaining web section from 5 ft to 5.5 ft = 0.4375 in – 0.125 in = 0.3125 in
Remaining bottom flange section of angle from 5 ft to 10 ft = (0.625 in + 0.125 in) / 2 = 0.375 in




GIRDER G2 SECTION LOSS NOTES

Floorbeam 3 has section loss in the web as shown in the sketch below.
Web area without section loss = 0.564 in x (26 in + 20 in) / 2 = 12.972 in$^2$
Web area with section loss = (0.564 in – 0.375 in ) x 5.30 in = 1.002 in$^2$
Remaining web section from 0 ft to 6.5 ft = (12.972 in$^2$ + 1.002 in$^2$) / (30 in – 2 x 0.85 in) = 0.494 in



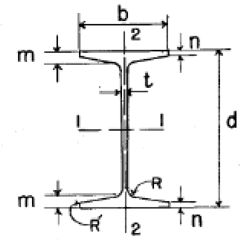

FLOORBEAM F3 SECTION LOSS NOTES

# 30" BEAMS

REFERENCES; SEE COLUMN (1) AND PAGE 4

| | | | |
|---|---|---|---|
| 2 6<br>3 7<br>4 13<br>5 16<br>See Page 41<br>10 14<br>See Page 42 | 11<br>S12-1922<br>S15-1924<br>S16-1925<br>S18-1926 | 12<br>S24-1927<br>S27-1928<br>S34-1930<br>S35-1930 | 14<br>C1927<br>C1928<br>C1929<br>C1930 |

FLOORBEAM 3




*Computed

| SECT. NO. OR NOM. SIZE | COL. (1) | WEIGHT PER FOOT Lb. | AREA Sq.In. | DEPTH d In. | FLANGE WIDTH b In. | WEB THICK t In. | DIMENSIONS m In. | n In. | R In. | R′ In. | SLOPE INSIDE FLANGE % | AXIS 1—1 I In.⁴ | S In.³ | r In. | AXIS 2—2 I In.⁴ | S In.³ | r In. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| CB30I 30X10½ | 13 | 125.0 | 36.75 | 30.148 | 10.546 | .576 | .956 | .956 | .70 | 0 | 0 | 5441.7 | 361.0 | 12.17 | 187.4 | 35.5 | 2.26 |
| 30WF (B30) 30X10½ | 4 | 124.0 | 36.45 | 30.160 | 10.521 | .585 | .930† | | .65 | 0 | 5.0 | 5347.1 | 354.6 | 12.11 | 169.7 | 32.3 | 2.16 |
| 30WF CB30I 30X10½ | 18 | 124.0 | 36.45 | 30.160 | 10.521 | .585 | .930 | .930 | .70 | 0 | 0 | 5347.1 | 354.6 | 12.11 | 169.7 | 32.3 | 2.16 |
| B30 30X10½ | 2 | 122.0 | 35.87 | 30.120 | 10.525 | .580 | 1.117 | .703 | .65 | 0 | 8⅓* | 5235.7 | 347.7 | 12.08 | 158.4 | 30.1 | 2.10 |
| CB30IN 30X10½ | 16 | 122.0 | 35.85 | 30.120 | 10.525 | .580 | .910 | .910 | .65 | 0 | 0 | 5238.2 | 347.8 | 12.09 | 177.3 | 33.7 | 2.22 |
| B30 | 3 | 121.0 | 35.65 | 30.000 | 10.500 | .550 | 1.155 | .740 | .65 | 0 | 8⅓* | 5269.7 | 351.3 | 12.16 | 164.3 | 31.3 | 2.15 |
| B30 | 10 | 121.0 | 35.36 | 30.000 | 10.500 | .550 | 1.155 | .740 | .65 | 0 | 8⅓* | 5213.6 | 347.6 | 12.14 | 164.3 | 31.3 | 2.16 |
| B30 | 11 | 121.0 | 35.30 | 30.000 | 10.500 | .540 | 1.183 | .735 | .65 | 0 | 9.0* | 5239.6 | 349.3 | 12.18 | 165.0 | 31.4 | 2.16 |
| B30 | 7 | 120.0 | 35.30 | 30.000 | 10.500 | .540 | 1.183 | .735 | .65 | 0 | 9.0* | 5239.6 | 349.3 | 12.18 | 165.0 | 31.4* | 2.16 |
| B30 | 5 | 120.0 | 35.25 | 30.000 | 10.000 | .520 | 1.333 | .740 | .62 | 0 | 12½* | 5270.9 | 351.4 | 12.23 | 149.7 | 29.9* | 2.11 |
| 30WF (B30) 30X10½ | 4 | 116.0 | 34.13 | 30.000 | 10.500 | .564 | .850† | | .65 | 0 | 5.0 | 4919.1 | 327.9 | 12.00 | 153.2 | 29.2 | 2.12 |
| 30WF CB30I 30X10½ | 18 | 116.0 | 34.13 | 30.000 | 10.500 | .564 | .850 | .850 | .70 | 0 | 0 | 4919.1 | 327.9 | 12.00 | 153.2 | 29.2 | 2.12 |
| B30 | 2 | 115.0 | 33.85 | 30.000 | 10.500 | .555 | 1.057 | .643 | .65 | 0 | 8⅓* | 4894.1 | 326.3 | 12.02 | 145.6 | 27.7 | 2.07 |
| CB30IN 30X10½ | 16 | 115.0 | 33.84 | 30.000 | 10.500 | .555 | .850 | .850 | .65 | 0 | 0 | 4896.6 | 326.4 | 12.03 | 164.5 | 31.3 | 2.20 |
| CB30I | 14 | 115.0 | 33.81 | 30.000 | 10.500 | .530 | .882 | .882 | .70 | 0 | 0 | 4985.3 | 332.4 | 12.14 | 170.6 | 32.5 | 2.25 |
| B30 | 3 | 115.0 | 33.80 | 29.880 | 10.480 | .530 | 1.095 | .680 | .65 | 0 | 8⅓* | 4942.9 | 330.8 | 12.09 | 151.8 | 29.0 | 2.12 |
| B30 | 10 | 115.0 | 33.50 | 29.880 | 10.480 | .530 | 1.095 | .680 | .65 | 0 | 8⅓* | 4886.8 | 327.1 | 12.08 | 151.8 | 29.0 | 2.13 |
| B30 | 12 | 110.0 | 32.45 | 29.780 | 10.470 | .520 | 1.045 | .630 | .65 | 0 | 8⅓* | 4687.7 | 314.8 | 12.02 | 141.8 | 27.1 | 2.09 |
| B30 30X10½ | 2 | 108.0 | 31.85 | 29.880 | 10.475 | .530 | .997 | .583 | .65 | 0 | 8⅓* | 4556.2 | 305.0 | 11.96 | 132.9 | 25.4 | 2.04 |
| CB30IN 30X10½ | 16 | 108.0 | 31.77 | 29.880 | 10.473 | .528 | .790 | .790 | .65 | 0 | 0 | 4554.2 | 304.8 | 11.97 | 151.6 | 29.0 | 2.18 |
| 30WF (B30) 30X10½ | 4 | 108.0 | 31.77 | 29.820 | 10.484 | .548 | .760† | | .65 | 0 | 5.0 | 4461.0 | 299.2 | 11.85 | 135.1 | 25.8 | 2.06 |
| 30WF CB30I 30X10½ | 18 | 108.0 | 31.77 | 29.820 | 10.484 | .548 | .760 | .760 | .70 | 0 | 0 | 4461.0 | 299.2 | 11.85 | 135.1 | 25.8 | 2.06 |

†Average thickness

Lateral Brace Points

Lateral brace points for a girder and floorbeam should be entered based on which flange is in compression. For a simple span bridge, the top flange is in compression for the entire span length while the bottom flange is in tension.

- Bracing or Stiffener Code = BF for brace points of a floorbeam
- Span Number = 4
- Bracing Point or Stiffener Code = C for continuously braced top flange
- Number of Spaces = 1, Spacing = 27.00 ft
- Bracing or Stiffener Code = BG for brace points of a girder
- Span Number = 1
- Bracing Point or Stiffener Code = T for transversely stiffened girder web
- Number of Spaces = 1, Spacing = 9.17 ft
- Number of Spaces = 3, Spacing = 9.50 ft
- Number of Spaces = 6, Spacing = 9.67 ft
- Number of Spaces = 1, Spacing = 9.50 ft
- Bracing or Stiffener Code = SG for stiffener spacings in a girder
- Span Number = 1
- Bracing Point or Stiffener Code = L for longitudinally stiffened girder web
- Number of Spaces = 22, Spacing = 4.81 ft

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 41-3024-0020-0000 | | |
| Description: | EXT BEAM 2 W/ FB | | |
| Bridge Type: | GFF | | Girder-Floorbeam (No Stringer) |
| SLC Level: | | | |
| Lanes: | D | | D for Design Lane Placement |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | 3 | | Load Factor Ratings |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | Y | | |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | | | Leave blank and calculate DF |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | | | |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | 25.00 | ft | Leave blank for GGG |
| Overhang or Spacing: | 0.00 | ft | |
| CL of Girder: | 2.17 | ft | Leave blank for GGG |
| Roadway Width: | 23.00 | ft | Leave blank for GGG |
| Distr Factor -Shear: | | | |
| Distr Factor -Moment: | | | |
| Distr Factor -Deflect: | | | |
| Slab Thickness: | 8.50 | in | Per plans |
| Haunch: | 0.00 | in | |
| Bridge DL1: | 0.083 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.648 | kip/ft | See Hand Calcs |
| F'c: | 4.0 | ksi | Per plans |
| N: | 8 | | Class AAA per DM-4 8.2 |
| Symmetry: | | | |
| LL Location: | | | Only applies to truss |
| Number of Panels: | | | Only applies to truss |
| End Connections: | | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | | | Only applies to truss |
| Stringer DL: | | kip/ft | N/A for CTB |
| Floorbeam DL: | 0.071 | kip/ft | See Hand Calcs |
| Unit weight of Concrete: | | lb/cf | |
| Gusset Plate Analysis: | | | |
| Patch Load Analysis: | | | |
| Unsymmetrical Pier Support: | | | |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 106.00 | ft | Per plans |

**Stringer (Floorbeam) Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple span |
| Span Length 1: | 9.17 | ft | Per plans/See Hand Calcs |
| Span Length 2: | 9.50 | ft | Per plans/See Hand Calcs |
| Span Length 3: | 9.50 | ft | Per plans/See Hand Calcs |
| Span Length 4: | 9.50 | ft | Per plans/See Hand Calcs |
| Span Length 5: | 9.67 | ft | Per plans/See Hand Calcs |
| Span Length 6: | 9.67 | ft | Per plans/See Hand Calcs |
| Span Length 7: | 9.67 | ft | Per plans/See Hand Calcs |
| Span Length 8: | 9.67 | ft | Per plans/See Hand Calcs |
| Span Length 9: | 9.67 | ft | Per plans/See Hand Calcs |
| Span Length 10: | 9.67 | ft | Per plans/See Hand Calcs |
| Span Length 11: | 9.50 | ft | Per plans/See Hand Calcs |

**Concrete Member Properties**

| | | | |
|---|---|---|---|
| Type: | O | | O for Steel Bridge |
| Depth: | | in | |
| B: | | in | |
| D: | | in | |
| AS: | | $in^2$ | |
| $f_y$: | 60 | ksi | Per DM-4 8.2 |
| AV: | | $in^2$ | |
| Specs: | | | |
| Alpha: | | Deg | |
| Integral Wearing Surface: | | in | |

**Steel Member Data Tables**

| 12 | Steel Member Lines | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Record | Member Type | Span No. | Last? | Range | Section Type | MOI or VLeg | Area or HLeg | F Thickness | F Width | D Varies? | WF Beam Depth | W Thickness | Top Width | Top Thick. | Bot Width | Bot Thick. | Shored/Comp. | Fy |
| 1 | G | 1 | | 5 | B | 8 | 8 | 0.625 | | | 93 | 0.4375 | 18 | 0.5 | | | N | 33 |
| 2 | G | 1 | | 5.5 | B | 8 | 8 | 0.375 | | | 93 | 0.3125 | 18 | 0.5 | | | N | 33 |
| 3 | G | 1 | | 10 | B | 8 | 8 | 0.375 | | | 93 | 0.4375 | 18 | 0.5 | | | N | 33 |
| 4 | G | 1 | | 14 | B | 8 | 8 | 0.625 | | | 93 | 0.4375 | 18 | 0.5 | | | N | 33 |
| 5 | G | 1 | | 25 | B | 8 | 8 | 0.625 | | | 93 | 0.4375 | 18 | 0.5 | 18 | 0.5 | N | 33 |
| 6 | G | 1 | | 81 | B | 8 | 8 | 0.625 | | | 93 | 0.4375 | 18 | 1 | 18 | 1 | N | 33 |
| 7 | G | 1 | | 92 | B | 8 | 8 | 0.625 | | | 93 | 0.4375 | 18 | 0.5 | 18 | 0.5 | N | 33 |
| 8 | G | 1 | | 106 | B | 8 | 8 | 0.625 | | | 93 | 0.4375 | 18 | 0.5 | | | N | 33 |
| 9 | F | 4 | | 6.5 | W | 4919.1 | 34.13 | 0.85 | 10.5 | | 30 | 0.494 | | | | | Y | 33 |
| 10 | F | 4 | | 12 | W | 4919.1 | 34.13 | 0.85 | 10.5 | | 30 | 0.564 | | | | | Y | 33 |
| 11 | F | 4 | | 14 | W | 4919.1 | 34.13 | 0.85 | 10.5 | | 30 | 0.564 | | | | | Y | 33 |
| 12 | F | 4 | | 27 | W | 4919.1 | 34.13 | 0.85 | 10.5 | | 30 | 0.564 | | | | | Y | 33 |

**Lateral Bracing**

| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
|---|---|---|---|---|---|---|---|---|
| Bracing or Stiff Code | BF | BG | SG | | | | | |
| Span Number | 4 | 1 | 1 | | | | | |
| Code 1 | C | T | L | | | | | |
| Num of Spaces | 1 | 1 | 22 | | | | | |
| Spacing 1 | 27.00 | 9.17 | 4.81 | | | | | |
| Code 2 | | T | | | | | | |
| Num of Spaces | | 3 | | | | | | |
| Spacing 2 | | 9.50 | | | | | | |
| Code 3 | | T | | | | | | |
| Num of Spaces | | 6 | | | | | | |
| Spacing 3 | | 9.67 | | | | | | |
| Code 4 | | T | | | | | | |
| Num of Spaces | | 1 | | | | | | |
| Spacing 4 | | 9.50 | | | | | | |
| Code 5 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 5 | | | | | | | | |
| Code 6 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 6 | | | | | | | | |
| Code 7 | | | | | | | | |
| Num of Spaces 7 | | | | | | | | |
| Spacing 7 | | | | | | | | |
| Code 8 | | | | | | | | |
| Num of Spaces 8 | | | | | | | | |
| Spacing 8 | | | | | | | | |

3.9.9.2.2 BAR7 Output

```
******************************************************************************
*                                                                            *
*                   BRIDGE ANALYSIS AND RATING (BAR7)                330427 *
*                                                                            *
*                         COPYRIGHT (C) 1990-2018                            *
*                                                                            *
*                       COMMONWEALTH OF PENNSYLVANIA                         *
*                       DEPARTMENT OF TRANSPORTATION                         *
*                                                                            *
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                     BRIDGE ANALYSIS AND RATING (BAR7)                330427

PROGRAM P4353000                                                  04/11/2024 20:21
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: Steel Girder and Floorbeam BAR7 Analysis.dat




                             PROJECT IDENTIFICATION


STRUCTURE ID - 41302400200000 - EXT BEAM 2 W/ FLOORBEAM

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT   FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 GFF       D              0.00  0.0  0.0        Y                 0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     0.000        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
25.00    0.00       2.17       23.00   0.000  0.000  0.000

   SLAB               DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   8.50      0.00   0.083   0.648   4.000    8.

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.071        0.00


                       SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH    106.00


             TRAFFIC LANE LOCATIONS

LANE #    1       2       3       4       5       6       7
DIST
WIDTH
% LL


             STRINGER SPAN LENGTHS (SIMPLE)

SPAN #       1       2       3       4       5       6       7       8
LENGTH      9.17    9.50    9.50    9.50    9.67    9.67    9.67    9.67
```

```
SPAN #        9       10       11
LENGTH      9.67    9.67    9.50

    ***** INPUT WARNING  1 *****

    WARNING - SINCE THE BRIDGE TYPE IS SPECIFIED AS GFF OR TFF
    AND THE DISTRIBUTION FACTOR FOR MOMENT IS NOT ENTERED,
    THE WHEEL LOAD ACTING ON THE FLOORBEAM WILL BE CALCULATED
    ASSUMING THE FLOORING BETWEEN FLOORBEAMS TO ACT AS A
    SIMPLE SPAN BEAM.


            CONCRETE MEMBER PROPERTIES

                                                   FY
TYPE  DEPTH     B      D     AS      D'   A'S   REINF
 O     0.00   0.00  0.00  0.00  0.00  0.00    60.


     ALLOWABLE FS     ST                          INTEGRAL
       IR    OR       DET   AV   SPECS  ALPHA  WEARING SURFACE
       0.0   0.0           0.00    0      0.         0.0


                           STEEL MEMBER PROPERTIES

  S        T  WF BM    WF BM   FLANGE            WF BM
G P        Y  M OF I    AREA     OR          V  OR WEB
F A        P  OR VRT   OR HRZ  ANGLE  FLANGE  A   PLATE    WEB
S N  RANGE E   LEG      LEG    THICK   WIDTH  R   DEPTH   THICK  SHAPE
G 1   5.00 B     8.00    8.00  0.6250  0.000      93.00  0.4375
              TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 0.5000  0.00 0.0000   N  33.0   0.0    0.0   0.000  0.000

     RANGE    VT LEG   HZ LEG  THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1   5.50 B     8.00    8.00  0.3750  0.000      93.00  0.3125
              TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 0.5000  0.00 0.0000   N  33.0   0.0    0.0   0.000  0.000

     RANGE    VT LEG   HZ LEG  THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1  10.00 B     8.00    8.00  0.3750  0.000      93.00  0.4375
              TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 0.5000  0.00 0.0000   N  33.0   0.0    0.0   0.000  0.000

     RANGE    VT LEG   HZ LEG  THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1  14.00 B     8.00    8.00  0.6250  0.000      93.00  0.4375
              TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 0.5000  0.00 0.0000   N  33.0   0.0    0.0   0.000  0.000

     RANGE    VT LEG   HZ LEG  THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1  25.00 B     8.00    8.00  0.6250  0.000      93.00  0.4375
              TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 0.5000 18.00 0.5000   N  33.0   0.0    0.0   0.000  0.000

     RANGE    VT LEG   HZ LEG  THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1  81.00 B     8.00    8.00  0.6250  0.000      93.00  0.4375
              TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 1.0000 18.00 1.0000   N  33.0   0.0    0.0   0.000  0.000

     RANGE    VT LEG   HZ LEG  THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1  92.00 B     8.00    8.00  0.6250  0.000      93.00  0.4375
              TPW   TPT    BPW   BPT   COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 0.5000 18.00 0.5000   N  33.0   0.0    0.0   0.000  0.000
```

```
     RANGE    VT LEG   HZ LEG  THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1 106.00 B     8.00    8.00  0.6250  0.000       93.00  0.4375
              TPW    TPT    BPW    BPT    COMP  FY   FY TOP FY BOT CG TOP CG BOT
             18.00 0.5000  0.00 0.0000   N  33.0    0.0    0.0   0.000  0.000

     RANGE    M OF I    AREA   THICK   WIDTH  V   DEPTH   THICK  SHAPE
F 4   6.50 W  4919.10   34.13  0.8500 10.500       30.00  0.4940
              TPW    TPT    BPW    BPT    COMP  FY   FY TOP FY BOT CG TOP CG BOT
              0.00 0.0000  0.00 0.0000   Y  33.0    0.0    0.0   0.000  0.000

     RANGE    M OF I    AREA   THICK   WIDTH  V   DEPTH   THICK  SHAPE
F 4  27.00 W  4919.10   34.13  0.8500 10.500       30.00  0.5640
              TPW    TPT    BPW    BPT    COMP  FY   FY TOP FY BOT CG TOP CG BOT
              0.00 0.0000  0.00 0.0000   Y  33.0    0.0    0.0   0.000  0.000


               LATERAL BRACE POINTS AND STIFFENER SPACINGS

             C                C                C                C
B OR S       O  NO.           O  NO.           O  NO.           O  NO.
G OR F       D  OF            D  OF            D  OF            D  OF
 CODE  SPAN  E SPCS SPACING   E SPCS SPACING   E SPCS SPACING   E SPCS SPACING
  BF     4   C   1   24.00        0    0.00        0    0.00        0    0.00
                 0    0.00        0    0.00        0    0.00        0    0.00
  BG     1   T   1    9.17    T   3    9.50    T   6    9.67    T   1    9.50
                 0    0.00        0    0.00        0    0.00        0    0.00
  SG     1   L  22    4.81        0    0.00        0    0.00        0    0.00
                 0    0.00        0    0.00        0    0.00        0    0.00


                                      DEFAULT VALUES

                                        UNIT                                INTEGRAL
  SLC             GAGE     PASSING     WEIGHT    FY    ALLOWABLE FS     WEARING
 LEVEL LANES  DISTANCE  DISTANCE     DECK    REINF   IR      OR       SURFACE
   I    ---       6.0        4.0      150.0    ---   24.0    36.0        0.5


    THERE IS A TOTAL OF   1 INPUT WARNING IN THE PROGRAM.
    THE PROGRAM WLL NOT BE TERMINATED.
    PLEASE REVIEW YOUR INPUT WARNING AND RESUBMIT THE JOB IF NEEDED.


                ++++++++++++++++++++++++++++++++++++++++++++++
                +                                            +
                +       G I R D E R   A N A L Y S I S        +
                +                                            +
                ++++++++++++++++++++++++++++++++++++++++++++++


                        LIVE LOAD DISTRIBUTION FACTORS

                            BASED ON DESIGN LANES
                              GIRDER     GIRDER
                              MOMENT   DEFLECTION
                               1.055(2)  1.000(2)


                          DEAD LOADS ACTING ON GIRDER

 INPUT  GIRDER   SLAB   FL BEAM  STRINGER  FL BEAM STRINGER   TOTAL   TOTAL
```

| DL1 | WEIGHT | WEIGHT | WEIGHT | WEIGHT | DL1 | DL1 | DL1 | DL2 |
|---|---|---|---|---|---|---|---|---|
| 0.083 | 0.352 | 1.328 | 0.328 | 0.000 | 0.101 | 0.000 | 2.193 | 0.648 |

NOTE: IF THE LIVE LOAD STRESS IS ZERO AT ANY SECTION THE RATING FACTOR IS PRINTED AS 999.99 INDICATING THAT IT IS INFINITE.

NOTE: IF A SECTION DOES NOT MEET FLANGE OR WEB BUCKLING CRITERIA OF CURRENT AASHTO SPECIFICATIONS FOR LOAD FACTOR METHOD, THE RATING FACTORS ARE REPRINTED AS 888.88. THIS INDICATES THAT THERE IS A POTENTIAL FATIGUE PROBLEM.

GIRDER SECTION PROPERTIES (NON-COMPOSITE)

SPAN 1
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT |
|---|---|---|---|---|---|---|
| 0.00 | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |
| 5.00L | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |
| 5.00R | 93.50 | 61.50 | 84009.33 | 53.34 | 2091.94 | 1574.93 |
| 5.50L | 93.50 | 61.50 | 84009.33 | 53.34 | 2091.94 | 1574.93 |
| 5.50R | 93.50 | 73.12 | 92845.66 | 52.25 | 2251.01 | 1776.82 |
| 10.00L | 93.50 | 73.12 | 92845.66 | 52.25 | 2251.01 | 1776.82 |
| 10.00R | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |
| 10.60 | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |
| 14.00L | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |
| 14.00R | 94.00 | 97.12 | 144230.73 | 47.00 | 3068.74 | 3068.74 |
| 21.20 | 94.00 | 97.12 | 144230.73 | 47.00 | 3068.74 | 3068.74 |
| 25.00L | 94.00 | 97.12 | 144230.73 | 47.00 | 3068.74 | 3068.74 |
| 25.00R | 95.00 | 115.12 | 184417.23 | 47.50 | 3882.47 | 3882.47 |
| 31.80 | 95.00 | 115.12 | 184417.23 | 47.50 | 3882.47 | 3882.47 |
| 42.40 | 95.00 | 115.12 | 184417.23 | 47.50 | 3882.47 | 3882.47 |
| 53.00 | 95.00 | 115.12 | 184417.23 | 47.50 | 3882.47 | 3882.47 |
| 63.60 | 95.00 | 115.12 | 184417.23 | 47.50 | 3882.47 | 3882.47 |
| 74.20 | 95.00 | 115.12 | 184417.23 | 47.50 | 3882.47 | 3882.47 |
| 81.00L | 95.00 | 115.12 | 184417.23 | 47.50 | 3882.47 | 3882.47 |
| 81.00R | 94.00 | 97.12 | 144230.73 | 47.00 | 3068.74 | 3068.74 |
| 84.80 | 94.00 | 97.12 | 144230.73 | 47.00 | 3068.74 | 3068.74 |
| 92.00L | 94.00 | 97.12 | 144230.73 | 47.00 | 3068.74 | 3068.74 |
| 92.00R | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |
| 95.40 | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |
| 106.00 | 93.50 | 88.12 | 122551.62 | 51.27 | 2902.31 | 2390.11 |

DEFLECTIONS

SPAN 1 - LIVE LOAD IMPACT FACTOR FOR DEFLECTION: 1.22
=======

| | DEAD LOAD | | ---------------- LIVE LOAD + IMPACT ---------------- | | | |
|---|---|---|---|---|---|---|
| X | DL1 | DL2 | H20 | HS20 | TK527 | ML80 |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 10.60 | 0.411 | 0.121 | 0.205 | 0.228 | 0.260 | 0.244 |
| 21.20 | 0.763 | 0.225 | 0.382 | 0.427 | 0.486 | 0.457 |
| 31.80 | 1.023 | 0.302 | 0.513 | 0.579 | 0.658 | 0.619 |
| 42.40 | 1.184 | 0.350 | 0.596 | 0.674 | 0.766 | 0.722 |
| 53.00 | 1.238 | 0.366 | 0.625 | 0.705 | 0.801 | 0.757 |
| 63.60 | 1.183 | 0.350 | 0.595 | 0.674 | 0.765 | 0.722 |
| 74.20 | 1.022 | 0.302 | 0.512 | 0.578 | 0.658 | 0.618 |
| 84.80 | 0.761 | 0.225 | 0.381 | 0.427 | 0.485 | 0.456 |
| 95.40 | 0.409 | 0.121 | 0.204 | 0.227 | 0.259 | 0.243 |
| 106.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

```
******************************
*  GIRDER  -  LIVE LOAD  H20   *
******************************
```

MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 116.2 | 34.3 | 76.9 L | 0.0 | 1.22 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | IM | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 116.2 | 34.3 | 76.9L | 0.0 | 1.22 |
| | SIMULT SHEAR | | 0.0 | 0.0 | | SIMULT MOM | | 0.0 | 0.0 | |
| 5.00 | 553.7 | 163.6 | 317.4L | 0.0 | | 105.2 | 31.1 | 71.6L | -1.9 | 1.22 |
| | SIMULT SHEAR | | 61.7 | 0.0 | | SIMULT MOM | | 356.5 | 195.6 | |
| 5.50 | 606.0 | 179.1 | 347.4L | 0.0 | | 104.2 | 30.8 | 71.1L | -2.1 | 1.22 |
| | SIMULT SHEAR | | 61.2 | 0.0 | | SIMULT MOM | | 389.2 | 214.1 | |
| 10.00 | 1052.5 | 311.0 | 603.4L | 0.0 | | 94.3 | 27.9 | 66.4L | -3.9 | 1.23 |
| | SIMULT SHEAR | | 56.7 | 0.0 | | SIMULT MOM | | 659.2 | 371.9 | |
| 10.60 | 1108.7 | 327.6 | 635.6L | 0.0 | | 93.0 | 27.5 | 65.8L | -4.1 | 1.23 |
| | SIMULT SHEAR | | 56.1 | 0.0 | | SIMULT MOM | | 692.0 | 391.7 | |
| 14.00 | 1412.1 | 417.3 | 809.5L | 0.0 | | 85.5 | 25.3 | 62.5L | -5.5 | 1.23 |
| | SIMULT SHEAR | | 52.7 | 0.0 | | SIMULT MOM | | 864.4 | 498.9 | |
| 21.20 | 1971.0 | 582.5 | 1129.9L | 0.0 | | 69.7 | 20.6 | 55.5L | -9.1 | 1.24 |
| | SIMULT SHEAR | | 45.4 | 0.0 | | SIMULT MOM | | 1156.4 | 755.5 | |
| 25.00 | 2220.1 | 656.1 | 1272.8L | 0.0 | | 61.4 | 18.1 | 52.0L | -11.0 | 1.24 |
| | SIMULT SHEAR | | 41.5 | 0.0 | | SIMULT MOM | | 1272.8 | 870.7 | |
| 31.80 | 2586.9 | 764.5 | 1483.0L | 0.0 | | 46.5 | 13.7 | 46.0L | -14.4 | 1.25 |
| | SIMULT SHEAR | | 34.5 | 0.0 | | SIMULT MOM | | 1420.9 | 1042.0 | |
| 42.40 | 2956.5 | 873.7 | 1694.9L | 0.0 | | 23.2 | 6.9 | 37.1L | -21.1L | 1.27 |
| | SIMULT SHEAR | | 23.5 | 0.0 | | SIMULT MOM | | 1513.1 | 1291.7 | |
| 53.00 | 3079.6 | 910.1 | 1765.5L | 0.0 | | -0.0 | -0.0 | 29.0L | -29.0L | 1.28 |
| | SIMULT SHEAR | | -12.2 | 0.0 | | SIMULT MOM | | 1460.8 | 1460.8 | |
| 63.60 | 2956.5 | 873.7 | 1694.9L | 0.0 | | -23.2 | -6.9 | 21.1L | -37.1L | 1.27 |
| | SIMULT SHEAR | | -23.5 | 0.0 | | SIMULT MOM | | 1291.7 | 1513.1 | |
| 74.20 | 2586.9 | 764.5 | 1483.0L | 0.0 | | -46.5 | -13.7 | 14.4 | -46.0L | 1.25 |
| | SIMULT SHEAR | | -34.5 | 0.0 | | SIMULT MOM | | 1042.0 | 1420.9 | |
| 81.00 | 2220.1 | 656.1 | 1272.8L | 0.0 | | -61.4 | -18.1 | 11.0 | -52.0L | 1.24 |
| | SIMULT SHEAR | | -41.5 | 0.0 | | SIMULT MOM | | 870.7 | 1272.8 | |
| 84.80 | 1971.0 | 582.5 | 1129.9L | 0.0 | | -69.7 | -20.6 | 9.1 | -55.5L | 1.24 |
| | SIMULT SHEAR | | -45.4 | 0.0 | | SIMULT MOM | | 755.5 | 1156.4 | |
| 92.00 | 1412.1 | 417.3 | 809.5L | 0.0 | | -85.5 | -25.3 | 5.5 | -62.5L | 1.23 |
| | SIMULT SHEAR | | -52.7 | 0.0 | | SIMULT MOM | | 498.9 | 864.4 | |
| 95.40 | 1108.7 | 327.6 | 635.6L | 0.0 | | -93.0 | -27.5 | 4.1 | -65.8L | 1.23 |
| | SIMULT SHEAR | | -56.1 | 0.0 | | SIMULT MOM | | 391.7 | 692.0 | |
| 106.00 | 0.0 | 0.0 | 0.0 | 0.0 | | -116.2 | -34.3 | 0.0 | -76.9L | 1.22 |
| | SIMULT SHEAR | | 0.0 | 0.0 | | SIMULT MOM | | 0.0 | 0.0 | |

FLEXURAL STRESSES - BEAM

SPAN 1
=======

| X | TOP FIBER STEEL STRESS DL1 | DL2 | +(LL+I) | -(LL+I) | BOTTOM FIBER STEEL STRESS DL1 | DL2 | +(LL+I) | -(LL+I) |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 5.00 | -3.176 | -0.939 | -1.821 | 0.000 | 4.219 | 1.247 | 2.418 | 0.000 |
| 5.50 | -3.476 | -1.027 | -1.993 | 0.000 | 4.617 | 1.365 | 2.647 | 0.000 |
| 10.00 | -5.611 | -1.658 | -3.217 | 0.000 | 7.108 | 2.101 | 4.075 | 0.000 |
| 10.60 | -4.584 | -1.355 | -2.628 | 0.000 | 5.566 | 1.645 | 3.191 | 0.000 |
| 14.00 | -5.839 | -1.725 | -3.347 | 0.000 | 7.090 | 2.095 | 4.064 | 0.000 |
| 21.20 | -7.707 | -2.278 | -4.418 | 0.000 | 7.707 | 2.278 | 4.418 | 0.000 |
| 25.00 | -8.682 | -2.566 | -4.977 | 0.000 | 8.682 | 2.566 | 4.977 | 0.000 |
| 31.80 | -7.996 | -2.363 | -4.584 | 0.000 | 7.996 | 2.363 | 4.584 | 0.000 |
| 42.40 | -9.138 | -2.700 | -5.239 | 0.000 | 9.138 | 2.700 | 5.239 | 0.000 |
| 53.00 | -9.519 | -2.813 | -5.457 | 0.000 | 9.519 | 2.813 | 5.457 | 0.000 |
| 63.60 | -9.138 | -2.700 | -5.239 | 0.000 | 9.138 | 2.700 | 5.239 | 0.000 |
| 74.20 | -7.996 | -2.363 | -4.584 | 0.000 | 7.996 | 2.363 | 4.584 | 0.000 |
| 81.00 | -8.682 | -2.566 | -4.977 | 0.000 | 8.682 | 2.566 | 4.977 | 0.000 |
| 84.80 | -7.707 | -2.278 | -4.418 | 0.000 | 7.707 | 2.278 | 4.418 | 0.000 |
| 92.00 | -5.839 | -1.725 | -3.347 | 0.000 | 7.090 | 2.095 | 4.064 | 0.000 |
| 95.40 | -4.584 | -1.355 | -2.628 | 0.000 | 5.566 | 1.645 | 3.191 | 0.000 |
| 106.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 2.895 | 0.856 | 1.915 | 0.000 | 1.000 | 1.18 V | 2.28 V |
| 5.00 | 3.651 | 1.079 | 2.483 | -0.067 | 1.000 | 1.69 V | 2.94 V |
| 5.50 | 3.613 | 1.068 | 2.465 | -0.074 | 1.000 | 1.72 V | 2.98 V |
| 10.00 | 2.349 | 0.694 | 1.655 | -0.097 | 1.000 | 1.91 I | 3.32 I |
| 10.60 | 2.316 | 0.684 | 1.640 | -0.103 | 1.000 | 2.64 I | 4.31 I |
| 14.00 | 2.130 | 0.630 | 1.556 | -0.137 | 1.000 | 2.04 I | 3.49 I |
| 21.20 | 1.737 | 0.513 | 1.383 | -0.226 | 1.000 | 1.85 T | 3.34 T |
| 25.00 | 1.530 | 0.452 | 1.296 | -0.274 | 1.000 | 1.39 T | 2.71 T |
| 31.80 | 1.158 | 0.342 | 1.145 | -0.360 | 1.000 | 1.70 T | 3.14 T |
| 42.40 | 0.579 | 0.171 | 0.925 | -0.526 | 1.000 | 1.20 T | 2.46 T |
| 53.00 | -0.000 | -0.000 | 0.723 | -0.723 | 1.000 | 1.07 T | 2.28 T |
| 63.60 | -0.579 | -0.171 | 0.526 | -0.925 | 1.000 | 1.20 T | 2.46 T |
| 74.20 | -1.158 | -0.342 | 0.360 | -1.145 | 1.000 | 1.70 T | 3.14 T |
| 81.00 | -1.530 | -0.452 | 0.274 | -1.296 | 1.000 | 1.39 T | 2.71 T |
| 84.80 | -1.737 | -0.513 | 0.226 | -1.383 | 1.000 | 1.85 T | 3.34 T |
| 92.00 | -2.130 | -0.630 | 0.137 | -1.556 | 1.000 | 2.04 I | 3.49 I |
| 95.40 | -2.316 | -0.684 | 0.103 | -1.640 | 1.000 | 2.64 I | 4.31 I |
| 106.00 | -2.895 | -0.856 | 0.000 | -1.915 | 1.000 | 1.18 V | 2.28 V |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 6572.8 B | 5258.2 | 420.1 | 1.35 V | 2.24 V | | | |
| 5.00 | 4331.1 U | 3464.9 | 447.2 | 1.74 V | 2.90 V | | | |
| 5.50 | 4331.1 U | 3464.9 | 447.2 | 1.77 V | 2.94 V | | | |
| 10.00 | 4886.3 U | 3909.0 | 676.5 | 2.04 I | 3.40 I | | | |
| 10.60 | 6572.8 B | 5258.2 | 676.5 | 2.63 I | 4.38 I | | | |
| 14.00 | 6572.8 B | 5258.2 | 676.5 | 2.15 I | 3.58 I | | | |

| | | | | | |
|---|---|---|---|---|---|
| 21.20 | 8439.0 B | 6751.2 | 676.5 | 2.09 T | 3.49 T |
| 25.00 | 8439.0 B | 6751.2 | 676.5 | 1.70 T | 2.84 T |
| 31.80 | 10676.8 B | 8541.4 | 676.5 | 1.97 T | 3.28 T |
| 42.40 | 10676.8 B | 8541.4 | 676.5 | 1.55 T | 2.59 T |
| 53.00 | 10676.8 B | 8541.4 | 676.5 | 1.44 T | 2.39 T |
| 63.60 | 10676.8 B | 8541.4 | 676.5 | 1.55 T | 2.59 T |
| 74.20 | 10676.8 B | 8541.4 | 676.5 | 1.97 T | 3.28 T |
| 81.00 | 8439.0 B | 6751.2 | 676.5 | 1.70 T | 2.84 T |
| 84.80 | 8439.0 B | 6751.2 | 676.5 | 2.09 T | 3.49 T |
| 92.00 | 6572.8 B | 5258.2 | 676.5 | 2.15 I | 3.58 I |
| 95.40 | 6572.8 B | 5258.2 | 676.5 | 2.63 I | 4.38 I |
| 106.00 | 6572.8 B | 5258.2 | 420.1 | 1.35 V | 2.24 V |

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********************************
*  GIRDER  -  LIVE LOAD  HS20  *
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MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 116.2 | 34.3 | 84.3 | 0.0 | 1.22 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 116.2 | 34.3 | 84.3 | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 5.00 | 553.7 | 163.6 | 399.5 | 0.0 | | 105.2 | 31.1 | 80.2 | -1.9 | 1.22 |
| | SIMULT | SHEAR | 80.2 | 0.0 | | SIMULT | MOM | 399.5 | 195.6 | |
| 5.50 | 606.0 | 179.1 | 437.0 | 0.0 | | 104.2 | 30.8 | 79.8 | -2.1 | 1.22 |
| | SIMULT | SHEAR | 79.8 | 0.0 | | SIMULT | MOM | 437.0 | 214.1 | |
| 10.00 | 1052.5 | 311.0 | 755.4 | 0.0 | | 94.3 | 27.9 | 76.1 | -3.9 | 1.23 |
| | SIMULT | SHEAR | 76.1 | 0.0 | | SIMULT | MOM | 755.4 | 371.9 | |
| 10.60 | 1108.7 | 327.6 | 795.2 | 0.0 | | 93.0 | 27.5 | 75.7 | -4.1 | 1.23 |
| | SIMULT | SHEAR | 75.7 | 0.0 | | SIMULT | MOM | 795.2 | 391.7 | |
| 14.00 | 1412.1 | 417.3 | 1008.7 | 0.0 | | 85.5 | 25.3 | 72.9 | -5.5 | 1.23 |
| | SIMULT | SHEAR | 72.9 | 0.0 | | SIMULT | MOM | 1008.7 | 498.9 | |
| 21.20 | 1971.0 | 582.5 | 1394.5 | 0.0 | | 69.7 | 20.6 | 67.0 | -11.2 | 1.24 |
| | SIMULT | SHEAR | 67.0 | 0.0 | | SIMULT | MOM | 1394.5 | 932.9 | |
| 25.00 | 2220.1 | 656.1 | 1561.6 | 0.0 | | 61.4 | 18.1 | 63.8 | -14.2 | 1.24 |
| | SIMULT | SHEAR | 63.8 | 0.0 | | SIMULT | MOM | 1561.6 | 1129.6 | |
| 31.80 | 2586.9 | 764.5 | 1797.9 | 0.0 | | 46.5 | 13.7 | 58.1 | -20.1 | 1.25 |
| | SIMULT | SHEAR | 58.1 | 0.0 | | SIMULT | MOM | 1797.9 | 1453.0 | |
| 42.40 | 2956.5 | 873.7 | 2034.2 | 0.0 | | 23.2 | 6.9 | 49.2 | -30.0 | 1.27 |
| | SIMULT | SHEAR | 42.7 | 0.0 | | SIMULT | MOM | 2005.5 | 1833.0 | |
| 53.00 | 3079.6 | 910.1 | 2089.0 | 0.0 | | -0.0 | -0.0 | 40.1 | -40.1 | 1.28 |
| | SIMULT | SHEAR | -33.5 | 0.0 | | SIMULT | MOM | 2017.2 | 2017.2 | |
| 63.60 | 2956.5 | 873.7 | 2034.2 | 0.0 | | -23.2 | -6.9 | 30.0 | -49.2 | 1.27 |
| | SIMULT | SHEAR | -42.7 | 0.0 | | SIMULT | MOM | 1833.0 | 2005.5 | |
| 74.20 | 2586.9 | 764.5 | 1797.9 | 0.0 | | -46.5 | -13.7 | 20.1 | -58.1 | 1.25 |
| | SIMULT | SHEAR | -58.1 | 0.0 | | SIMULT | MOM | 1453.0 | 1797.9 | |
| 81.00 | 2220.1 | 656.1 | 1561.6 | 0.0 | | -61.4 | -18.1 | 14.2 | -63.8 | 1.24 |
| | SIMULT | SHEAR | -63.8 | 0.0 | | SIMULT | MOM | 1129.6 | 1561.6 | |
| 84.80 | 1971.0 | 582.5 | 1394.5 | 0.0 | | -69.7 | -20.6 | 11.2 | -67.0 | 1.24 |
| | SIMULT | SHEAR | -67.0 | 0.0 | | SIMULT | MOM | 932.9 | 1394.5 | |
| 92.00 | 1412.1 | 417.3 | 1008.7 | 0.0 | | -85.5 | -25.3 | 5.5 | -72.9 | 1.23 |
| | SIMULT | SHEAR | -72.9 | 0.0 | | SIMULT | MOM | 498.9 | 1008.7 | |

```
   95.40  1108.7   327.6  795.2      0.0      -93.0  -27.5    4.1   -75.7  1.23
            SIMULT SHEAR  -75.7      0.0       SIMULT MOM   391.7   795.2
  106.00     0.0     0.0    0.0      0.0     -116.2  -34.3    0.0   -84.3  1.22
            SIMULT SHEAR    0.0      0.0       SIMULT MOM     0.0     0.0
```

FLEXURAL STRESSES - BEAM

SPAN 1
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 5.00 | -3.176 | -0.939 | -2.292 | 0.000 | 4.219 | 1.247 | 3.044 | 0.000 |
| 5.50 | -3.476 | -1.027 | -2.507 | 0.000 | 4.617 | 1.365 | 3.330 | 0.000 |
| 10.00 | -5.611 | -1.658 | -4.027 | 0.000 | 7.108 | 2.101 | 5.102 | 0.000 |
| 10.60 | -4.584 | -1.355 | -3.288 | 0.000 | 5.566 | 1.645 | 3.992 | 0.000 |
| 14.00 | -5.839 | -1.725 | -4.171 | 0.000 | 7.090 | 2.095 | 5.065 | 0.000 |
| 21.20 | -7.707 | -2.278 | -5.453 | 0.000 | 7.707 | 2.278 | 5.453 | 0.000 |
| 25.00 | -8.682 | -2.566 | -6.107 | 0.000 | 8.682 | 2.566 | 6.107 | 0.000 |
| 31.80 | -7.996 | -2.363 | -5.557 | 0.000 | 7.996 | 2.363 | 5.557 | 0.000 |
| 42.40 | -9.138 | -2.700 | -6.287 | 0.000 | 9.138 | 2.700 | 6.287 | 0.000 |
| 53.00 | -9.519 | -2.813 | -6.457 | 0.000 | 9.519 | 2.813 | 6.457 | 0.000 |
| 63.60 | -9.138 | -2.700 | -6.287 | 0.000 | 9.138 | 2.700 | 6.287 | 0.000 |
| 74.20 | -7.996 | -2.363 | -5.557 | 0.000 | 7.996 | 2.363 | 5.557 | 0.000 |
| 81.00 | -8.682 | -2.566 | -6.107 | 0.000 | 8.682 | 2.566 | 6.107 | 0.000 |
| 84.80 | -7.707 | -2.278 | -5.453 | 0.000 | 7.707 | 2.278 | 5.453 | 0.000 |
| 92.00 | -5.839 | -1.725 | -4.171 | 0.000 | 7.090 | 2.095 | 5.065 | 0.000 |
| 95.40 | -4.584 | -1.355 | -3.288 | 0.000 | 5.566 | 1.645 | 3.992 | 0.000 |
| 106.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 2.895 | 0.856 | 2.099 | 0.000 | 1.000 | 1.08 V | 2.08 V |
| 5.00 | 3.651 | 1.079 | 2.782 | -0.067 | 1.000 | 1.50 V | 2.63 V |
| 5.50 | 3.613 | 1.068 | 2.768 | -0.074 | 1.000 | 1.53 V | 2.66 V |
| 10.00 | 2.349 | 0.694 | 1.897 | -0.097 | 1.000 | 1.67 I | 2.90 I |
| 10.60 | 2.316 | 0.684 | 1.885 | -0.103 | 1.000 | 2.30 I | 3.75 I |
| 14.00 | 2.130 | 0.630 | 1.816 | -0.137 | 1.000 | 1.74 I | 2.99 I |
| 21.20 | 1.737 | 0.513 | 1.668 | -0.279 | 1.000 | 1.50 T | 2.71 T |
| 25.00 | 1.530 | 0.452 | 1.590 | -0.355 | 1.000 | 1.13 T | 2.21 T |
| 31.80 | 1.158 | 0.342 | 1.449 | -0.502 | 1.000 | 1.40 T | 2.59 T |
| 42.40 | 0.579 | 0.171 | 1.225 | -0.747 | 1.000 | 1.00 T | 2.05 T |
| 53.00 | -0.000 | -0.000 | 0.998 | -0.998 | 1.000 | 0.90 T | 1.92 T |
| 63.60 | -0.579 | -0.171 | 0.747 | -1.225 | 1.000 | 1.00 T | 2.05 T |
| 74.20 | -1.158 | -0.342 | 0.502 | -1.449 | 1.000 | 1.40 T | 2.59 T |
| 81.00 | -1.530 | -0.452 | 0.355 | -1.590 | 1.000 | 1.13 T | 2.21 T |
| 84.80 | -1.737 | -0.513 | 0.279 | -1.668 | 1.000 | 1.50 T | 2.71 T |
| 92.00 | -2.130 | -0.630 | 0.137 | -1.816 | 1.000 | 1.74 I | 2.99 I |
| 95.40 | -2.316 | -0.684 | 0.103 | -1.885 | 1.000 | 2.30 I | 3.75 I |
| 106.00 | -2.895 | -0.856 | 0.000 | -2.099 | 1.000 | 1.08 V | 2.08 V |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 6572.8 B | 5258.2 | 420.1 | 1.23 V | 2.05 V | | | |
| 5.00 | 4331.1 U | 3464.9 | 447.2 | 1.55 V | 2.59 V | | | |
| 5.50 | 4331.1 U | 3464.9 | 447.2 | 1.57 V | 2.62 V | | | |
| 10.00 | 4886.3 U | 3909.0 | 676.5 | 1.78 I | 2.97 I | | | |
| 10.60 | 6572.8 B | 5258.2 | 676.5 | 2.29 I | 3.81 I | | | |
| 14.00 | 6572.8 B | 5258.2 | 676.5 | 1.84 I | 3.07 I | | | |
| 21.20 | 8439.0 B | 6751.2 | 676.5 | 1.69 T | 2.82 T | | | |
| 25.00 | 8439.0 B | 6751.2 | 676.5 | 1.39 T | 2.32 T | | | |
| 31.80 | 10676.8 B | 8541.4 | 676.5 | 1.62 T | 2.70 T | | | |
| 42.40 | 10676.8 B | 8541.4 | 676.5 | 1.29 T | 2.15 T | | | |
| 53.00 | 10676.8 B | 8541.4 | 676.5 | 1.21 T | 2.02 T | | | |
| 63.60 | 10676.8 B | 8541.4 | 676.5 | 1.29 T | 2.15 T | | | |
| 74.20 | 10676.8 B | 8541.4 | 676.5 | 1.62 T | 2.70 T | | | |
| 81.00 | 8439.0 B | 6751.2 | 676.5 | 1.39 T | 2.32 T | | | |
| 84.80 | 8439.0 B | 6751.2 | 676.5 | 1.69 T | 2.82 T | | | |
| 92.00 | 6572.8 B | 5258.2 | 676.5 | 1.84 I | 3.07 I | | | |
| 95.40 | 6572.8 B | 5258.2 | 676.5 | 2.29 I | 3.81 I | | | |
| 106.00 | 6572.8 B | 5258.2 | 420.1 | 1.23 V | 2.05 V | | | |

********************************
* GIRDER - LIVE LOAD TK527 *
********************************

MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 116.2 | 34.3 | 95.8 | 0.0 | 1.22 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 116.2 | 34.3 | 95.8 | 0.0 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 5.00 | 553.7 | 163.6 | 453.9 | 0.0 | | 105.2 | 31.1 | 91.1 | -1.5 | 1.22 |
| | SIMULT | SHEAR | 91.1 | 0.0 | | SIMULT | MOM | 453.9 | 151.1 | |
| 5.50 | 606.0 | 179.1 | 496.5 | 0.0 | | 104.2 | 30.8 | 90.7 | -1.8 | 1.22 |
| | SIMULT | SHEAR | 90.7 | 0.0 | | SIMULT | MOM | 496.5 | 175.4 | |
| 10.00 | 1052.5 | 311.0 | 857.8 | 0.0 | | 94.3 | 27.9 | 86.5 | -4.2 | 1.23 |
| | SIMULT | SHEAR | 86.5 | 0.0 | | SIMULT | MOM | 857.8 | 402.2 | |
| 10.60 | 1108.7 | 327.6 | 903.0 | 0.0 | | 93.0 | 27.5 | 85.9 | -4.6 | 1.23 |
| | SIMULT | SHEAR | 85.9 | 0.0 | | SIMULT | MOM | 903.0 | 433.9 | |
| 14.00 | 1412.1 | 417.3 | 1145.1 | 0.0 | | 85.5 | 25.3 | 82.7 | -6.9 | 1.23 |
| | SIMULT | SHEAR | 82.7 | 0.0 | | SIMULT | MOM | 1145.1 | 624.0 | |
| 21.20 | 1971.0 | 582.5 | 1581.8 | 0.0 | | 69.7 | 20.6 | 76.0 | -12.7 | 1.24 |
| | SIMULT | SHEAR | 76.0 | 0.0 | | SIMULT | MOM | 1581.8 | 1055.7 | |
| 25.00 | 2220.1 | 656.1 | 1770.6 | 0.0 | | 61.4 | 18.1 | 72.4 | -16.2 | 1.24 |
| | SIMULT | SHEAR | 72.4 | 0.0 | | SIMULT | MOM | 1770.6 | 1284.7 | |
| 31.80 | 2586.9 | 764.5 | 2057.6 | 0.0 | | 46.5 | 13.7 | 65.9 | -22.6 | 1.25 |
| | SIMULT | SHEAR | 42.8 | 0.0 | | SIMULT | MOM | 2036.5 | 1629.8 | |
| 42.40 | 2956.5 | 873.7 | 2330.4 | 0.0 | | 23.2 | 6.9 | 55.6 | -33.6 | 1.27 |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | SIMULT | SHEAR | 32.3 | 0.0 | SIMULT | MOM | 2267.0 | 2055.5 | |
| 53.00 | 3079.6 | 910.1 | 2379.0 | 0.0 | -0.0 | -0.0 | 45.2 | -45.2 | 1.28 |
| | SIMULT | SHEAR | -13.2 | 0.0 | SIMULT | MOM | 2273.3 | 2273.3 | |
| 63.60 | 2956.5 | 873.7 | 2330.4 | 0.0 | -23.2 | -6.9 | 33.6 | -55.6 | 1.27 |
| | SIMULT | SHEAR | -32.3 | 0.0 | SIMULT | MOM | 2055.5 | 2267.0 | |
| 74.20 | 2586.9 | 764.5 | 2057.6 | 0.0 | -46.5 | -13.7 | 22.6 | -65.9 | 1.25 |
| | SIMULT | SHEAR | -42.8 | 0.0 | SIMULT | MOM | 1629.8 | 2036.5 | |
| 81.00 | 2220.1 | 656.1 | 1770.6 | 0.0 | -61.4 | -18.1 | 16.2 | -72.4 | 1.24 |
| | SIMULT | SHEAR | -72.4 | 0.0 | SIMULT | MOM | 1284.7 | 1770.6 | |
| 84.80 | 1971.0 | 582.5 | 1581.8 | 0.0 | -69.7 | -20.6 | 12.7 | -76.0 | 1.24 |
| | SIMULT | SHEAR | -76.0 | 0.0 | SIMULT | MOM | 1055.7 | 1581.8 | |
| 92.00 | 1412.1 | 417.3 | 1145.1 | 0.0 | -85.5 | -25.3 | 6.9 | -82.7 | 1.23 |
| | SIMULT | SHEAR | -82.7 | 0.0 | SIMULT | MOM | 624.0 | 1145.1 | |
| 95.40 | 1108.7 | 327.6 | 903.0 | 0.0 | -93.0 | -27.5 | 4.6 | -85.9 | 1.23 |
| | SIMULT | SHEAR | -85.9 | 0.0 | SIMULT | MOM | 433.9 | 903.0 | |
| 106.00 | 0.0 | 0.0 | 0.0 | 0.0 | -116.2 | -34.3 | 0.0 | -95.8 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | SIMULT | MOM | 0.0 | 0.0 | |

FLEXURAL STRESSES - BEAM

SPAN 1
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 5.00 | -3.176 | -0.939 | -2.603 | 0.000 | 4.219 | 1.247 | 3.458 | 0.000 |
| 5.50 | -3.476 | -1.027 | -2.848 | 0.000 | 4.617 | 1.365 | 3.783 | 0.000 |
| 10.00 | -5.611 | -1.658 | -4.573 | 0.000 | 7.108 | 2.101 | 5.794 | 0.000 |
| 10.60 | -4.584 | -1.355 | -3.733 | 0.000 | 5.566 | 1.645 | 4.534 | 0.000 |
| 14.00 | -5.839 | -1.725 | -4.735 | 0.000 | 7.090 | 2.095 | 5.749 | 0.000 |
| 21.20 | -7.707 | -2.278 | -6.185 | 0.000 | 7.707 | 2.278 | 6.185 | 0.000 |
| 25.00 | -8.682 | -2.566 | -6.924 | 0.000 | 8.682 | 2.566 | 6.924 | 0.000 |
| 31.80 | -7.996 | -2.363 | -6.360 | 0.000 | 7.996 | 2.363 | 6.360 | 0.000 |
| 42.40 | -9.138 | -2.700 | -7.203 | 0.000 | 9.138 | 2.700 | 7.203 | 0.000 |
| 53.00 | -9.519 | -2.813 | -7.353 | 0.000 | 9.519 | 2.813 | 7.353 | 0.000 |
| 63.60 | -9.138 | -2.700 | -7.203 | 0.000 | 9.138 | 2.700 | 7.203 | 0.000 |
| 74.20 | -7.996 | -2.363 | -6.360 | 0.000 | 7.996 | 2.363 | 6.360 | 0.000 |
| 81.00 | -8.682 | -2.566 | -6.924 | 0.000 | 8.682 | 2.566 | 6.924 | 0.000 |
| 84.80 | -7.707 | -2.278 | -6.185 | 0.000 | 7.707 | 2.278 | 6.185 | 0.000 |
| 92.00 | -5.839 | -1.725 | -4.735 | 0.000 | 7.090 | 2.095 | 5.749 | 0.000 |
| 95.40 | -4.584 | -1.355 | -3.733 | 0.000 | 5.566 | 1.645 | 4.534 | 0.000 |
| 106.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 2.895 | 0.856 | 2.386 | 0.000 | 1.000 | 0.95 V | 1.83 V |
| 5.00 | 3.651 | 1.079 | 3.161 | -0.052 | 1.000 | 1.32 V | 2.31 V |
| 5.50 | 3.613 | 1.068 | 3.145 | -0.061 | 1.000 | 1.35 V | 2.34 V |
| 10.00 | 2.349 | 0.694 | 2.154 | -0.105 | 1.000 | 1.47 I | 2.55 I |
| 10.60 | 2.316 | 0.684 | 2.140 | -0.114 | 1.000 | 2.02 I | 3.30 I |
| 14.00 | 2.130 | 0.630 | 2.061 | -0.171 | 1.000 | 1.54 I | 2.64 I |
| 21.20 | 1.737 | 0.513 | 1.892 | -0.316 | 1.000 | 1.32 T | 2.39 T |
| 25.00 | 1.530 | 0.452 | 1.802 | -0.404 | 1.000 | 1.00 T | 1.95 T |
| 31.80 | 1.158 | 0.342 | 1.641 | -0.563 | 1.000 | 1.23 T | 2.26 T |
| 42.40 | 0.579 | 0.171 | 1.385 | -0.837 | 1.000 | 0.88 T | 1.79 T |
| 53.00 | -0.000 | -0.000 | 1.125 | -1.125 | 1.000 | 0.79 T | 1.69 T |
| 63.60 | -0.579 | -0.171 | 0.837 | -1.385 | 1.000 | 0.88 T | 1.79 T |

```
 74.20  -1.158  -0.342   0.563  -1.641      1.000     1.23 T    2.26 T
 81.00  -1.530  -0.452   0.404  -1.802      1.000     1.00 T    1.95 T
 84.80  -1.737  -0.513   0.316  -1.892      1.000     1.32 T    2.39 T
 92.00  -2.130  -0.630   0.171  -2.061      1.000     1.54 I    2.64 I
 95.40  -2.316  -0.684   0.114  -2.140      1.000     2.02 I    3.30 I
106.00  -2.895  -0.856   0.000  -2.386      1.000     0.95 V    1.83 V


NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR
      UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER
      SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.


                     STRENGTHS AND LOAD FACTOR RATINGS

SPAN  1
=======
        NON-COMP OVERLOAD          NON-COMPACT     COMPACT      COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS   MOMENT   RATING FACTORS
   X    STRENGTH STRENGTH STRENGTH   IR       OR    STRENGTH    IR       OR
  0.00   6572.8 B  5258.2   420.1   1.08 V   1.80 V
  5.00   4331.1 U  3464.9   447.2   1.37 V   2.28 V
  5.50   4331.1 U  3464.9   447.2   1.38 V   2.31 V
 10.00   4886.3 U  3909.0   676.5   1.57 I   2.61 I
 10.60   6572.8 B  5258.2   676.5   2.01 I   3.36 I
 14.00   6572.8 B  5258.2   676.5   1.62 I   2.70 I
 21.20   8439.0 B  6751.2   676.5   1.49 T   2.49 T
 25.00   8439.0 B  6751.2   676.5   1.23 T   2.04 T
 31.80  10676.8 B  8541.4   676.5   1.42 T   2.36 T
 42.40  10676.8 B  8541.4   676.5   1.13 T   1.88 T
 53.00  10676.8 B  8541.4   676.5   1.07 T   1.78 T
 63.60  10676.8 B  8541.4   676.5   1.13 T   1.88 T
 74.20  10676.8 B  8541.4   676.5   1.42 T   2.36 T
 81.00   8439.0 B  6751.2   676.5   1.23 T   2.04 T
 84.80   8439.0 B  6751.2   676.5   1.49 T   2.49 T
 92.00   6572.8 B  5258.2   676.5   1.62 I   2.70 I
 95.40   6572.8 B  5258.2   676.5   2.01 I   3.36 I
106.00   6572.8 B  5258.2   420.1   1.08 V   1.80 V


                       ********************************
                       *  GIRDER  -  LIVE LOAD  ML80  *
                       ********************************

                                  MAXIMUM REACTIONS

                                                        REACTIONS     MOMENTS
    SUPPORT    DL1       DL2    +(LL+I)     -(LL+I)   +I.F. -I.F.  +I.F. -I.F.
       1     116.2      34.3     90.9         0.0     1.22


                         UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.22
=======
            DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR   SHEAR  SHEAR   SHEAR  I.F.
    0.00     0.0     0.0    0.0      0.0      116.2    34.3   90.9      0.0  1.22
            SIMULT SHEAR    0.0      0.0       SIMULT MOM      0.0      0.0
    5.00   553.7   163.6  431.6      0.0      105.2    31.1   86.7     -1.5  1.22
            SIMULT SHEAR   86.7      0.0       SIMULT MOM    431.6    151.1
    5.50   606.0   179.1  472.2      0.0      104.2    30.8   86.2     -1.8  1.22
            SIMULT SHEAR   86.2      0.0       SIMULT MOM    472.2    175.4
```

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 10.00 | 1052.5 | 311.0 | 817.4 | 0.0 | 94.3 | 27.9 | 82.4 | -4.5 | 1.23 |
| | SIMULT | SHEAR | 82.4 | 0.0 | SIMULT | MOM | 817.4 | 430.9 | |
| 10.60 | 1108.7 | 327.6 | 860.7 | 0.0 | 93.0 | 27.5 | 81.9 | -5.0 | 1.23 |
| | SIMULT | SHEAR | 81.9 | 0.0 | SIMULT | MOM | 860.7 | 471.0 | |
| 14.00 | 1412.1 | 417.3 | 1093.3 | 0.0 | 85.5 | 25.3 | 79.0 | -7.6 | 1.23 |
| | SIMULT | SHEAR | 79.0 | 0.0 | SIMULT | MOM | 1093.3 | 688.3 | |
| 21.20 | 1971.0 | 582.5 | 1516.0 | 0.0 | 69.7 | 20.6 | 72.8 | -13.6 | 1.24 |
| | SIMULT | SHEAR | 72.8 | 0.0 | SIMULT | MOM | 1516.0 | 1136.1 | |
| 25.00 | 2220.1 | 656.1 | 1701.0 | 0.0 | 61.4 | 18.1 | 69.5 | -17.2 | 1.24 |
| | SIMULT | SHEAR | 69.5 | 0.0 | SIMULT | MOM | 1701.0 | 1366.5 | |
| 31.80 | 2586.9 | 764.5 | 1976.5 | 0.0 | 46.5 | 13.7 | 63.6 | -23.7 | 1.25 |
| | SIMULT | SHEAR | 40.2 | 0.0 | SIMULT | MOM | 1966.0 | 1712.8 | |
| 42.40 | 2956.5 | 873.7 | 2260.0 | 0.0 | 23.2 | 6.9 | 54.2 | -34.1 | 1.27 |
| | SIMULT | SHEAR | 30.5 | 0.0 | SIMULT | MOM | 2210.7 | 2084.1 | |
| 53.00 | 3079.6 | 910.1 | 2338.0 | 0.0 | -0.0 | -0.0 | 44.7 | -44.7 | 1.28 |
| | SIMULT | SHEAR | 20.7 | 0.0 | SIMULT | MOM | 2250.1 | 2250.1 | |
| 63.60 | 2956.5 | 873.7 | 2260.0 | 0.0 | -23.2 | -6.9 | 34.1 | -54.2 | 1.27 |
| | SIMULT | SHEAR | -30.5 | 0.0 | SIMULT | MOM | 2084.1 | 2210.7 | |
| 74.20 | 2586.9 | 764.5 | 1976.5 | 0.0 | -46.5 | -13.7 | 23.7 | -63.6 | 1.25 |
| | SIMULT | SHEAR | -40.2 | 0.0 | SIMULT | MOM | 1712.8 | 1966.0 | |
| 81.00 | 2220.1 | 656.1 | 1701.0 | 0.0 | -61.4 | -18.1 | 17.2 | -69.5 | 1.24 |
| | SIMULT | SHEAR | -69.5 | 0.0 | SIMULT | MOM | 1366.5 | 1701.0 | |
| 84.80 | 1971.0 | 582.5 | 1516.0 | 0.0 | -69.7 | -20.6 | 13.6 | -72.8 | 1.24 |
| | SIMULT | SHEAR | -72.8 | 0.0 | SIMULT | MOM | 1136.1 | 1516.0 | |
| 92.00 | 1412.1 | 417.3 | 1093.3 | 0.0 | -85.5 | -25.3 | 7.6 | -79.0 | 1.23 |
| | SIMULT | SHEAR | -79.0 | 0.0 | SIMULT | MOM | 688.3 | 1093.3 | |
| 95.40 | 1108.7 | 327.6 | 860.7 | 0.0 | -93.0 | -27.5 | 5.0 | -81.9 | 1.23 |
| | SIMULT | SHEAR | -81.9 | 0.0 | SIMULT | MOM | 471.0 | 860.7 | |
| 106.00 | 0.0 | 0.0 | 0.0 | 0.0 | -116.2 | -34.3 | 0.0 | -90.9 | 1.22 |
| | SIMULT | SHEAR | 0.0 | 0.0 | SIMULT | MOM | 0.0 | 0.0 | |

FLEXURAL STRESSES - BEAM

SPAN 1
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 5.00 | -3.176 | -0.939 | -2.476 | 0.000 | 4.219 | 1.247 | 3.288 | 0.000 |
| 5.50 | -3.476 | -1.027 | -2.709 | 0.000 | 4.617 | 1.365 | 3.598 | 0.000 |
| 10.00 | -5.611 | -1.658 | -4.358 | 0.000 | 7.108 | 2.101 | 5.521 | 0.000 |
| 10.60 | -4.584 | -1.355 | -3.559 | 0.000 | 5.566 | 1.645 | 4.321 | 0.000 |
| 14.00 | -5.839 | -1.725 | -4.520 | 0.000 | 7.090 | 2.095 | 5.489 | 0.000 |
| 21.20 | -7.707 | -2.278 | -5.928 | 0.000 | 7.707 | 2.278 | 5.928 | 0.000 |
| 25.00 | -8.682 | -2.566 | -6.651 | 0.000 | 8.682 | 2.566 | 6.651 | 0.000 |
| 31.80 | -7.996 | -2.363 | -6.109 | 0.000 | 7.996 | 2.363 | 6.109 | 0.000 |
| 42.40 | -9.138 | -2.700 | -6.985 | 0.000 | 9.138 | 2.700 | 6.985 | 0.000 |
| 53.00 | -9.519 | -2.813 | -7.226 | 0.000 | 9.519 | 2.813 | 7.226 | 0.000 |
| 63.60 | -9.138 | -2.700 | -6.985 | 0.000 | 9.138 | 2.700 | 6.985 | 0.000 |
| 74.20 | -7.996 | -2.363 | -6.109 | 0.000 | 7.996 | 2.363 | 6.109 | 0.000 |
| 81.00 | -8.682 | -2.566 | -6.651 | 0.000 | 8.682 | 2.566 | 6.651 | 0.000 |
| 84.80 | -7.707 | -2.278 | -5.928 | 0.000 | 7.707 | 2.278 | 5.928 | 0.000 |
| 92.00 | -5.839 | -1.725 | -4.520 | 0.000 | 7.090 | 2.095 | 5.489 | 0.000 |
| 95.40 | -4.584 | -1.355 | -3.559 | 0.000 | 5.566 | 1.645 | 4.321 | 0.000 |
| 106.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

SHEAR STRESSES ALLOW COMPR RATING FACTORS

| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
|---|---|---|---|---|---|---|---|
| 0.00 | 2.895 | 0.856 | 2.264 | 0.000 | 1.000 | 1.00 V | 1.93 V |
| 5.00 | 3.651 | 1.079 | 3.006 | -0.052 | 1.000 | 1.39 V | 2.43 V |
| 5.50 | 3.613 | 1.068 | 2.991 | -0.061 | 1.000 | 1.42 V | 2.46 V |
| 10.00 | 2.349 | 0.694 | 2.053 | -0.113 | 1.000 | 1.54 I | 2.68 I |
| 10.60 | 2.316 | 0.684 | 2.040 | -0.124 | 1.000 | 2.12 I | 3.46 I |
| 14.00 | 2.130 | 0.630 | 1.968 | -0.189 | 1.000 | 1.61 I | 2.76 I |
| 21.20 | 1.737 | 0.513 | 1.814 | -0.340 | 1.000 | 1.38 T | 2.49 T |
| 25.00 | 1.530 | 0.452 | 1.732 | -0.429 | 1.000 | 1.04 T | 2.03 T |
| 31.80 | 1.158 | 0.342 | 1.584 | -0.591 | 1.000 | 1.28 T | 2.36 T |
| 42.40 | 0.579 | 0.171 | 1.351 | -0.849 | 1.000 | 0.90 T | 1.85 T |
| 53.00 | -0.000 | -0.000 | 1.114 | -1.114 | 1.000 | 0.81 T | 1.72 T |
| 63.60 | -0.579 | -0.171 | 0.849 | -1.351 | 1.000 | 0.90 T | 1.85 T |
| 74.20 | -1.158 | -0.342 | 0.591 | -1.584 | 1.000 | 1.28 T | 2.36 T |
| 81.00 | -1.530 | -0.452 | 0.429 | -1.732 | 1.000 | 1.04 T | 2.03 T |
| 84.80 | -1.737 | -0.513 | 0.340 | -1.814 | 1.000 | 1.38 T | 2.49 T |
| 92.00 | -2.130 | -0.630 | 0.189 | -1.968 | 1.000 | 1.61 I | 2.76 I |
| 95.40 | -2.316 | -0.684 | 0.124 | -2.040 | 1.000 | 2.12 I | 3.46 I |
| 106.00 | -2.895 | -0.856 | 0.000 | -2.264 | 1.000 | 1.00 V | 1.93 V |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| | NON-COMP MOMENT | OVERLOAD MOMENT | SHEAR | NON-COMPACT RATING FACTORS | | COMPACT MOMENT | COMPACT RATING FACTORS | |
|---|---|---|---|---|---|---|---|---|
| X | STRENGTH | STRENGTH | STRENGTH | IR | OR | STRENGTH | IR | OR |
| 0.00 | 6572.8 B | 5258.2 | 420.1 | 1.14 V | 1.90 V | | | |
| 5.00 | 4331.1 U | 3464.9 | 447.2 | 1.44 V | 2.40 V | | | |
| 5.50 | 4331.1 U | 3464.9 | 447.2 | 1.45 V | 2.42 V | | | |
| 10.00 | 4886.3 U | 3909.0 | 676.5 | 1.65 I | 2.74 I | | | |
| 10.60 | 6572.8 B | 5258.2 | 676.5 | 2.11 I | 3.52 I | | | |
| 14.00 | 6572.8 B | 5258.2 | 676.5 | 1.70 I | 2.83 I | | | |
| 21.20 | 8439.0 B | 6751.2 | 676.5 | 1.56 T | 2.60 T | | | |
| 25.00 | 8439.0 B | 6751.2 | 676.5 | 1.28 T | 2.13 T | | | |
| 31.80 | 10676.8 B | 8541.4 | 676.5 | 1.48 T | 2.46 T | | | |
| 42.40 | 10676.8 B | 8541.4 | 676.5 | 1.16 T | 1.94 T | | | |
| 53.00 | 10676.8 B | 8541.4 | 676.5 | 1.08 T | 1.81 T | | | |
| 63.60 | 10676.8 B | 8541.4 | 676.5 | 1.16 T | 1.94 T | | | |
| 74.20 | 10676.8 B | 8541.4 | 676.5 | 1.48 T | 2.46 T | | | |
| 81.00 | 8439.0 B | 6751.2 | 676.5 | 1.28 T | 2.13 T | | | |
| 84.80 | 8439.0 B | 6751.2 | 676.5 | 1.56 T | 2.60 T | | | |
| 92.00 | 6572.8 B | 5258.2 | 676.5 | 1.70 I | 2.83 I | | | |
| 95.40 | 6572.8 B | 5258.2 | 676.5 | 2.11 I | 3.52 I | | | |
| 106.00 | 6572.8 B | 5258.2 | 420.1 | 1.14 V | 1.90 V | | | |

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+++++++++++++++++++++++++++++++++++++++++++++
+                                           +
+    F L O O R B E A M   A N A L Y S I S    +
+                                           +
+++++++++++++++++++++++++++++++++++++++++++++
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FLOORBEAM SPAN INFORMATION
ANALYZING HALF THE FLOORBEAM

LEFT CANTILEVER: 0.00 FT, FLOORBEAM MAIN SPAN: 27.34 FT
RIGHT CANTILEVER: 0.00 FT
TOTAL FLOORBEAM SPAN (INCLUDING ANY EXISTING CANTILEVER): 27.34 FT

NUMBER OF TRAFFIC LANES: 2, WHERE LANE WIDTH IS BASED ON DESIGN LANE:
DECK WIDTH: 25.00 FT, ROADWAY WIDTH: 23.00 FT
CL OF GIRDER OR TRUSS TO CURB: 2.17 FT

MINIMAL X1 AT ROADWAY: 4.17 FT, MAXIMAL X2 AT ROADWAY: 23.17 FT

NOTE: THE WHEEL LOAD POSITIONS (X1 AND X2) MUST BE BETWEEN MINIMAL_X1_AT_ROADWAY AND MAXIMAL_X2_AT_ROADWAY

FLOORBEAM LIVE LOAD MOMENT AND SHEAR FACTORS

BASED ON DESIGN LANE

| | | | ----------- WHEEL LOAD POSITIONS ----------- | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | LANE 1/4/7 | | LANE 2/5/8 | | LANE 3/6/9 | |
| X | FACTOR | | X1 | X2 | X1 | X2 | X1 | X2 |
| 0.00 | 0.000 | M | 0.00 | 0.00 | | | | |
| | 1.055 | V | 4.17 | 10.17 | 15.67 | 21.67 | | |
| 2.73 | 2.884 | M | 4.17 | 10.17 | 15.67 | 21.67 | | |
| | 1.055 | V | 4.17 | 10.17 | 15.67 | 21.67 | | |
| 5.47 | 5.508 | M | 5.47 | 11.47 | 15.67 | 21.67 | | |
| | 1.007 | V | 5.47 | 11.47 | 15.67 | 21.67 | | |
| 8.20 | 6.936 | M | 5.67 | 11.67 | 15.67 | 21.67 | | |
| | 0.588 | V | 8.27 | 14.27 | | | | |
| 10.94 | 8.303 | M | 5.67 | 11.67 | 15.67 | 21.67 | | |
| | 0.526 | V | 4.97 | 10.97 | 15.67 | 21.67 | | |
| 13.67 | 8.670 | M | 5.67 | 11.67 | 15.67 | 21.67 | | |
| | 0.390 | V | 13.67 | 19.67 | | | | |

MOMENT/SHEAR FACTOR CODES: M - MOMENT, V - SHEAR

NOTE: THE EFFECT OF A SET OF WHEELS PLACED TRANSVERSELY ACROSS THE BRIDGE (ALONG THE FLOORBEAM SPAN) TO FIND THE MAXIMUM POS AND NEG MOMENTS OR SHEARS EXPRESSED IN TERMS OF THE NUMBER OF AXLES IS REFERRED TO AS MOMENT OR SHEAR FACTORS.

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<    FLOORBEAM  4    >
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FLOORBEAM SECTION PROPERTIES
(NON-UNIFORM SECTION & NON-COMPOSITE)

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT |
|---|---|---|---|---|---|---|
| 0.00 | 30.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 |
| 2.73 | 30.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 |
| 5.47 | 30.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 |
| 6.50R | 30.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 |
| 8.20 | 30.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 |
| 10.94 | 30.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 |
| 13.67 | 30.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 |

FLOORBEAM SECTION PROPERTIES
(NON-UNIFORM SECTION & COMPOSITE, N= 8)

EFFECTIVE SLAB WIDTH: 82.02 THICKNESS: 8.00

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S CONC |
|---|---|---|---|---|---|---|---|
| 0.00 | 38.00 | 116.15 | 14057.0 | 28.42 | 8879.78 | 494.67 | 1466.87 |
| 2.73 | 38.00 | 116.15 | 14057.0 | 28.42 | 8879.78 | 494.67 | 1466.87 |
| 5.47 | 38.00 | 116.15 | 14057.0 | 28.42 | 8879.78 | 494.67 | 1466.87 |
| 6.50R | 38.00 | 116.15 | 14057.0 | 28.42 | 8879.78 | 494.67 | 1466.87 |
| 8.20 | 38.00 | 116.15 | 14057.0 | 28.42 | 8879.78 | 494.67 | 1466.87 |
| 10.94 | 38.00 | 116.15 | 14057.0 | 28.42 | 8879.78 | 494.67 | 1466.87 |
| 13.67 | 38.00 | 116.15 | 14057.0 | 28.42 | 8879.78 | 494.67 | 1466.87 |

FLOORBEAM SECTION PROPERTIES
(NON-UNIFORM SECTION & COMPOSITE, N=24)

EFFECTIVE SLAB WIDTH: 82.02 THICKNESS: 8.00

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S CONC |
|---|---|---|---|---|---|---|---|
| 0.00 | 38.00 | 61.47 | 10544.9 | 23.45 | 1610.06 | 449.66 | 724.77 |
| 2.73 | 38.00 | 61.47 | 10544.9 | 23.45 | 1610.06 | 449.66 | 724.77 |
| 5.47 | 38.00 | 61.47 | 10544.9 | 23.45 | 1610.06 | 449.66 | 724.77 |
| 6.50R | 38.00 | 61.47 | 10544.9 | 23.45 | 1610.06 | 449.66 | 724.77 |
| 8.20 | 38.00 | 61.47 | 10544.9 | 23.45 | 1610.06 | 449.66 | 724.77 |
| 10.94 | 38.00 | 61.47 | 10544.9 | 23.45 | 1610.06 | 449.66 | 724.77 |
| 13.67 | 38.00 | 61.47 | 10544.9 | 23.45 | 1610.06 | 449.66 | 724.77 |

FLOORBEAM SECTION PROPERTIES
(NON-UNIFORM SECTION & COMPOSITE, NEGATIVE MOMENT)

EFFECTIVE SLAB WIDTH: 82.02 THICKNESS: 8.00

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S REINF |
|---|---|---|---|---|---|---|---|
| 0.00 | 38.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 | N/A |
| 2.73 | 38.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 | N/A |
| 5.47 | 38.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 | N/A |
| 6.50R | 38.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 | N/A |
| 8.20 | 38.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 | N/A |
| 10.94 | 38.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 | N/A |
| 13.67 | 38.00 | 34.13 | 4919.1 | 15.00 | 327.94 | 327.94 | N/A |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

DEAD LOADS ACTING ON THE FLOORBEAM

| | UNIFORM LOAD | | |
|---|---|---|---|
| FLOORBEAM WEIGHT | SLAB WEIGHT | INPUT DL1FB | INPUT DL2 |
| 0.229 | 0.923 | 0.071 | 0.450 |

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************************************
*  FLOORBEAM  4 -  LIVE LOAD  H20   *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT, DFSM_FB: 0.00

DFSM_FB = 0 INDICATES THE REACTION DUE TO LIVE
LOAD ACTING AT THIS FLOORBEAM WAS CALCULATED BASED ON THE

INFLUENCE LINE METHOD ASSUMING FLOORING TO ACT AS A SIMPLE SPAN OR CONTINUOUS SPAN IN THE DIRECTION OF TRAFFIC.

LIVE LOAD REACTION FROM DECK (ONE LANE): 32.08

LIVE LOAD IMPACT FACTORS: POS MOM 1.30

UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 16.7 | 6.2 | 44.0 | 1.30 |
| 2.73 | 41.1 | 15.1 | 120.3 | 13.4 | 4.9 | 44.0 | 1.30 |
| 5.47 | 73.2 | 26.9 | 229.7 | 10.0 | 3.7 | 42.0 | 1.30 |
| 6.50R | 82.9 | 30.5 | 258.0 | 8.8 | 3.2 | 35.4 | 1.30 |
| 8.20 | 96.0 | 35.3 | 289.3 | 6.7 | 2.5 | 24.5 | 1.30 |
| 10.94 | 109.7 | 40.4 | 346.3 | 3.3 | 1.2 | 21.9 | 1.30 |
| 13.67 | 114.3 | 42.1 | 361.6 | 0.0 | 0.0 | 16.3 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2.73 | -1.506 | -0.113 | -0.163 | 0.000 | 1.506 | 0.404 | 2.918 | 0.000 |
| 5.47 | -2.677 | -0.201 | -0.310 | 0.000 | 2.677 | 0.719 | 5.572 | 0.000 |
| 6.50 | -3.032 | -0.227 | -0.349 | 0.000 | 3.032 | 0.814 | 6.260 | 0.000 |
| 8.20 | -3.513 | -0.263 | -0.391 | 0.000 | 3.513 | 0.943 | 7.017 | 0.000 |
| 10.94 | -4.015 | -0.301 | -0.468 | 0.000 | 4.015 | 1.078 | 8.400 | 0.000 |
| 13.67 | -4.183 | -0.314 | -0.489 | 0.000 | 4.183 | 1.123 | 8.771 | 0.000 |

FLEXURAL STRESSES - SLAB

| | CONCRETE STRESS | | SLAB REINF STRESS | |
|---|---|---|---|---|
| X | DL2 | +(LL+I) | DL2 | -(LL+I) |
| 0.00 | 0.000 | 0.000 | N/A | N/A |
| 2.73 | -0.010 | -0.123 | N/A | N/A |
| 5.47 | -0.019 | -0.235 | N/A | N/A |
| 6.50 | -0.021 | -0.264 | N/A | N/A |
| 8.20 | -0.024 | -0.296 | N/A | N/A |
| 10.94 | -0.028 | -0.354 | N/A | N/A |
| 13.67 | -0.029 | -0.370 | N/A | N/A |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 1.196 | 0.440 | 3.147 | 0.000 | 1.000 | 2.98 V | 4.23 V |
| 2.73 | 0.957 | 0.352 | 3.147 | 0.000 | 1.000 | 3.08 V | 4.34 V |
| 5.47 | 0.718 | 0.264 | 3.005 | 0.000 | 1.000 | 2.65 B | 3.83 B |
| 6.50 | 0.550 | 0.202 | 2.218 | 0.000 | 1.000 | 2.29 B | 3.34 B |
| 8.20 | 0.419 | 0.154 | 1.536 | 0.000 | 1.000 | 1.95 B | 2.89 B |
| 10.94 | 0.210 | 0.077 | 1.373 | 0.000 | 1.000 | 1.55 B | 2.34 B |
| 13.67 | 0.000 | 0.000 | 1.020 | 0.000 | 1.000 | 1.46 B | 2.22 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 1360.3 B | 1292.3 | 267.6 | 2.50 V | 4.16 V | 1848.4 | 2.50 V | 4.16 V |
| 2.73 | 1360.3 B | 1292.3 | 267.6 | 2.56 V | 4.26 V | 1848.4 | 2.56 V | 4.26 V |
| 5.47 | 1360.3 B | 1292.3 | 267.6 | 2.37 B | 3.95 B | 1848.4 | 2.74 V | 4.57 V |
| 6.50 | 1360.3 B | 1292.3 | 305.5 | 2.06 B | 3.44 B | 1952.6 | 2.64 O | 4.39 O |
| 8.20 | 1360.3 B | 1292.3 | 305.5 | 1.79 B | 2.98 B | 1952.6 | 2.30 O | 3.83 O |
| 10.94 | 1360.3 B | 1292.3 | 305.5 | 1.45 B | 2.42 B | 1952.6 | 1.88 O | 3.13 O |
| 13.67 | 1360.3 B | 1292.3 | 305.5 | 1.37 B | 2.29 B | 1952.6 | 1.78 O | 2.97 O |

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*************************************
*  FLOORBEAM  4 -  LIVE LOAD  HS20  *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT, DFSM_FB:   0.00

DFSM_FB = 0 INDICATES THE REACTION DUE TO LIVE LOAD ACTING AT THIS FLOORBEAM WAS CALCULATED BASED ON THE INFLUENCE LINE METHOD ASSUMING FLOORING TO ACT AS A SIMPLE SPAN OR CONTINUOUS SPAN IN THE DIRECTION OF TRAFFIC.

LIVE LOAD REACTION FROM DECK (ONE LANE):  32.08

LIVE LOAD IMPACT FACTORS: POS MOM 1.30

UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 16.7 | 6.2 | 44.0 | 1.30 |
| 2.73 | 41.1 | 15.1 | 120.3 | 13.4 | 4.9 | 44.0 | 1.30 |
| 5.47 | 73.2 | 26.9 | 229.7 | 10.0 | 3.7 | 42.0 | 1.30 |
| 6.50R | 82.9 | 30.5 | 258.0 | 8.8 | 3.2 | 35.4 | 1.30 |
| 8.20 | 96.0 | 35.3 | 289.3 | 6.7 | 2.5 | 24.5 | 1.30 |
| 10.94 | 109.7 | 40.4 | 346.3 | 3.3 | 1.2 | 21.9 | 1.30 |
| 13.67 | 114.3 | 42.1 | 361.6 | 0.0 | 0.0 | 16.3 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| X | TOP FIBER STEEL STRESS DL1 | TOP DL2 | TOP +(LL+I) | TOP -(LL+I) | BOTTOM FIBER STEEL STRESS DL1 | BOTTOM DL2 | BOTTOM +(LL+I) | BOTTOM -(LL+I) |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2.73 | -1.506 | -0.113 | -0.163 | 0.000 | 1.506 | 0.404 | 2.918 | 0.000 |
| 5.47 | -2.677 | -0.201 | -0.310 | 0.000 | 2.677 | 0.719 | 5.572 | 0.000 |
| 6.50 | -3.032 | -0.227 | -0.349 | 0.000 | 3.032 | 0.814 | 6.260 | 0.000 |
| 8.20 | -3.513 | -0.263 | -0.391 | 0.000 | 3.513 | 0.943 | 7.017 | 0.000 |
| 10.94 | -4.015 | -0.301 | -0.468 | 0.000 | 4.015 | 1.078 | 8.400 | 0.000 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 13.67 | -4.183 | -0.314 | -0.489 | 0.000 | 4.183 | 1.123 | 8.771 | 0.000 |

FLEXURAL STRESSES - SLAB

| | CONCRETE STRESS | | SLAB REINF STRESS | |
|---|---|---|---|---|
| X | DL2 | +(LL+I) | DL2 | -(LL+I) |
| 0.00 | 0.000 | 0.000 | N/A | N/A |
| 2.73 | -0.010 | -0.123 | N/A | N/A |
| 5.47 | -0.019 | -0.235 | N/A | N/A |
| 6.50 | -0.021 | -0.264 | N/A | N/A |
| 8.20 | -0.024 | -0.296 | N/A | N/A |
| 10.94 | -0.028 | -0.354 | N/A | N/A |
| 13.67 | -0.029 | -0.370 | N/A | N/A |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 1.196 | 0.440 | 3.147 | 0.000 | 1.000 | 2.98 V | 4.23 V |
| 2.73 | 0.957 | 0.352 | 3.147 | 0.000 | 1.000 | 3.08 V | 4.34 V |
| 5.47 | 0.718 | 0.264 | 3.005 | 0.000 | 1.000 | 2.65 B | 3.83 B |
| 6.50 | 0.550 | 0.202 | 2.218 | 0.000 | 1.000 | 2.29 B | 3.34 B |
| 8.20 | 0.419 | 0.154 | 1.536 | 0.000 | 1.000 | 1.95 B | 2.89 B |
| 10.94 | 0.210 | 0.077 | 1.373 | 0.000 | 1.000 | 1.55 B | 2.34 B |
| 13.67 | 0.000 | 0.000 | 1.020 | 0.000 | 1.000 | 1.46 B | 2.22 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| | NON-COMP | OVERLOAD | | NON-COMPACT | | COMPACT | COMPACT | |
|---|---|---|---|---|---|---|---|---|
| | MOMENT | MOMENT | SHEAR | RATING FACTORS | | MOMENT | RATING FACTORS | |
| X | STRENGTH | STRENGTH | STRENGTH | IR | OR | STRENGTH | IR | OR |
| 0.00 | 1360.3 B | 1292.3 | 267.6 | 2.50 V | 4.16 V | 1848.4 | 2.50 V | 4.16 V |
| 2.73 | 1360.3 B | 1292.3 | 267.6 | 2.56 V | 4.26 V | 1848.4 | 2.56 V | 4.26 V |
| 5.47 | 1360.3 B | 1292.3 | 267.6 | 2.37 B | 3.95 B | 1848.4 | 2.74 V | 4.57 V |
| 6.50 | 1360.3 B | 1292.3 | 305.5 | 2.06 B | 3.44 B | 1952.6 | 2.64 O | 4.39 O |
| 8.20 | 1360.3 B | 1292.3 | 305.5 | 1.79 B | 2.98 B | 1952.6 | 2.30 O | 3.83 O |
| 10.94 | 1360.3 B | 1292.3 | 305.5 | 1.45 B | 2.42 B | 1952.6 | 1.88 O | 3.13 O |
| 13.67 | 1360.3 B | 1292.3 | 305.5 | 1.37 B | 2.29 B | 1952.6 | 1.78 O | 2.97 O |

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*  FLOORBEAM  4 -  LIVE LOAD TK527  *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT, DFSM_FB:   0.00

DFSM_FB = 0 INDICATES THE REACTION DUE TO LIVE LOAD ACTING AT THIS FLOORBEAM WAS CALCULATED BASED ON THE INFLUENCE LINE METHOD ASSUMING FLOORING TO ACT AS A SIMPLE SPAN OR CONTINUOUS SPAN IN THE DIRECTION OF TRAFFIC.

LIVE LOAD REACTION FROM DECK (ONE TRUCK):  38.60

LIVE LOAD IMPACT FACTORS: POS MOM 1.30

UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 16.7 | 6.2 | 52.9 | 1.30 |
| 2.73 | 41.1 | 15.1 | 144.7 | 13.4 | 4.9 | 52.9 | 1.30 |
| 5.47 | 73.2 | 26.9 | 276.4 | 10.0 | 3.7 | 50.5 | 1.30 |
| 6.50R | 82.9 | 30.5 | 310.5 | 8.8 | 3.2 | 42.6 | 1.30 |
| 8.20 | 96.0 | 35.3 | 348.0 | 6.7 | 2.5 | 29.5 | 1.30 |
| 10.94 | 109.7 | 40.4 | 416.6 | 3.3 | 1.2 | 26.4 | 1.30 |
| 13.67 | 114.3 | 42.1 | 435.0 | 0.0 | 0.0 | 19.6 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2.73 | -1.506 | -0.113 | -0.196 | 0.000 | 1.506 | 0.404 | 3.510 | 0.000 |
| 5.47 | -2.677 | -0.201 | -0.373 | 0.000 | 2.677 | 0.719 | 6.704 | 0.000 |
| 6.50 | -3.032 | -0.227 | -0.420 | 0.000 | 3.032 | 0.814 | 7.532 | 0.000 |
| 8.20 | -3.513 | -0.263 | -0.470 | 0.000 | 3.513 | 0.943 | 8.443 | 0.000 |
| 10.94 | -4.015 | -0.301 | -0.563 | 0.000 | 4.015 | 1.078 | 10.107 | 0.000 |
| 13.67 | -4.183 | -0.314 | -0.588 | 0.000 | 4.183 | 1.123 | 10.553 | 0.000 |

FLEXURAL STRESSES - SLAB

| | CONCRETE STRESS | | SLAB REINF STRESS | |
|---|---|---|---|---|
| X | DL2 | +(LL+I) | DL2 | -(LL+I) |
| 0.00 | 0.000 | 0.000 | N/A | N/A |
| 2.73 | -0.010 | -0.148 | N/A | N/A |
| 5.47 | -0.019 | -0.283 | N/A | N/A |
| 6.50 | -0.021 | -0.317 | N/A | N/A |
| 8.20 | -0.024 | -0.356 | N/A | N/A |
| 10.94 | -0.028 | -0.426 | N/A | N/A |
| 13.67 | -0.029 | -0.445 | N/A | N/A |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 1.196 | 0.440 | 3.786 | 0.000 | 1.000 | 2.47 V | 3.52 V |
| 2.73 | 0.957 | 0.352 | 3.786 | 0.000 | 1.000 | 2.56 V | 3.61 V |
| 5.47 | 0.718 | 0.264 | 3.615 | 0.000 | 1.000 | 2.20 B | 3.19 B |
| 6.50 | 0.550 | 0.202 | 2.669 | 0.000 | 1.000 | 1.90 B | 2.78 B |
| 8.20 | 0.419 | 0.154 | 1.848 | 0.000 | 1.000 | 1.62 B | 2.40 B |
| 10.94 | 0.210 | 0.077 | 1.652 | 0.000 | 1.000 | 1.29 B | 1.94 B |
| 13.67 | 0.000 | 0.000 | 1.227 | 0.000 | 1.000 | 1.22 B | 1.84 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 1360.3 B | 1292.3 | 267.6 | 2.07 V | 3.46 V | 1848.4 | 2.07 V | 3.46 V |
| 2.73 | 1360.3 B | 1292.3 | 267.6 | 2.13 V | 3.54 V | 1848.4 | 2.13 V | 3.54 V |
| 5.47 | 1360.3 B | 1292.3 | 267.6 | 1.97 B | 3.28 B | 1848.4 | 2.28 V | 3.80 V |
| 6.50 | 1360.3 B | 1292.3 | 305.5 | 1.72 B | 2.86 B | 1952.6 | 2.19 O | 3.65 O |
| 8.20 | 1360.3 B | 1292.3 | 305.5 | 1.49 B | 2.48 B | 1952.6 | 1.91 O | 3.19 O |
| 10.94 | 1360.3 B | 1292.3 | 305.5 | 1.20 B | 2.01 B | 1952.6 | 1.56 O | 2.60 O |
| 13.67 | 1360.3 B | 1292.3 | 305.5 | 1.14 B | 1.90 B | 1952.6 | 1.48 O | 2.47 O |

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*  FLOORBEAM  4 -  LIVE LOAD  ML80  *
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LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT, DFSM_FB: 0.00

DFSM_FB = 0 INDICATES THE REACTION DUE TO LIVE LOAD ACTING AT THIS FLOORBEAM WAS CALCULATED BASED ON THE INFLUENCE LINE METHOD ASSUMING FLOORING TO ACT AS A SIMPLE SPAN OR CONTINUOUS SPAN IN THE DIRECTION OF TRAFFIC.

LIVE LOAD REACTION FROM DECK (ONE TRUCK): 44.45

LIVE LOAD IMPACT FACTORS: POS MOM 1.30

UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 16.7 | 6.2 | 61.0 | 1.30 |
| 2.73 | 41.1 | 15.1 | 166.7 | 13.4 | 4.9 | 61.0 | 1.30 |
| 5.47 | 73.2 | 26.9 | 318.3 | 10.0 | 3.7 | 58.2 | 1.30 |
| 6.50R | 82.9 | 30.5 | 357.6 | 8.8 | 3.2 | 49.1 | 1.30 |
| 8.20 | 96.0 | 35.3 | 400.8 | 6.7 | 2.5 | 34.0 | 1.30 |
| 10.94 | 109.7 | 40.4 | 479.8 | 3.3 | 1.2 | 30.4 | 1.30 |
| 13.67 | 114.3 | 42.1 | 501.0 | 0.0 | 0.0 | 22.6 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| X | TOP FIBER STEEL STRESS DL1 | DL2 | +(LL+I) | -(LL+I) | BOTTOM FIBER STEEL STRESS DL1 | DL2 | +(LL+I) | -(LL+I) |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2.73 | -1.506 | -0.113 | -0.225 | 0.000 | 1.506 | 0.404 | 4.043 | 0.000 |
| 5.47 | -2.677 | -0.201 | -0.430 | 0.000 | 2.677 | 0.719 | 7.721 | 0.000 |
| 6.50 | -3.032 | -0.227 | -0.483 | 0.000 | 3.032 | 0.814 | 8.674 | 0.000 |
| 8.20 | -3.513 | -0.263 | -0.542 | 0.000 | 3.513 | 0.943 | 9.723 | 0.000 |
| 10.94 | -4.015 | -0.301 | -0.648 | 0.000 | 4.015 | 1.078 | 11.640 | 0.000 |
| 13.67 | -4.183 | -0.314 | -0.677 | 0.000 | 4.183 | 1.123 | 12.154 | 0.000 |

FLEXURAL STRESSES - SLAB

| X | CONCRETE STRESS DL2 | CONCRETE STRESS +(LL+I) | SLAB REINF STRESS DL2 | SLAB REINF STRESS -(LL+I) |
|---|---|---|---|---|

| | | | | |
|---|---|---|---|---|
| 0.00 | 0.000 | 0.000 | N/A | N/A |
| 2.73 | -0.010 | -0.170 | N/A | N/A |
| 5.47 | -0.019 | -0.325 | N/A | N/A |
| 6.50 | -0.021 | -0.366 | N/A | N/A |
| 8.20 | -0.024 | -0.410 | N/A | N/A |
| 10.94 | -0.028 | -0.491 | N/A | N/A |
| 13.67 | -0.029 | -0.512 | N/A | N/A |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 1.196 | 0.440 | 4.360 | 0.000 | 1.000 | 2.15 V | 3.06 V |
| 2.73 | 0.957 | 0.352 | 4.360 | 0.000 | 1.000 | 2.22 V | 3.13 V |
| 5.47 | 0.718 | 0.264 | 4.164 | 0.000 | 1.000 | 1.91 B | 2.77 B |
| 6.50 | 0.550 | 0.202 | 3.074 | 0.000 | 1.000 | 1.65 B | 2.41 B |
| 8.20 | 0.419 | 0.154 | 2.128 | 0.000 | 1.000 | 1.41 B | 2.09 B |
| 10.94 | 0.210 | 0.077 | 1.903 | 0.000 | 1.000 | 1.12 B | 1.69 B |
| 13.67 | 0.000 | 0.000 | 1.413 | 0.000 | 1.000 | 1.06 B | 1.60 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 1360.3 B | 1292.3 | 267.6 | 1.80 V | 3.00 V | 1848.4 | 1.80 V | 3.00 V |
| 2.73 | 1360.3 B | 1292.3 | 267.6 | 1.85 V | 3.08 V | 1848.4 | 1.85 V | 3.08 V |
| 5.47 | 1360.3 B | 1292.3 | 267.6 | 1.71 B | 2.85 B | 1848.4 | 1.98 V | 3.30 V |
| 6.50 | 1360.3 B | 1292.3 | 305.5 | 1.49 B | 2.48 B | 1952.6 | 1.90 O | 3.17 O |
| 8.20 | 1360.3 B | 1292.3 | 305.5 | 1.29 B | 2.15 B | 1952.6 | 1.66 O | 2.77 O |
| 10.94 | 1360.3 B | 1292.3 | 305.5 | 1.05 B | 1.74 B | 1952.6 | 1.35 O | 2.26 O |
| 13.67 | 1360.3 B | 1292.3 | 305.5 | 0.99 B | 1.65 B | 1952.6 | 1.29 O | 2.14 O |

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+                                           +
+       R A T I N G   S U M M A R Y         +
+                                           +
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MEMBER: GIRDER

| LOAD | | ALLOWABLE STRESS RATING FACTOR | TONS | X | SPAN | LOAD FACTOR RATING FACTOR | TONS | X | SPAN |
|---|---|---|---|---|---|---|---|---|---|
| H20 | IR (CRITICAL) | 1.07 T | 21.3 | 53.00 | 1 | 1.35 V | 26.9 | 106.00 | 1 |
| | OR (CRITICAL) | 2.28 T | 45.5 | 53.00 | 1 | 2.24 V | 44.9 | 106.00 | 1 |
| | IR ( POS MOM) | 1.07 T | 21.3 | 53.00 | 1 | 1.44 T | 28.7 | 53.00 | 1 |
| | OR ( POS MOM) | 2.28 T | 45.5 | 53.00 | 1 | 2.39 T | 47.8 | 53.00 | 1 |
| HS20 | IR (CRITICAL) | 0.90 T | 32.4 | 53.00 | 1 | 1.21 T | 43.7 | 53.00 | 1 |
| | OR (CRITICAL) | 1.92 T | 69.2 | 53.00 | 1 | 2.02 T | 72.8 | 53.00 | 1 |
| | IR ( POS MOM) | 0.90 T | 32.4 | 53.00 | 1 | 1.21 T | 43.7 | 53.00 | 1 |
| | OR ( POS MOM) | 1.92 T | 69.2 | 53.00 | 1 | 2.02 T | 72.8 | 53.00 | 1 |
| TK527 | IR (CRITICAL) | 0.79 T | 31.7 | 53.00 | 1 | 1.07 T | 42.6 | 53.00 | 1 |
| | OR (CRITICAL) | 1.69 T | 67.6 | 53.00 | 1 | 1.78 T | 71.0 | 53.00 | 1 |
| | IR ( POS MOM) | 0.79 T | 31.7 | 53.00 | 1 | 1.07 T | 42.6 | 53.00 | 1 |
| | OR ( POS MOM) | 1.69 T | 67.6 | 53.00 | 1 | 1.78 T | 71.0 | 53.00 | 1 |
| ML80 | IR (CRITICAL) | 0.81 T | 29.5 | 53.00 | 1 | 1.08 T | 39.7 | 53.00 | 1 |

```
       OR (CRITICAL)    1.72 T    63.0    53.00  1    1.81 T    66.2    53.00  1
       IR ( POS MOM)    0.81 T    29.5    53.00  1    1.08 T    39.7    53.00  1
       OR ( POS MOM)    1.72 T    63.0    53.00  1    1.81 T    66.2    53.00  1

  NOTE: THIS RATING SUMMARY OF GIRDERS DOES FOLLOW THE ALL-OR-NONE
        COMPACT REQUIREMENTS.
        THIS GIRDER DOES NOT HAVE ANY SECTIONS QUALIFYING AS COMPACT
        SECTIONS.  THEREFORE, THE RATING PRINTED IN THE SUMMARY IS THAT
        BASED ON A NON-COMPACT SECTION.

MEMBER: FLOORBEAM
                           ALLOWABLE STRESS RATING       LOAD FACTOR RATING
  LOAD                 FACTOR     TONS      X   FLBM  FACTOR     TONS      X   FLBM
  H20  IR (CRITICAL)    1.46 B    29.3    13.67  4    1.78 O    35.6    13.67  4
       OR (CRITICAL)    2.22 B    44.3    13.67  4    2.97 O    59.4    13.67  4
       IR ( POS MOM)    1.46 B    29.3    13.67  4    1.78 O    35.6    13.67  4
       OR ( POS MOM)    2.22 B    44.3    13.67  4    2.97 O    59.4    13.67  4
  HS20 IR (CRITICAL)    1.46 B    52.7    13.67  4    1.78 O    64.1    13.67  4
       OR (CRITICAL)    2.22 B    79.8    13.67  4    2.97 O   106.9    13.67  4
       IR ( POS MOM)    1.46 B    52.7    13.67  4    1.78 O    64.1    13.67  4
       OR ( POS MOM)    2.22 B    79.8    13.67  4    2.97 O   106.9    13.67  4
 TK527 IR (CRITICAL)    1.22 B    48.7    13.67  4    1.48 O    59.2    13.67  4
       OR (CRITICAL)    1.84 B    73.7    13.67  4    2.47 O    98.7    13.67  4
       IR ( POS MOM)    1.22 B    48.7    13.67  4    1.48 O    59.2    13.67  4
       OR ( POS MOM)    1.84 B    73.7    13.67  4    2.47 O    98.7    13.67  4
  ML80 IR (CRITICAL)    1.06 B    38.7    13.67  4    1.29 O    47.1    13.67  4
       OR (CRITICAL)    1.60 B    58.6    13.67  4    2.14 O    78.5    13.67  4
       IR ( POS MOM)    1.06 B    38.7    13.67  4    1.29 O    47.1    13.67  4
       OR ( POS MOM)    1.60 B    58.6    13.67  4    2.14 O    78.5    13.67  4

  NOTE: THIS RATING SUMMARY OF FLOORBEAMS DOES FOLLOW THE ALL-OR-NONE
        COMPACT REQUIREMENTS.
        THIS FLOORBEAM DOES HAVE ALL SECTIONS QUALIFYING AS COMPACT
        SECTIONS.  THEREFORE, THE RATING PRINTED IN THE SUMMARY
        IS THAT BASED ON COMPACT SECTIONS.

      RATING FACTOR CODES: FIRST CHARACTER AFTER THE RATING FACTOR
      T - TOP STEEL STRESS/STRENGTH GOVERNS
      B - BOTTOM STEEL STRESS/STRENGTH GOVERNS
      C - CONCRETE STRESS/STRENGTH GOVERNS
      R - REINFORCEMENT STRESS/STRENGTH GOVERNS
      V - SHEAR STRESS/STRENGTH GOVERNS
      blank - COMPACT MOMENT STRENGTH GOVERNS
      O - OVERLOAD PROVISIONS GOVERN
      I - MOMENT-SHEAR INTERACTION GOVERNS

      RATING FACTOR CODES FOR FLANGE/WEB BUCKLING: SECOND CHARACTER AFTER THE RATING
FACTOR
      blank - SECTION DOES MEET FLANGE/WEB BUCKLING CRITERIA
          AND WILL NOT BUCKLE.
      F - SECTION DOES NOT MEET FLANGE PROJECTION/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE.
      W - SECTION DOES NOT MEET WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN WEB.
      Z - SECTION DOES NOT MEET BOTH FLANGE PROJECTION/THICKNESS RATIO
          CRITERIA AND WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE OR WEB.

      NON-COMPACT MOMENT STRENGTH CODES:
      B - SECTION IS BRACED
      U - SECTION IS UNBRACED

  NOTE: ALL RATINGS ARE BASED ON THE NUMBER OF DESIGN LANES OR THE ACTUAL
```

```
        TRAFFIC LANES AS DEFINED BY "D" OR "L" ENTERED FOR LANES IN THE
        PROJECT IDENTIFICATION.

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

## 3.10 CONCRETE ENCASED STEEL I-BEAM

This Section covers the rating of concrete encased steel I-beam bridges.

### 3.10.1 Policies and Guidelines

Most encased steel beam bridges can be rated by analytical methods based on design plans and field measurements. If design plans are not available, PennDOT standard plans can be utilized if field measured values are in reasonable agreement with one of the standard plans.

If plans are not available and field measurements are not in reasonable agreement with PennDOT standard plans, engineering judgement rating shall be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B.5.2.1 for guidance on structural steel, MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.10.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating.

#### 3.10.2.1 LFR or ASR Method

Based on the age of these bridges, the typical load rating method will be LFR. PennDOT's BAR7 program (Bridge Type EIB or GGG) should be utilized to analyze and rate these bridges when the necessary information is available to complete an analytical rating.

#### 3.10.2.2 LRFR Method

Most concrete encased steel I-beam bridges were built prior to 2011 and designed by load factor or allowable stress methods; therefore, the LRFR rating method would rarely be utilized.

### 3.10.3 Live Load and Dead Load Distribution

#### 3.10.3.1 LFR or ASR Method

Concrete encased steel I-beam bridges built prior to 2011 and designed by load factor or allowable stress methods should be rated using either the LFR or ASR rating method. See AASHTO Standard Specification Section 3.23 for guidance on distribution of live load to longitudinal girders.

The typical distribution factors are as follows when AASHTO distribution factors are applicable:

| Exterior | | Interior | |
|---|---|---|---|
| Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.3.1.2) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO Table 3.23.1) |
| Lever Rule* | Lever Rule* | Lever Rule* | • S/7.0 (one lane)<br>If S > 10' use Lever Rule<br>• S/5.5 (2 or more lanes)<br>If S > 14' use Lever Rule |

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2/2/3 and Pub. 238 IP 3.3.3.1 for additional discussion of skew.

Dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all girders (AASHTO Standard Specification 3.23.2.3.1.1).

3.10.3.2 LRFR Method

Most concrete encased steel I-beam bridges were built prior to 2011 and designed by load factor or allowable stress methods; therefore, the LRFR rating method would rarely be utilized.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO distribution factors. See DM-4 Table 4.6.2.2.3c-1 for the formulas for the skew correction factor.

### 3.10.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1918-1930 Volume 1 | Reinforced Concrete Bridge | 2/27/1922 | 620-B Series, 622-B Series, 624-B Series, 626-B Series, 628-B Series, 630-B Series, 632-B Series, 634-B Series, 636-B Series, 638-B Series, 640-B Series |

### 3.10.5 Modeling Section Properties and Deterioration

See Section 2.6.1.2 for a discussion of modeling section loss in steel beams when the encased I-beam is exposed.

### 3.10.6 Standard Practices

If plans and/or details are unavailable to verify the design:

- The concrete encased I-beam may be considered composite for live load (Bridge type EIB in BAR7) if the conditions listed below are met:
  - There is no distress noted between the concrete encasement around the beams and the deck indicating a loss of composite action.
  - The superstructure and deck ratings are both greater than 4.

- If the conditions above are not met, the bridge must be modeled as non-composite for live load (bridge type GGG in BAR7). Note, there may be situations when the load rater believes the beam is still acting compositely with the deck even though the deck or superstructure condition is a'4'. For these situations, the load rater may consider the bridge deck to be composite (bridge type EIB in BAR7) as long as good justification is provided in the load rating documentation.
- Concrete encased beam bridges may have been constructed with shored or unshored formwork. For shored construction (as seen in the BAR7 User's Manual, Section 5.24, Load Type notes), the DL1 loads are applied to the composite section properties, as the shoring would remain until the concrete sets.

BAR7 does not consider concrete below the neutral axis when determining the flexural capacity of the structure. Therefore, for any spalls below the neutral axis:

- If there is exposed steel that does not exhibit section loss, no modifications to the design section in BAR7 is needed.
- If the exposed steel has section loss and/or is deemed ineffective, the section loss must be accounted for in the BAR7 run by modifying the section properties of the encased I-beam.

Individual beams may be analyzed as steel multi-girder bridge (GGG) in BAR7 if they are non-composite or their deterioration has eliminated composite action.

Concrete encased beam will often have negative haunches, as the top of the steel beam was embedded into the bottom of the slab.

Concrete encased beam bridges may have been constructed with shored or unshored formwork. For shored construction (as seen in the BAR7 User's Manual, Section 5.24, Load Type notes), the DL1 loads are applied to the composite section properties, as the shoring would remain until the concrete sets.

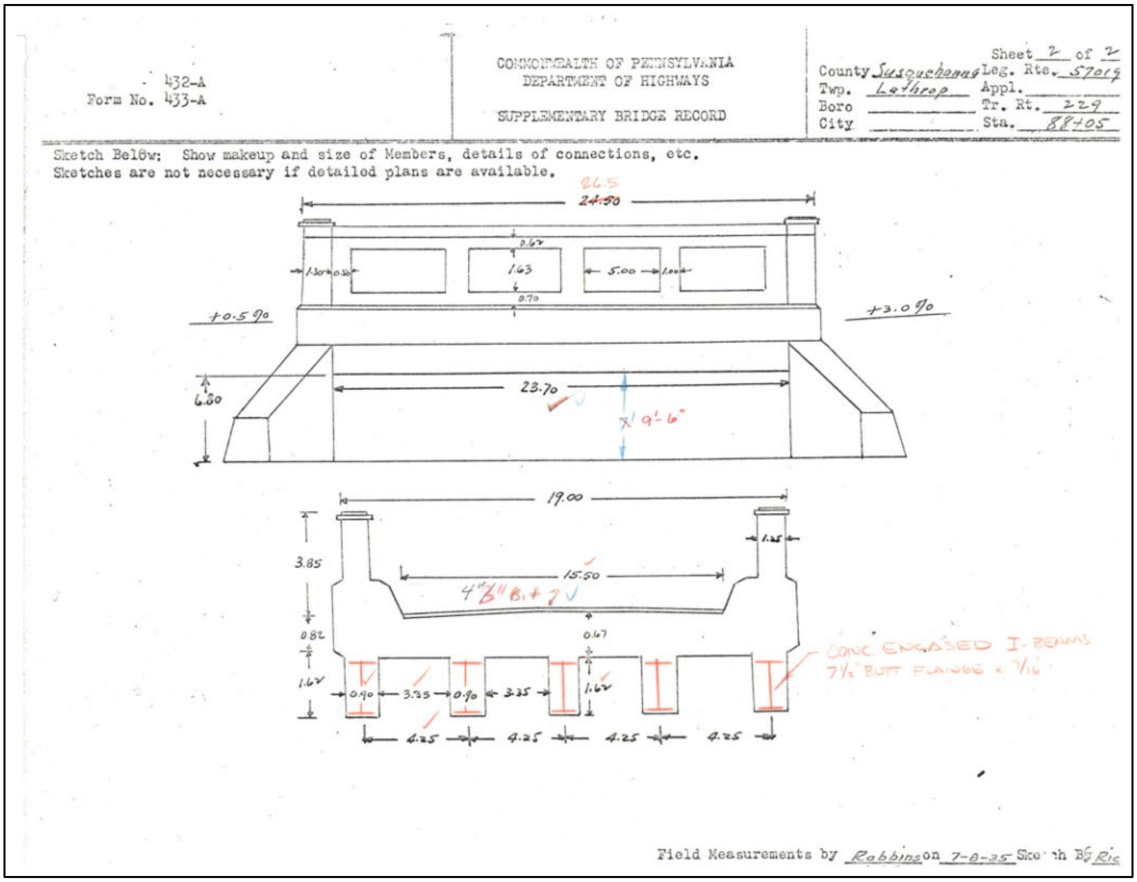
### 3.10.7 Common QA Findings

This Section is in development and will be provided in future editions of this manual.

### 3.10.8 Sample Load Rating

This Section contains sample load rating calculations for an interior and exterior concrete encased I-beam of a single span steel beam structure shown below. The analysis will be performed using PennDOT's Bridge Analysis and Rating (BAR7) Program, Version 7.15.0.0 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part-IV (DM-4).

The single span superstructure consists of an 8" reinforced concrete deck supported on five (5) concrete encased steel I-beams (WF18/CB181). Since existing bridge plans are not available but the superstructure and deck condition is a '5' and there is no distress between the concrete encasement and beams; therefore, assume the bridge is composite for live load (use Bridge type EIB). The steel beams are assumed to be Grade 33.


432-A
Form No. 433-A
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF HIGHWAYS
SUPPLEMENTARY BRIDGE RECORD
Sheet 2 of 2
County Susquehanna Leg. Rte. 57019
Twp. Lathrop Appl.
Boro Tr. Rt. 229
City Sta. 88+05
Sketch Below: Show makeup and size of Members, details of connections, etc.
Sketches are not necessary if detailed plans are available.
26.5
24.50
0.67
1.63
5.00
1.00
1.30
0.50
0.70
+0.5%
+3.0%
23.70
9'-6"
6.80
19.00
1.25
3.85
15.50
0.82
0.67
1.62
0.90
3.35
0.90
3.35
1.62
4.25
4.25
4.25
4.25
CONC. ENCASED I-BEAMS
7½" BUTT FLANGE x 7/16
Field Measurements by Robbins on 7-8-35 Sketch By Ric

### 3.10.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

Done By: Date:

Checked By: Date:

| | |
|---|---|
| Structure ID (5A01): | 57209600400545 |
| Inspection Date (7A01): | 8/24/2022 |
| Facility Carried (5A08): | SR 2096 over Horton Creek |
| Feature Intersected (5A07): | Horton Creek |
| Structure Type (6A26 - 6A29): | Concrete Encased Rolled Steel I-Beam Bridge |
| Spans / Members Analyzed: | Span 1 / Interior and Exterior Beams |
| Analysis Method: | LFD |
| PennDOT Program / Version: | BAR7 Version 7.15.0.0 |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 1.62 | 32.5 | 2.71 | 54.1 | 2.71 | 54.1 | * | * | O | O |
| HS20 | 1.61 | 57.8 | 2.68 | 96.3 | 2.68 | 96.3 | * | * | V | V |
| ML80 | 1.07 | 39.3 | 1.79 | 65.6 | 1.79 | 65.6 | * | * | O | O |
| TK527 | 1.20 | 48.2 | 2.01 | 80.3 | 2.01 | 80.3 | * | * | O | O |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 1.55 | 44.6 | 2.58 | 74.3 | 2.58 | 74.3 | * | * | O | O |
| EV3 | 1.00 | 43.0 | 1.67 | 71.7 | 1.67 | 71.7 | * | * | O | O |

Comments/Assumptions*:

Superstructure and substructure condition ratings are 5 and 5, respectively. Therefore, the SLC Factor equals 1.0 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is in Interior Beam 3.

BAR7 analysis includes section loss/deterioration on the interior beam.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

#### 3.10.8.2 Interior Beam Load Rating Analysis

##### 3.10.8.2.1 BAR7 Input Parameters

A typical interior beam will be rated using BAR7. Some of the BAR7 input parameters can be left blank if the default values are correct. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

<u>Project Identification</u>

- Project Identification = "=BRRAT"
- Structure ID = 57209600400545
- Description = INTERIOR BEAM 3
- Bridge Type = EIB
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = 3 for Load Factor Ratings
- Concrete Deck = Blank

<u>Bridge Cross Section and Loading</u>

- Beam Spacing, S = 4.25 ft per Field Sketch
- Distribution Factor – Shear

  Per AASHTO 3.23.1.2, the interior beam distribution factor for shear is based on the Lever Rule. Since the beam spacing is less than 6 feet, only 1 wheel can be considered.
  $DF_V$ = 1 wheel
  $DF_V$ = 1 wheel / (2 wheels/axle) = 0.500 axles

- Distribution Factor – Moment

  Per AASHTO Table 3.23.1, the interior beam distribution factor for moment is calculated as S/7 for bridges with one traffic lane for a concrete floor on steel. In accordance with AASHTO 3.12.2, the reduction in load intensity is not applicable when Table 3.23.1 is used for moment in longitudinal beams.

  $DF_M$ = S / 7 = 4.25 ft / 7 = 0.607 wheels x (1 axle/2 wheels) = 0.304 axles

- Distribution Factor – Deflection
  Per AASHTO 10.6.4, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
  Roadway Width = 15.5 ft per Inspection Sketch
  $N_L$ = 15.5 ft / 12 ft = 1.29 ==> Use 1 lane
  Reduction Factor = 1.0 for 1 lane per AASHTO 3.12.1
  $DF_{Defl}$ = 1 lane x 1.0 / 5 beams = 0.200 lanes/beam

- Deck Thickness = 8 in

- Haunch
  Haunch Depth = 19 in encasement height – 17.9 in beam depth (see Steel Member Properties) - 2.125 in cover = -1.025 in. Per the BAR7 Users Manual, "For the bridge type "EIB", enter this distance without a negative sign. The program will assume a negative value for type "EIB". Entering a negative HAUNCH for bridge type "EIB" will indicate that the top of beam is below the bottom of the slab."

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight, deck slab weight, and encasement weight. Girder self-weight and deck slab weight are calculated by BAR7.

  Encasement Weight
  Encasement Weight = (12 in width x 19 in height – 13.91 $in^2$) / 144 $in^2/ft^2$ x 0.150 kcf = 0.223 kips/ft

  Total Dead Load 1 = 0.223 kips/ft

- Dead Loads – DL2
  Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  Barrier Height = 75.5 in outside fascia height – 18" beam depth – 8 in slab = 49.5 in
  Barrier Weight = 13.5 in x 49.5 in / 144 $in^2/ft^2$ x 0.150 kcf = 0.696 kips/ft

  Asphalt Thickness = 10.50 in = 0.875 ft
  Asphalt Wearing Surface = 15.54 ft width x 0.875 ft x 0.140 kcf = 1.904 kips/ft

  Total Dead Load 2 = (2 x 0.696 kips/ft + 1.904 kips/ft) / 5 beams = 0.659 kips/ft

- Concrete Compressive Strength, $f'_c$ = 2.5 ksi per AASHTO MBE Section 6B.5.2.4 for structure being built prior to 1959.
- Modular Ratio, n = 10 based on DM-4 8.2

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = (26.0 ft + 22.417 ft clear / sin 75°) / 2 = 24.60 ft

Steel Member Properties

Field measurements indicated that the concrete encasement depth was 2” shallower than the 20” depth shown on PennDOT Standard Drawing S-624B-75. To account for this difference, an 18 in deep beam was selected for analysis. Based on the bottom flange dimensions of 7 ½” x 7/16” measured in the field, an 18WF/CB181 was selected using AISC Rehabilitation and Retrofit Guide, Table 2.3.1.

**Table 2.3.1 Dimensions and Primary Properties – Steel Sections 1887-1952**

| Designation | Source Reference Number | Wt. per ft | Area | Depth | Web Thickness | Flange Width | Average Flange Thickness | Distance | Distance | Compact Section Criteria | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | A | d | $t_w$ | $b_f$ | $t_f$ | T | k | $b_f/2t_f$ | $h/t_w$ | $Fy^{**}$ |
| | | lb | in.2 | in. | in. | in. | in. | in. | in. | | | ksi |
| 18WF, CB181 | 20 | 47.0 | 13.81 | 17.90 | 0.350 | 7.492 | 0.520 | 15.875 | 1.013 | 7.20 | 45.4 | 31 |

| Designation | $X_1$ | $X_2$ x 106 | Elastic Properties | | | | | | Plastic Modulus | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Axis x-x | | | Axis y-y | | | | |
| | | | $I_x$ | $S_x$ | $r_x$ | $I_y$ | $S_y$ | $r_y$ | $Z_x$ | $Z_y$ |
| | ksi | (1/ksi)2 | in.4 | in.3 | in. | in.4 | in.3 | in. | in.3 | in.3 |
| 18WF, CB181 | 1823 | 15766 | 736.4 | 82.3 | 7.30 | 33.5 | 9.0 | 1.56 | 92.6 | 15.1 |

A recent inspection noted a 14” long x 6” wide exposed region on the bottom flange of beam 3, located approximately 8’ from the far abutment. This exposed region has a 1/16” section loss on the bottom flange. Since there is longitudinally cracking beyond this region, assume the deterioration extends to the far abutment.

- GFS Type = G for Girder
- Span Number = 1
- Range = 24.60 ft – 8 ft = 16.60 ft
- Section Type = W for Wide Flange Beam
- Moment of Inertia = 736.4 $in^4$
- Beam Area = 13.81 $in^2$
- Flange Thickness = 0.52 in
- Flange Width = 7.492 in
- Beam Depth = 17.90 in
- Composite = “N”. This input value is used to differentiate between shored and unshored construction. The example assumes unshored construction; enter "N" so the stresses due to DL1 will be calculated using the non-composite section properties. If shored construction was used, "Y" should be entered and the stresses due to DL1 will be calculated using the composite section properties, (n=3n).
- Web Thickness = 0.35 in
- Yield Strength of Girder = 33 ksi

- GFS Type = G for Girder
- Span Number = 1
- Range = 24.60 ft
- Section Type = W for Wide Flange Beam
- Moment of Inertia = 690 $in^4$ via separate analysis
- Beam Area = 13.81 $in^2$ – 7.492 in x 0.0625 in = 13.34 $in^2$
- Flange Thickness = 0.52 in – 0.0625 in / 2 = 0.48875 in, Use 0.490 in
- Flange Width = 7.492 in

- Beam Depth = 17.90 in – 0.0625 in = 17.8375 in, Use 17.84 in
- Composite = “N”
- Web Thickness = 0.35 in
- Yield Strength of Girder = 33 ksi

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 57-2096-0040-0545 | | |
| Description: | INTERIOR BEAM 3 | | |
| Bridge Type: | EIB | | Concrete Encased I-Beam |
| SLC Level: | | | |
| Lanes: | | | |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | | | Leave Blank |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | | | Leave Blank |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | | | Leave blank and calculate DF |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | | | |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for EIB |
| Overhang or Spacing: | 4.25 | ft | Per Field Sketch |
| CL of Girder: | -- | ft | Leave blank for EIB |
| Roadway Width: | -- | ft | Leave blank for EIB |
| Distr Factor -Shear: | 0.500 | | See Hand Calcs |
| Distr Factor -Moment: | 0.304 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.200 | | See Hand Calcs |
| Slab Thickness: | 8.00 | in | Per plans |
| Haunch: | -1.03 | in | See Hand Calcs |
| Bridge DL1: | 0.223 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.659 | kip/ft | See Hand Calcs |
| F'c: | 2.5 | ksi | Per AASHTO MBE 6B.5.2.4 |
| N: | 10 | | Per DM-4 8.2 |
| Symmetry: | | | |
| LL Location: | | | Only applies to truss |
| Number of Panels: | | | Only applies to truss |
| End Connections: | | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | | | Only applies to truss |
| Stringer DL: | | kip/ft | N/A for EIB |
| Floorbeam DL: | | kip/ft | N/A for EIB |
| Unit weight of Concrete: | | lb/cf | N/A for steel grid |
| Gusset Plate Analysis: | | | N/A for EIB |
| Patch Load Analysis: | | | N/A for EIB |
| Unsymmetrical Pier Support: | | | N/A for EIB |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 24.60 | ft | See Hand Calcs |

| Steel Member Properties | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| GFS | G | G | | | | | | |
| Span Number | 1 | 1 | | | | | | |
| Range | 16.60 | 24.60 | | | | | | |
| Section Type | W | W | | | | | | |
| WF Beam M of I | 736.4 | 690 | | | | | | |
| WF Beam A | 13.81 | 13.34 | | | | | | |
| Flange Thickness | 0.52 | 0.49 | | | | | | |
| Flange Width | 7.492 | 7.492 | | | | | | |
| Varies | | | | | | | | |
| WF Beam Depth | 17.90 | 17.84 | | | | | | |
| Web Thickness | 0.35 | 0.35 | | | | | | |
| Top Plate Width | | | | | | | | |
| Top Plate Thickness | | | | | | | | |
| Bottom Plate Width | | | | | | | | |
| Bottom Plate Thickness | | | | | | | | |
| Composite | N | N | | | | | | |
| Yield Strength | 33 | 33 | | | | | | |
| Hybrid fy Top | | | | | | | | |
| Hybrid fy Bottom | | | | | | | | |

| Lateral Bracing | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Bracing or Stiff Code | | | | | | | | |
| Span Number | | | | | | | | |
| Code 1 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 1 | | | | | | | | |
| Code 2 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 2 | | | | | | | | |
| Code 3 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 3 | | | | | | | | |
| Code 4 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 4 | | | | | | | | |
| Code 5 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 5 | | | | | | | | |
| Code 6 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 6 | | | | | | | | |
| Code 7 | | | | | | | | |
| Num of Spaces 7 | | | | | | | | |
| Spacing 7 | | | | | | | | |
| Code 8 | | | | | | | | |
| Num of Spaces 8 | | | | | | | | |
| Spacing 8 | | | | | | | | |

3.10.8.2.2 BAR7 Output

```
*****************************************************************************
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                     BRIDGE ANALYSIS AND RATING (BAR7)                330427

PROGRAM P4353000                                                06/19/2024 13:44
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: Interior Encased I-Beam BAR7 Analysis.dat




                             PROJECT IDENTIFICATION


STRUCTURE ID - 57209600400545 - INTERIOR BEAM 3

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 EIB                 3   0.00  0.0  0.0                          0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     0.000        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    4.25       0.00        0.00   0.500  0.304  0.200

   SLAB               DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   8.00     -1.03   0.223   0.659   2.500   10.

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00


                         SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     24.60


                             STEEL MEMBER PROPERTIES

  S        T  WF BM    WF BM   FLANGE             WF BM
G P        Y  M OF I    AREA     OR           V  OR WEB
F A        P  OR VRT   OR HRZ  ANGLE  FLANGE  A   PLATE    WEB
S N  RANGE E   LEG      LEG    THICK   WIDTH  R   DEPTH   THICK  SHAPE
G 1  16.60 W   736.40   13.81  0.5200  7.492      17.90  0.3500
              TPW   TPT    BPW   BPT   COMP  FY  FY TOP FY BOT CG TOP CG BOT
              0.00 0.0000  0.00 0.0000   N  33.0   0.0    0.0   0.000  0.000

     RANGE    M OF I    AREA   THICK   WIDTH  V   DEPTH   THICK  SHAPE
G 1  24.60 W   690.00   13.34  0.4900  7.492      17.84  0.3500
              TPW   TPT    BPW   BPT   COMP  FY  FY TOP FY BOT CG TOP CG BOT
```

```
0.00 0.0000  0.00 0.0000   N  33.0   0.0    0.0   0.000  0.000
```

DEFAULT VALUES

| SLC LEVEL | GAGE DISTANCE | PASSING DISTANCE | UNIT WEIGHT DECK | INTEGRAL WEARING SURFACE | SKEW CORR FACTOR |
|---|---|---|---|---|---|
| I | 6.0 | 4.0 | 150.0 | --- | 1.000 |

```
+++++++++++++++++++++++++++++++++++++++++++++
+                                           +
+       G I R D E R   A N A L Y S I S       +
+                                           +
+++++++++++++++++++++++++++++++++++++++++++++
```

DEAD LOADS ACTING ON GIRDER

| INPUT DL1 | GIRDER WEIGHT | SLAB WEIGHT | FL BEAM WEIGHT | STRINGER WEIGHT | FL BEAM DL1 | STRINGER DL1 | TOTAL DL1 | TOTAL DL2 |
|---|---|---|---|---|---|---|---|---|
| 0.223 | 0.046 | 0.425 | 0.000 | 0.000 | 0.000 | 0.000 | 0.694 | 0.659 |

NOTE: IF A SECTION DOES NOT MEET FLANGE OR WEB BUCKLING CRITERIA OF CURRENT AASHTO SPECIFICATIONS FOR LOAD FACTOR METHOD, THE RATING FACTORS ARE REPRINTED AS 888.88. THIS INDICATES THAT THERE IS A POTENTIAL FATIGUE PROBLEM.

GIRDER SECTION PROPERTIES (NON-COMPOSITE)

SPAN 1
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT |
|---|---|---|---|---|---|---|
| 0.00 | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 2.46 | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 4.92 | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 7.38 | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 9.84 | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 12.30 | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 14.76 | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 16.60L | 17.90 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 |
| 16.60R | 17.84 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 |
| 17.22 | 17.84 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 |
| 19.68 | 17.84 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 |
| 22.14 | 17.84 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 |
| 24.60 | 17.84 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 |

GIRDER SECTION PROPERTIES (COMPOSITE, N = 10)

SPAN 1 - EFFECTIVE SLAB WIDTH: 51.00 THICKNESS: 8.00
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S CONC |
|---|---|---|---|---|---|---|---|
| 0.00 | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |
| 2.46 | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |
| 4.92 | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |
| 7.38 | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |
| 9.84 | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |
| 12.30 | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |
| 14.76 | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 16.60L | 24.87 | 54.61 | 2418.25 | 17.91 | -423436.54 | 135.05 | 347.24 |
| 16.60R | 24.81 | 54.14 | 2326.56 | 17.94 | -23144.58 | 129.68 | 338.68 |
| 17.22 | 24.81 | 54.14 | 2326.56 | 17.94 | -23144.58 | 129.68 | 338.68 |
| 19.68 | 24.81 | 54.14 | 2326.56 | 17.94 | -23144.58 | 129.68 | 338.68 |
| 22.14 | 24.81 | 54.14 | 2326.56 | 17.94 | -23144.58 | 129.68 | 338.68 |
| 24.60 | 24.81 | 54.14 | 2326.56 | 17.94 | -23144.58 | 129.68 | 338.68 |

GIRDER SECTION PROPERTIES (COMPOSITE, N = 30)

SPAN 1 - EFFECTIVE SLAB WIDTH: 51.00 THICKNESS: 8.00
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S CONC |
|---|---|---|---|---|---|---|---|
| 0.00 | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 2.46 | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 4.92 | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 7.38 | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 9.84 | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 12.30 | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 14.76 | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 16.60L | 24.87 | 27.41 | 1782.52 | 14.86 | 587.19 | 119.92 | 178.15 |
| 16.60R | 24.81 | 26.94 | 1714.59 | 14.92 | 587.67 | 114.90 | 173.41 |
| 17.22 | 24.81 | 26.94 | 1714.59 | 14.92 | 587.67 | 114.90 | 173.41 |
| 19.68 | 24.81 | 26.94 | 1714.59 | 14.92 | 587.67 | 114.90 | 173.41 |
| 22.14 | 24.81 | 26.94 | 1714.59 | 14.92 | 587.67 | 114.90 | 173.41 |
| 24.60 | 24.81 | 26.94 | 1714.59 | 14.92 | 587.67 | 114.90 | 173.41 |

GIRDER SECTION PROPERTIES (COMPOSITE, NEGATIVE MOMENT)

SPAN 1 - EFFECTIVE SLAB WIDTH: 51.00 THICKNESS: 8.00
=======

| X | DEPTH | AREA | M OF I | C BOT | S TOP | S BOT | S REINF |
|---|---|---|---|---|---|---|---|
| 0.00 | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 2.46 | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 4.92 | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 7.38 | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 9.84 | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 12.30 | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 14.76 | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 16.60L | 24.87 | 13.81 | 736.40 | 8.95 | 82.28 | 82.28 | N/A |
| 16.60R | 24.81 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 | N/A |
| 17.22 | 24.81 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 | N/A |
| 19.68 | 24.81 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 | N/A |
| 22.14 | 24.81 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 | N/A |
| 24.60 | 24.81 | 13.34 | 690.00 | 8.92 | 77.35 | 77.35 | N/A |

```
********************************
*  GIRDER  -  LIVE LOAD  H20   *
********************************
```

MAXIMUM REACTIONS

| | | | | | REACTIONS | | MOMENTS | |
|---|---|---|---|---|---|---|---|---|
| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | +I.F. | -I.F. | +I.F. | -I.F. |
| 1 | 8.5 | 8.1 | 22.2 | 0.0 | 1.30 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 8.5 | 8.1 | 22.2 | 0.0 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 2.46 | 18.9 | 17.9 | 30.6 | 0.0 | | 6.8 | 6.5 | 12.4 | -1.3 | 1.30 |
| | SIMULT | SHEAR | 12.4 | 0.0 | | SIMULT | MOM | 30.6 | 28.0 | |
| 4.92 | 33.6 | 31.9 | 53.4 | 0.0 | | 5.1 | 4.9 | 10.8 | -2.5 | 1.30 |
| | SIMULT | SHEAR | 10.8 | 0.0 | | SIMULT | MOM | 53.4 | 49.8 | |
| 7.38 | 44.1 | 41.9 | 68.4 | 0.0 | | 3.4 | 3.2 | 9.3 | -3.8 | 1.30 |
| | SIMULT | SHEAR | 9.3 | 0.0 | | SIMULT | MOM | 68.4 | 65.3 | |
| 9.84 | 50.4 | 47.9 | 75.6 | 0.0 | | 1.7 | 1.6 | 7.7 | -5.1 | 1.30 |
| | SIMULT | SHEAR | 7.7 | 0.0 | | SIMULT | MOM | 75.6 | 74.7 | |
| 12.30 | 52.5 | 49.9 | 77.8 | 0.0 | | 0.0 | 0.0 | 6.3 | -6.3 | 1.30 |
| | SIMULT | SHEAR | -6.3 | 0.0 | | SIMULT | MOM | 77.8 | 77.8 | |
| 14.76 | 50.4 | 47.9 | 75.6 | 0.0 | | -1.7 | -1.6 | 5.1 | -7.7 | 1.30 |
| | SIMULT | SHEAR | -7.7 | 0.0 | | SIMULT | MOM | 74.7 | 75.6 | |
| 16.60 | 46.1 | 43.8 | 70.9 | 0.0 | | -3.0 | -2.8 | 4.1 | -8.9 | 1.30 |
| | SIMULT | SHEAR | -8.9 | 0.0 | | SIMULT | MOM | 68.3 | 70.9 | |
| 17.22 | 44.1 | 41.9 | 68.4 | 0.0 | | -3.4 | -3.2 | 3.8 | -9.3 | 1.30 |
| | SIMULT | SHEAR | -9.3 | 0.0 | | SIMULT | MOM | 65.3 | 68.4 | |
| 19.68 | 33.6 | 31.9 | 53.4 | 0.0 | | -5.1 | -4.9 | 2.5 | -10.8 | 1.30 |
| | SIMULT | SHEAR | -10.8 | 0.0 | | SIMULT | MOM | 49.8 | 53.4 | |
| 22.14 | 18.9 | 17.9 | 30.6 | 0.0 | | -6.8 | -6.5 | 1.3 | -12.4 | 1.30 |
| | SIMULT | SHEAR | -12.4 | 0.0 | | SIMULT | MOM | 28.0 | 30.6 | |
| 24.60 | 0.0 | 0.0 | 0.0 | 0.0 | | -8.5 | -8.1 | 0.0 | -22.2 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 371.4 B | 352.8 | 112.9 | 1.90 V | 3.17 V | 500.6 | 1.90 V | 3.17 V |
| 2.46 | 371.4 B | 352.8 | 112.9 | 3.55 V | 5.92 V | 500.6 | 3.55 V | 5.92 V |
| 4.92 | 371.4 B | 352.8 | 112.9 | 2.19 B | 3.65 B | 500.6 | 2.94 O | 4.90 O |
| 7.38 | 371.4 B | 352.8 | 112.9 | 1.46 B | 2.43 B | 500.6 | 2.05 O | 3.41 O |
| 9.84 | 371.4 B | 352.8 | 112.9 | 1.18 B | 1.97 B | 500.6 | 1.71 O | 2.86 O |
| 12.30 | 371.4 B | 352.8 | 112.9 | 1.11 B | 1.84 B | 500.6 | 1.62 O | 2.71 O |
| 14.76 | 371.4 B | 352.8 | 112.9 | 1.18 B | 1.97 B | 500.6 | 1.71 O | 2.86 O |
| 16.60 | 356.6 B | 338.8 | 112.9 | 1.25 B | 2.08 B | 487.4 | 1.79 O | 2.99 O |
| 17.22 | 356.6 B | 338.8 | 112.9 | 1.34 B | 2.24 B | 487.4 | 1.91 O | 3.18 O |
| 19.68 | 356.6 B | 338.8 | 112.9 | 2.05 B | 3.41 B | 487.4 | 2.77 O | 4.62 O |
| 22.14 | 356.6 B | 338.8 | 112.9 | 3.55 V | 5.92 V | 487.4 | 3.55 V | 5.92 V |
| 24.60 | 356.6 B | 338.8 | 112.9 | 1.90 V | 3.17 V | 487.4 | 1.90 V | 3.17 V |

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*******************************
*  GIRDER  -  LIVE LOAD  HS20  *
*******************************
```

MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.5 | 8.1 | 26.2 | 0.0 | 1.30 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 8.5 | 8.1 | 26.2 | 0.0 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 2.46 | 18.9 | 17.9 | 38.3 | 0.0 | | 6.8 | 6.5 | 15.6 | -1.3 | 1.30 |
| | SIMULT | SHEAR | 15.6 | 0.0 | | SIMULT | MOM | 38.3 | 28.0 | |
| 4.92 | 33.6 | 31.9 | 64.1 | 0.0 | | 5.1 | 4.9 | 13.0 | -2.5 | 1.30 |
| | SIMULT | SHEAR | 13.0 | 0.0 | | SIMULT | MOM | 64.1 | 49.8 | |
| 7.38 | 44.1 | 41.9 | 77.5 | 0.0 | | 3.4 | 3.2 | 10.5 | -3.8 | 1.30 |
| | SIMULT | SHEAR | 10.5 | 0.0 | | SIMULT | MOM | 77.5 | 65.3 | |
| 9.84 | 50.4 | 47.9 | 78.5 | 0.0 | | 1.7 | 1.6 | 8.0 | -5.1 | 1.30 |
| | SIMULT | SHEAR | 8.0 | 0.0 | | SIMULT | MOM | 78.5 | 74.7 | |
| 12.30 | 52.5 | 49.9 | 77.8 | 0.0 | | 0.0 | 0.0 | 6.3 | -6.3 | 1.30 |
| | SIMULT | SHEAR | -6.3 | 0.0 | | SIMULT | MOM | 77.8 | 77.8 | |
| 14.76 | 50.4 | 47.9 | 78.5 | 0.0 | | -1.7 | -1.6 | 5.1 | -8.0 | 1.30 |
| | SIMULT | SHEAR | -8.0 | 0.0 | | SIMULT | MOM | 74.7 | 78.5 | |
| 16.60 | 46.1 | 43.8 | 79.0 | 0.0 | | -3.0 | -2.8 | 4.1 | -9.9 | 1.30 |
| | SIMULT | SHEAR | -9.9 | 0.0 | | SIMULT | MOM | 68.3 | 79.0 | |
| 17.22 | 44.1 | 41.9 | 77.5 | 0.0 | | -3.4 | -3.2 | 3.8 | -10.5 | 1.30 |
| | SIMULT | SHEAR | -10.5 | 0.0 | | SIMULT | MOM | 65.3 | 77.5 | |
| 19.68 | 33.6 | 31.9 | 64.1 | 0.0 | | -5.1 | -4.9 | 2.5 | -13.0 | 1.30 |
| | SIMULT | SHEAR | -13.0 | 0.0 | | SIMULT | MOM | 49.8 | 64.1 | |
| 22.14 | 18.9 | 17.9 | 38.3 | 0.0 | | -6.8 | -6.5 | 1.3 | -15.6 | 1.30 |
| | SIMULT | SHEAR | -15.6 | 0.0 | | SIMULT | MOM | 28.0 | 38.3 | |
| 24.60 | 0.0 | 0.0 | 0.0 | 0.0 | | -8.5 | -8.1 | 0.0 | -26.2 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 371.4 B | 352.8 | 112.9 | 1.61 V | 2.68 V | 500.6 | 1.61 V | 2.68 V |
| 2.46 | 371.4 B | 352.8 | 112.9 | 2.84 V | 4.73 V | 500.6 | 2.84 V | 4.73 V |
| 4.92 | 371.4 B | 352.8 | 112.9 | 1.82 B | 3.03 B | 500.6 | 2.45 O | 4.08 O |
| 7.38 | 371.4 B | 352.8 | 112.9 | 1.29 B | 2.14 B | 500.6 | 1.80 O | 3.01 O |
| 9.84 | 371.4 B | 352.8 | 112.9 | 1.14 B | 1.90 B | 500.6 | 1.65 O | 2.75 O |
| 12.30 | 371.4 B | 352.8 | 112.9 | 1.11 B | 1.84 B | 500.6 | 1.62 O | 2.71 O |
| 14.76 | 371.4 B | 352.8 | 112.9 | 1.14 B | 1.90 B | 500.6 | 1.65 O | 2.75 O |
| 16.60 | 356.6 B | 338.8 | 112.9 | 1.12 B | 1.87 B | 487.4 | 1.61 O | 2.69 O |
| 17.22 | 356.6 B | 338.8 | 112.9 | 1.18 B | 1.97 B | 487.4 | 1.68 O | 2.81 O |
| 19.68 | 356.6 B | 338.8 | 112.9 | 1.70 B | 2.84 B | 487.4 | 2.31 O | 3.84 O |
| 22.14 | 356.6 B | 338.8 | 112.9 | 2.84 V | 4.73 V | 487.4 | 2.84 V | 4.73 V |
| 24.60 | 356.6 B | 338.8 | 112.9 | 1.61 V | 2.68 V | 487.4 | 1.61 V | 2.68 V |

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*******************************
*  GIRDER  -  LIVE LOAD TK527  *
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MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.5 | 8.1 | 25.8 | 0.0 | 1.30 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 8.5 | 8.1 | 25.8 | 0.0 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 2.46 | 18.9 | 17.9 | 43.4 | 0.0 | | 6.8 | 6.5 | 17.6 | -0.8 | 1.30 |
| | SIMULT | SHEAR | 17.6 | 0.0 | | SIMULT | MOM | 43.4 | 18.0 | |
| 4.92 | 33.6 | 31.9 | 72.6 | 0.0 | | 5.1 | 4.9 | 14.8 | -1.9 | 1.30 |
| | SIMULT | SHEAR | 14.8 | 0.0 | | SIMULT | MOM | 72.6 | 38.0 | |
| 7.38 | 44.1 | 41.9 | 89.6 | 0.0 | | 3.4 | 3.2 | 12.1 | -3.6 | 1.30 |
| | SIMULT | SHEAR | 12.1 | 0.0 | | SIMULT | MOM | 89.6 | 61.3 | |
| 9.84 | 50.4 | 47.9 | 103.0 | 0.0 | | 1.7 | 1.6 | 9.7 | -5.4 | 1.30 |
| | SIMULT | SHEAR | 5.6 | 0.0 | | SIMULT | MOM | 95.5 | 80.2 | |
| 12.30 | 52.5 | 49.9 | 104.9 | 0.0 | | 0.0 | 0.0 | 7.4 | -7.4 | 1.30 |
| | SIMULT | SHEAR | -5.1 | 0.0 | | SIMULT | MOM | 91.3 | 91.3 | |
| 14.76 | 50.4 | 47.9 | 103.0 | 0.0 | | -1.7 | -1.6 | 5.4 | -9.7 | 1.30 |
| | SIMULT | SHEAR | -5.6 | 0.0 | | SIMULT | MOM | 80.2 | 95.5 | |
| 16.60 | 46.1 | 43.8 | 93.9 | 0.0 | | -3.0 | -2.8 | 4.0 | -11.5 | 1.30 |
| | SIMULT | SHEAR | -7.7 | 0.0 | | SIMULT | MOM | 65.9 | 91.9 | |
| 17.22 | 44.1 | 41.9 | 89.6 | 0.0 | | -3.4 | -3.2 | 3.6 | -12.1 | 1.30 |
| | SIMULT | SHEAR | -12.1 | 0.0 | | SIMULT | MOM | 61.3 | 89.6 | |
| 19.68 | 33.6 | 31.9 | 72.6 | 0.0 | | -5.1 | -4.9 | 1.9 | -14.8 | 1.30 |
| | SIMULT | SHEAR | -14.8 | 0.0 | | SIMULT | MOM | 38.0 | 72.6 | |
| 22.14 | 18.9 | 17.9 | 43.4 | 0.0 | | -6.8 | -6.5 | 0.8 | -17.6 | 1.30 |
| | SIMULT | SHEAR | -17.6 | 0.0 | | SIMULT | MOM | 18.0 | 43.4 | |
| 24.60 | 0.0 | 0.0 | 0.0 | 0.0 | | -8.5 | -8.1 | 0.0 | -25.8 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 371.4 B | 352.8 | 112.9 | 1.63 V | 2.72 V | 500.6 | 1.63 V | 2.72 V |
| 2.46 | 371.4 B | 352.8 | 112.9 | 2.50 V | 4.17 V | 500.6 | 2.50 V | 4.17 V |
| 4.92 | 371.4 B | 352.8 | 112.9 | 1.61 B | 2.68 B | 500.6 | 2.16 O | 3.61 O |
| 7.38 | 371.4 B | 352.8 | 112.9 | 1.11 B | 1.85 B | 500.6 | 1.56 O | 2.60 O |
| 9.84 | 371.4 B | 352.8 | 112.9 | 0.87 B | 1.45 B | 500.6 | 1.26 O | 2.10 O |
| 12.30 | 371.4 B | 352.8 | 112.9 | 0.82 B | 1.37 B | 500.6 | 1.20 O | 2.01 O |
| 14.76 | 371.4 B | 352.8 | 112.9 | 0.87 B | 1.45 B | 500.6 | 1.26 O | 2.10 O |
| 16.60 | 356.6 B | 338.8 | 112.9 | 0.94 B | 1.57 B | 487.4 | 1.36 O | 2.26 O |
| 17.22 | 356.6 B | 338.8 | 112.9 | 1.02 B | 1.71 B | 487.4 | 1.46 O | 2.43 O |
| 19.68 | 356.6 B | 338.8 | 112.9 | 1.50 B | 2.51 B | 487.4 | 2.04 O | 3.40 O |
| 22.14 | 356.6 B | 338.8 | 112.9 | 2.50 V | 4.17 V | 487.4 | 2.50 V | 4.17 V |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 24.60 | 356.6 B | 338.8 | 112.9 | 1.63 V | 2.72 V | 487.4 | 1.63 V | 2.72 V |

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*******************************
*  GIRDER  -  LIVE LOAD  ML80  *
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MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.5 | 8.1 | 27.2 | 0.0 | 1.30 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 8.5 | 8.1 | 27.2 | 0.0 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 2.46 | 18.9 | 17.9 | 46.5 | 0.0 | | 6.8 | 6.5 | 18.9 | -0.8 | 1.30 |
| | SIMULT | SHEAR | 18.9 | 0.0 | | SIMULT | MOM | 46.5 | 18.0 | |
| 4.92 | 33.6 | 31.9 | 78.4 | 0.0 | | 5.1 | 4.9 | 15.9 | -1.9 | 1.30 |
| | SIMULT | SHEAR | 15.9 | 0.0 | | SIMULT | MOM | 78.4 | 38.0 | |
| 7.38 | 44.1 | 41.9 | 98.8 | 0.0 | | 3.4 | 3.2 | 13.1 | -3.6 | 1.30 |
| | SIMULT | SHEAR | 9.7 | 0.0 | | SIMULT | MOM | 96.9 | 61.3 | |
| 9.84 | 50.4 | 47.9 | 113.3 | 0.0 | | 1.7 | 1.6 | 10.7 | -5.8 | 1.30 |
| | SIMULT | SHEAR | 6.7 | 0.0 | | SIMULT | MOM | 105.1 | 85.6 | |
| 12.30 | 52.5 | 49.9 | 117.6 | 0.0 | | 0.0 | 0.0 | 8.2 | -8.2 | 1.30 |
| | SIMULT | SHEAR | 4.1 | 0.0 | | SIMULT | MOM | 101.4 | 101.4 | |
| 14.76 | 50.4 | 47.9 | 113.3 | 0.0 | | -1.7 | -1.6 | 5.8 | -10.7 | 1.30 |
| | SIMULT | SHEAR | -6.7 | 0.0 | | SIMULT | MOM | 85.6 | 105.1 | |
| 16.60 | 46.1 | 43.8 | 103.9 | 0.0 | | -3.0 | -2.8 | 4.0 | -12.5 | 1.30 |
| | SIMULT | SHEAR | -8.9 | 0.0 | | SIMULT | MOM | 65.9 | 100.1 | |
| 17.22 | 44.1 | 41.9 | 98.8 | 0.0 | | -3.4 | -3.2 | 3.6 | -13.1 | 1.30 |
| | SIMULT | SHEAR | -9.7 | 0.0 | | SIMULT | MOM | 61.3 | 96.9 | |
| 19.68 | 33.6 | 31.9 | 78.4 | 0.0 | | -5.1 | -4.9 | 1.9 | -15.9 | 1.30 |
| | SIMULT | SHEAR | -15.9 | 0.0 | | SIMULT | MOM | 38.0 | 78.4 | |
| 22.14 | 18.9 | 17.9 | 46.5 | 0.0 | | -6.8 | -6.5 | 0.8 | -18.9 | 1.30 |
| | SIMULT | SHEAR | -18.9 | 0.0 | | SIMULT | MOM | 18.0 | 46.5 | |
| 24.60 | 0.0 | 0.0 | 0.0 | 0.0 | | -8.5 | -8.1 | 0.0 | -27.2 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 371.4 B | 352.8 | 112.9 | 1.55 V | 2.59 V | 500.6 | 1.55 V | 2.59 V |
| 2.46 | 371.4 B | 352.8 | 112.9 | 2.33 V | 3.89 V | 500.6 | 2.33 V | 3.89 V |
| 4.92 | 371.4 B | 352.8 | 112.9 | 1.49 B | 2.48 B | 500.6 | 2.00 O | 3.34 O |
| 7.38 | 371.4 B | 352.8 | 112.9 | 1.01 B | 1.68 B | 500.6 | 1.42 O | 2.36 O |
| 9.84 | 371.4 B | 352.8 | 112.9 | 0.79 B | 1.32 B | 500.6 | 1.14 O | 1.91 O |

```
 12.30    371.4 B   352.8   112.9   0.73 B   1.22 B    500.6   1.07 O   1.79 O
 14.76    371.4 B   352.8   112.9   0.79 B   1.32 B    500.6   1.14 O   1.91 O
 16.60    356.6 B   338.8   112.9   0.85 B   1.42 B    487.4   1.23 O   2.04 O
 17.22    356.6 B   338.8   112.9   0.93 B   1.55 B    487.4   1.32 O   2.20 O
 19.68    356.6 B   338.8   112.9   1.39 B   2.32 B    487.4   1.89 O   3.14 O
 22.14    356.6 B   338.8   112.9   2.33 V   3.89 V    487.4   2.33 V   3.89 V
 24.60    356.6 B   338.8   112.9   1.55 V   2.59 V    487.4   1.55 V   2.59 V


                +++++++++++++++++++++++++++++++++++++++++++++
                +                                           +
                +         R A T I N G   S U M M A R Y       +
                +                                           +
                +++++++++++++++++++++++++++++++++++++++++++++

MEMBER: GIRDER
                             LOAD FACTOR RATING
  LOAD                 FACTOR     TONS     X   SPAN
  H20  IR (CRITICAL)     1.62 O    32.5    12.30  1
       OR (CRITICAL)     2.71 O    54.1    12.30  1
       IR ( POS MOM)     1.62 O    32.5    12.30  1
       OR ( POS MOM)     2.71 O    54.1    12.30  1
  HS20 IR (CRITICAL)     1.61 V    57.8    24.60  1
       OR (CRITICAL)     2.68 V    96.3    24.60  1
       IR ( POS MOM)     1.61 O    58.0    16.60  1
       OR ( POS MOM)     2.69 O    96.7    16.60  1
 TK527 IR (CRITICAL)     1.20 O    48.2    12.30  1
       OR (CRITICAL)     2.01 O    80.3    12.30  1
       IR ( POS MOM)     1.20 O    48.2    12.30  1
       OR ( POS MOM)     2.01 O    80.3    12.30  1
  ML80 IR (CRITICAL)     1.07 O    39.3    12.30  1
       OR (CRITICAL)     1.79 O    65.6    12.30  1
       IR ( POS MOM)     1.07 O    39.3    12.30  1
       OR ( POS MOM)     1.79 O    65.6    12.30  1

  NOTE: THIS RATING SUMMARY OF GIRDERS DOES FOLLOW THE ALL-OR-NONE
        COMPACT REQUIREMENTS.
        THIS GIRDER DOES HAVE ALL SECTIONS QUALIFYING AS COMPACT
        SECTIONS.  THEREFORE, THE RATING PRINTED IN THE SUMMARY
        IS THAT BASED ON COMPACT SECTIONS

      RATING FACTOR CODES: FIRST CHARACTER AFTER THE RATING FACTOR
      T - TOP STEEL STRESS/STRENGTH GOVERNS
      B - BOTTOM STEEL STRESS/STRENGTH GOVERNS
      C - CONCRETE STRESS/STRENGTH GOVERNS
      R - REINFORCEMENT STRESS/STRENGTH GOVERNS
      V - SHEAR STRESS/STRENGTH GOVERNS
      blank - COMPACT MOMENT STRENGTH GOVERNS
      O - OVERLOAD PROVISIONS GOVERN
      I - MOMENT-SHEAR INTERACTION GOVERNS

      RATING FACTOR CODES FOR FLANGE/WEB BUCKLING: SECOND CHARACTER AFTER THE RATING
FACTOR
      blank - SECTION DOES MEET FLANGE/WEB BUCKLING CRITERIA
          AND WILL NOT BUCKLE.
      F - SECTION DOES NOT MEET FLANGE PROJECTION/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE.
      W - SECTION DOES NOT MEET WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN WEB.
      Z - SECTION DOES NOT MEET BOTH FLANGE PROJECTION/THICKNESS RATIO
          CRITERIA AND WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE OR WEB.
```

```
      NON-COMPACT MOMENT STRENGTH CODES:
      B - SECTION IS BRACED
      U - SECTION IS UNBRACED

  NOTE: ALL RATINGS ARE BASED ON THE NUMBER OF DESIGN LANES OR THE ACTUAL
        TRAFFIC LANES AS DEFINED BY "D" OR "L" ENTERED FOR LANES IN THE
        PROJECT IDENTIFICATION.

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

3.10.8.3 Exterior Beam Load Rating Analysis

3.10.8.3.1 BAR7 Input Parameters

Exterior beam 5 was selected and will be rated using BAR7. Some of the BAR7 input parameters can be left blank if the default values are correct. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 57209600400545
- Description = EXTERIOR BEAM 5
- Bridge Type = EIB
- Live Load = Blank for H20, HS20, ML80, TK527. Enter L for EV2 and EV3 vehicles.
- Output = 3 for Load Factor Ratings
- Concrete Deck = Blank
- Skew Correction Factor = 1.093 (See below)

  Per 1993 DM-4 Table 3.23.2(A), the Skew Correction Factor is equal to

$$\text{SCF} = 1.0 + 0.2\left(\frac{12Lt_s^3}{K_g}\right)^{0.3} \tan\,\theta$$

where,

L = Span Length = 24.60 ft
$t_s$ = Deck Thickness = 8 in
θ = AASHTO Skew Angle = (90° – 75°) = 15° (skew angle at obtuse corner of bridge)

$e_g$ = Distance between the c.g. of the deck and c.g. of the beam = 8.95 in + 1.975 in = 10.925 in
Beam c.g. from Top of Beam = 17.9 in / 2 = 8.95 in
Deck c.g. from Top of Beam = 8 in / 2 – 2.025 in (haunch) = 1.975 in

n = Modular Ratio Between the Deck and Beam = 29,000 ksi / 2850 ksi = 10.175
Steel Modulus of Elasticity, Es = 29,000 ksi
Concrete Compressive Strength, $f'_c$ = 2.5 ksi (AASHTO MBE Section 6B.5.2.4 - Prior to 1959)
Concrete Modulus of Elasticty, Ec = 57,000(f'c)0.5 = 57,000(2500 psi)0.5/1000 = 2850 ksi

I = Moment of Inertia of the Steel I-Beam = 736 $in^4$

A = Beam Area = 13.81 $in^2$

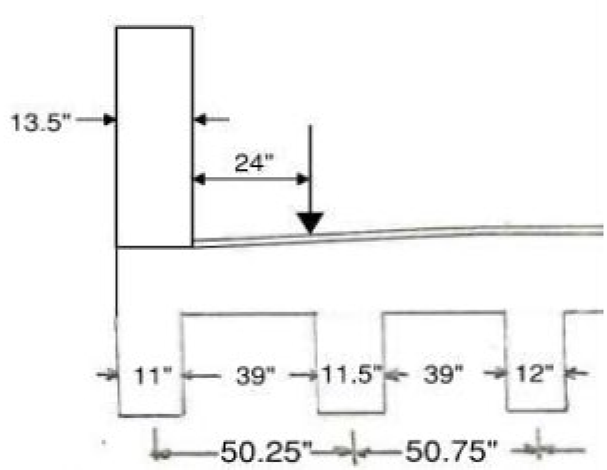
$K_g$ = Longitudinal Stiffness Parameter = $n(I+Ae_g^2)$ = 10.175[736 $in^4$ + 13.81 $in^2$ (10.925 in)$^2$] = 24260 $in^4$

The Range of Applicability checks per 1993 DM-4 Table 3.23.2(A):

| | |
|---|---|
| 0° ≤ θ ≤ 60° | θ = 15°, OK |
| 3.5 ft ≤ S ≤ 16 ft | S = 4.25 ft, OK |
| 20 ft ≤ L ≤ 240 ft | L = 24.60 ft, OK |
| $N_b$ ≥ 4 | $N_b$ = 5, OK |

Bridge Cross Section and Loading

- Spacing = 50.25 in /2 + 11 in / 2 = 30.625 in = 2.55 ft per field sketch




- Distribution Factor – Shear

  Per AASHTO 3.23.1.2, the exterior beam distribution factor for shear is based on the Lever Rule. Since the beam spacing is less than 6 feet, only 1 wheel can be considered.
  $DF_V$ = P x (50.25 in + 11 in / 2 - 13.5 in - 24 in) / 50.25 in = 0.363 wheels
  $DF_V$ = 0.363 wheels / (2 wheels/axle) = 0.182 axles

- Distribution Factor – Moment

  Per AASHTO 3.23.2.3.1.2, the exterior beam distribution factor for moment is based on the Lever Rule. Since the beam spacing is less than 6 feet, only 1 wheel can be considered.
  $DF_M$ = P x (50.25 in + 11 in / 2 - 13.5 in - 24 in) / 50.25 in = 0.363 wheels
  $DF_M$ = 0.363 wheels / (2 wheels/axle) = 0.182 axles

- Distribution Factor – Deflection
  Per AASHTO 10.6.4, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
  Roadway Width = 15.5 ft per Inspection Sketch
  $N_L$ = 15.5 ft / 12 ft = 1.29 ==> Use 1 lane
  Reduction Factor = 1.0 for 1 lane per AASHTO 3.12.1
  $DF_{Defl}$ = 1 lane x 1.0 / 5 beams = 0.200 lanes/beam

- Deck Thickness = 8 in
- Haunch
  Haunch Depth = 18 in encasement height – 17.9 in beam depth - 2.125 in cover = -2.025 in. Per the BAR7 Users Manual, "For the bridge type "EIB", enter this distance without a negative sign. The program will assume a negative value for type "EIB". Entering a negative haunch for bridge type "EIB" will indicate that the top of beam is below the bottom of the slab."

- Dead Loads – DL1
  The dead loads acting on the non-composite section include girder self-weight, deck slab weight, and encasement weight. Girder self-weight and deck slab weight are calculated by BAR7.

  Encasement Weight
  Encasement Weight = (12 in width x 18 in height – 13.91 $in^2$) / 144 $in^2/ft^2$ x 0.150 kcf = 0.211 kips/ft

  Total Dead Load 1 = 0.211 kips/ft

- Dead Loads – DL2
  Per AASHTO 3.23.2.3.1.1, loads placed after the deck slab has cured may be distributed equally to all the beams. Therefore, **DL2 loads should be evenly distributed to all girders**.

  Barrier Height = 75.5 in outside fascia height – 18" beam depth – 8 in slab = 49.5 in
  Barrier Weight = 13.5 in x 49.5 in / 144 $in^2/ft^2$ x 0.150 kcf = 0.696 kips/ft

  Asphalt Thickness = 10.50 in = 0.875 ft
  Asphalt Wearing Surface = 15.54 ft width x 0.875 ft x 0.140 kcf = 1.904 kips/ft

  Total Dead Load 2 = (2 x 0.696 kips/ft + 1.904 kips/ft) / 5 beams = 0.659 kips/ft

- Concrete Compressive Strength, $f'_c$ = 2.5 ksi per AASHTO MBE Section 6B.5.2.4 for structure being built prior to 1959.
- Modular Ratio, n = 10 based on DM-4 8.2

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = (26.0 ft + 22.417 ft clear / sin 75°) / 2 = 24.60 ft

Steel Member Properties

- GFS Type = G for Girder
- Span Number = 1
- Range = 24.60 ft
- Section Type = W for Wide Flange Beam
- Moment of Inertia = 736.4 $in^4$
- Beam Area = 13.81 $in^2$
- Flange Thickness = 0.52 in
- Flange Width = 7.492 in
- Beam Depth = 17.90 in
- Composite = "N" for Non-Composite
- Web Thickness = 0.35 in
- Yield Strength of Girder = 33 ksi

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 57-2096-0040-0545 | | |
| Description: | EXTERIOR BEAM 3 | | |
| Bridge Type: | EIB | | Concrete Encased I-Beam |
| SLC Level: | | | |
| Lanes: | | | |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | | | Leave Blank |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | | | Leave Blank |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | | | Leave blank and calculate DF |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | 1.093 | | See Hand Calcs |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | -- | ft | Leave blank for EIB |
| Overhang or Spacing: | 2.55 | ft | Per Field Sketch |
| CL of Girder: | -- | ft | Leave blank for EIB |
| Roadway Width: | -- | ft | Leave blank for EIB |
| Distr Factor -Shear: | 0.182 | | See Hand Calcs |
| Distr Factor -Moment: | 0.182 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.200 | | See Hand Calcs |
| Slab Thickness: | 8.00 | in | Per plans |
| Haunch: | -2.03 | in | See Hand Calcs |
| Bridge DL1: | 0.211 | kip/ft | See Hand Calcs |
| Bridge DL2: | 0.659 | kip/ft | See Hand Calcs |
| F'c: | 2.5 | ksi | Per AASHTO MBE 6B.5.2.4 |
| N: | 10 | | Per DM-4 8.2 |
| Symmetry: | | | |
| LL Location: | | | Only applies to truss |
| Number of Panels: | | | Only applies to truss |
| End Connections: | | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | | | Only applies to truss |
| Stringer DL: | | kip/ft | N/A for EIB |
| Floorbeam DL: | | kip/ft | N/A for EIB |
| Unit weight of Concrete: | | lb/cf | N/A for steel grid |
| Gusset Plate Analysis: | | | N/A for EIB |
| Patch Load Analysis: | | | N/A for EIB |
| Unsymmetrical Pier Support: | | | N/A for EIB |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 24.60 | ft | See Hand Calcs |

| Steel Member Properties | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| GFS | G | | | | | | | |
| Span Number | 1 | | | | | | | |
| Range | 24.60 | | | | | | | |
| Section Type | W | | | | | | | |
| WF Beam M of I | 736.4 | | | | | | | |
| WF Beam A | 13.81 | | | | | | | |
| Flange Thickness | 0.52 | | | | | | | |
| Flange Width | 7.492 | | | | | | | |
| Varies | | | | | | | | |
| WF Beam Depth | 17.90 | | | | | | | |
| Web Thickness | 0.35 | | | | | | | |
| Top Plate Width | | | | | | | | |
| Top Plate Thickness | | | | | | | | |
| Bottom Plate Width | | | | | | | | |
| Bottom Plate Thickness | | | | | | | | |
| Composite | N | | | | | | | |
| Yield Strength | 33 | | | | | | | |
| Hybrid fy Top | | | | | | | | |
| Hybrid fy Bottom | | | | | | | | |

| Lateral Bracing | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Bracing or Stiff Code | | | | | | | | |
| Span Number | | | | | | | | |
| Code 1 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 1 | | | | | | | | |
| Code 2 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 2 | | | | | | | | |
| Code 3 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 3 | | | | | | | | |
| Code 4 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 4 | | | | | | | | |
| Code 5 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 5 | | | | | | | | |
| Code 6 | | | | | | | | |
| Num of Spaces | | | | | | | | |
| Spacing 6 | | | | | | | | |
| Code 7 | | | | | | | | |
| Num of Spaces 7 | | | | | | | | |
| Spacing 7 | | | | | | | | |
| Code 8 | | | | | | | | |
| Num of Spaces 8 | | | | | | | | |
| Spacing 8 | | | | | | | | |

3.10.8.3.2 BAR7 Output

```
*****************************************************************************
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*****************************************************************************
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                    BRIDGE ANALYSIS AND RATING (BAR7)                330427

PROGRAM P4353000                                              06/19/2024 14:23
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: Exterior Encased I-Beam BAR7 Analysis.dat




                            PROJECT IDENTIFICATION


STRUCTURE ID - 57209600400545 - EXTERIOR BEAM 5

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 EIB                3   0.00  0.0  0.0                           0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     1.093        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
 0.00    2.55       0.00        0.00   0.182  0.182  0.200

   SLAB               DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   8.00     -2.03   0.211   0.659   2.500   10.

STRINGER   FLOORBEAM  UNIT WEIGHT  PATCH LOAD UNSYMM PIER
  DL1         DL1    DECK CONCRETE  ANALYSIS    SUPPORT
 0.000       0.000        0.00


                         SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     24.60


                            STEEL MEMBER PROPERTIES

  S        T  WF BM    WF BM   FLANGE            WF BM
G P        Y  M OF I    AREA     OR           V  OR WEB
F A        P  OR VRT   OR HRZ  ANGLE  FLANGE  A   PLATE    WEB
S N  RANGE E   LEG      LEG    THICK   WIDTH  R   DEPTH   THICK  SHAPE
G 1  24.60 W   736.40   13.81  0.5200  7.492      17.90  0.3500
              TPW   TPT    BPW   BPT    COMP  FY  FY TOP FY BOT CG TOP CG BOT
              0.00 0.0000  0.00 0.0000   N  33.0   0.0    0.0   0.000  0.000


                                   DEFAULT VALUES
```

```
                                UNIT    INTEGRAL
  SLC      GAGE      PASSING   WEIGHT   WEARING   SKEW CORR
 LEVEL   DISTANCE   DISTANCE    DECK    SURFACE    FACTOR
   I        6.0        4.0     150.0      ---      -----


                ++++++++++++++++++++++++++++++++++++++++++++++
                +                                            +
                +       G I R D E R   A N A L Y S I S        +
                +                                            +
                ++++++++++++++++++++++++++++++++++++++++++++++


                        DEAD LOADS ACTING ON GIRDER

 INPUT  GIRDER   SLAB   FL BEAM  STRINGER  FL BEAM STRINGER   TOTAL   TOTAL
  DL1   WEIGHT  WEIGHT   WEIGHT   WEIGHT     DL1      DL1       DL1     DL2
 0.211   0.047   0.255    0.000    0.000    0.000    0.000     0.513   0.659

NOTE: IF A SECTION DOES NOT MEET FLANGE OR WEB BUCKLING CRITERIA
      OF CURRENT AASHTO SPECIFICATIONS FOR LOAD FACTOR METHOD, THE
      RATING FACTORS ARE REPRINTED AS 888.88.  THIS INDICATES
      THAT THERE IS A POTENTIAL FATIGUE PROBLEM.


                           GIRDER SECTION PROPERTIES

SPAN  1 - EFFECTIVE SLAB WIDTH:  30.60  THICKNESS:  8.00
=======

                                                        SECTION MODULUS
                                 GROSS   MOMENT OF    C                     CONC OR
                         DEPTH    AREA    INERTIA  BOTTOM    TOP     BOTTOM NEG REINF
NON-COMPOSITE            17.90   13.81     736.40   8.95    82.28    82.28
COMPOSITE (N=10)         23.87   38.29    1919.81  15.93   975.34   120.50   241.84
COMPOSITE (N=30)         23.87   21.97    1391.56  13.01   284.33   107.00   128.09


                       ********************************
                       *  GIRDER  -  LIVE LOAD  H20   *
                       ********************************

                               MAXIMUM REACTIONS

                                                 REACTIONS     MOMENTS
    SUPPORT    DL1      DL2   +(LL+I)     -(LL+I)  +I.F. -I.F.  +I.F. -I.F.
       1       6.3      8.1      8.4         0.0    1.30


                        UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======
            DL1     DL2  +(LL+I)  -(LL+I)  I   DL1    DL2  +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT    MOMENT  M  SHEAR  SHEAR  SHEAR   SHEAR  I.F.
    0.00    0.0     0.0    0.0      0.0        6.3    8.1    9.2     0.0  1.30
            SIMULT SHEAR   0.0      0.0       SIMULT MOM     0.0     0.0
    2.46   14.0    17.9   18.3      0.0        5.0    6.5    8.1    -0.8  1.30
            SIMULT SHEAR   8.1      0.0       SIMULT MOM    18.3    16.8
    4.92   24.8    31.9   32.0      0.0        3.8    4.9    7.1    -1.7  1.30
            SIMULT SHEAR   7.1      0.0       SIMULT MOM    32.0    29.8
    7.38   32.6    41.9   40.9      0.0        2.5    3.2    6.1    -2.5  1.30
```

```
            SIMULT SHEAR    6.1       0.0       SIMULT MOM    40.9    39.1
    9.84    37.3    47.9   45.3       0.0        1.3    1.6    5.0    -3.3  1.30
            SIMULT SHEAR    5.0       0.0       SIMULT MOM    45.3    44.7
   12.30    38.8    49.9   46.6       0.0        0.0    0.0    4.1    -4.1  1.30
            SIMULT SHEAR   -4.1       0.0       SIMULT MOM    46.6    46.6
   14.76    37.3    47.9   45.3       0.0       -1.3   -1.6    3.3    -5.0  1.30
            SIMULT SHEAR   -5.0       0.0       SIMULT MOM    44.7    45.3
   17.22    32.6    41.9   40.9       0.0       -2.5   -3.2    2.5    -6.1  1.30
            SIMULT SHEAR   -6.1       0.0       SIMULT MOM    39.1    40.9
   19.68    24.8    31.9   32.0       0.0       -3.8   -4.9    1.7    -7.1  1.30
            SIMULT SHEAR   -7.1       0.0       SIMULT MOM    29.8    32.0
   22.14    14.0    17.9   18.3       0.0       -5.0   -6.5    0.8    -8.1  1.30
            SIMULT SHEAR   -8.1       0.0       SIMULT MOM    16.8    18.3
   24.60     0.0     0.0    0.0       0.0       -6.3   -8.1    0.0    -9.2  1.30
            SIMULT SHEAR    0.0       0.0       SIMULT MOM     0.0     0.0


NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR
      UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER
      SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.


                     STRENGTHS AND LOAD FACTOR RATINGS

SPAN  1
=======
        NON-COMP OVERLOAD           NON-COMPACT     COMPACT      COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS    MOMENT   RATING FACTORS
   X    STRENGTH STRENGTH STRENGTH   IR        OR    STRENGTH    IR        OR
  0.00    331.4 B   314.8   112.9   4.74 V   7.91 V    390.7   4.74 V   7.91 V
  2.46    331.4 B   314.8   112.9   5.56 V   9.27 V    390.7   5.56 V   9.27 V
  4.92    331.4 B   314.8   112.9   3.43 B   5.72 B    390.7   4.55 O   7.59 O
  7.38    331.4 B   314.8   112.9   2.34 B   3.91 B    390.7   3.22 O   5.37 O
  9.84    331.4 B   314.8   112.9   1.94 B   3.23 B    390.7   2.73 O   4.56 O
 12.30    331.4 B   314.8   112.9   1.83 B   3.05 B    390.7   2.60 O   4.33 O
 14.76    331.4 B   314.8   112.9   1.94 B   3.23 B    390.7   2.73 O   4.56 O
 17.22    331.4 B   314.8   112.9   2.34 B   3.91 B    390.7   3.22 O   5.37 O
 19.68    331.4 B   314.8   112.9   3.43 B   5.72 B    390.7   4.55 O   7.59 O
 22.14    331.4 B   314.8   112.9   5.56 V   9.27 V    390.7   5.56 V   9.27 V
 24.60    331.4 B   314.8   112.9   4.74 V   7.91 V    390.7   4.74 V   7.91 V


                       *******************************
                       *  GIRDER  -  LIVE LOAD  HS20  *
                       *******************************

                                  MAXIMUM REACTIONS

                                                        REACTIONS     MOMENTS
    SUPPORT     DL1       DL2    +(LL+I)     -(LL+I)   +I.F. -I.F.  +I.F. -I.F.
       1        6.3       8.1     10.8         0.0     1.30


                         UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======
            DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT    MOMENT   M  SHEAR   SHEAR  SHEAR    SHEAR  I.F.
    0.00    0.0     0.0    0.0       0.0        6.3    8.1    11.8     0.0  1.30
            SIMULT SHEAR    0.0       0.0       SIMULT MOM     0.0     0.0
    2.46    14.0    17.9   22.9       0.0        5.0    6.5    10.2    -0.8  1.30
            SIMULT SHEAR   10.2       0.0       SIMULT MOM    22.9    16.8
```

```
    4.92     24.8    31.9   38.4      0.0       3.8    4.9    8.5    -1.7  1.30
             SIMULT SHEAR    8.5      0.0      SIMULT MOM    38.4    29.8
    7.38     32.6    41.9   46.4      0.0       2.5    3.2    6.9    -2.5  1.30
             SIMULT SHEAR    6.9      0.0      SIMULT MOM    46.4    39.1
    9.84     37.3    47.9   47.0      0.0       1.3    1.6    5.2    -3.3  1.30
             SIMULT SHEAR    5.2      0.0      SIMULT MOM    47.0    44.7
   12.30     38.8    49.9   46.6      0.0       0.0    0.0    4.1    -4.1  1.30
             SIMULT SHEAR   -4.1      0.0      SIMULT MOM    46.6    46.6
   14.76     37.3    47.9   47.0      0.0      -1.3   -1.6    3.3    -5.2  1.30
             SIMULT SHEAR   -5.2      0.0      SIMULT MOM    44.7    47.0
   17.22     32.6    41.9   46.4      0.0      -2.5   -3.2    2.5    -6.9  1.30
             SIMULT SHEAR   -6.9      0.0      SIMULT MOM    39.1    46.4
   19.68     24.8    31.9   38.4      0.0      -3.8   -4.9    1.7    -8.5  1.30
             SIMULT SHEAR   -8.5      0.0      SIMULT MOM    29.8    38.4
   22.14     14.0    17.9   22.9      0.0      -5.0   -6.5    0.8   -10.2  1.30
             SIMULT SHEAR  -10.2      0.0      SIMULT MOM    16.8    22.9
   24.60      0.0     0.0    0.0      0.0      -6.3   -8.1    0.0   -11.8  1.30
             SIMULT SHEAR    0.0      0.0      SIMULT MOM     0.0     0.0


NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR
      UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER
      SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.


                     STRENGTHS AND LOAD FACTOR RATINGS

SPAN  1
=======
        NON-COMP OVERLOAD          NON-COMPACT     COMPACT      COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS    MOMENT   RATING FACTORS
   X    STRENGTH STRENGTH STRENGTH   IR        OR    STRENGTH    IR        OR
  0.00    331.4 B   314.8   112.9   3.67 V   6.12 V    390.7   3.67 V   6.12 V
  2.46    331.4 B   314.8   112.9   4.44 V   7.40 V    390.7   4.44 V   7.40 V
  4.92    331.4 B   314.8   112.9   2.85 B   4.76 B    390.7   3.79 O   6.32 O
  7.38    331.4 B   314.8   112.9   2.07 B   3.45 B    390.7   2.84 O   4.74 O
  9.84    331.4 B   314.8   112.9   1.87 B   3.12 B    390.7   2.63 O   4.39 O
 12.30    331.4 B   314.8   112.9   1.83 B   3.05 B    390.7   2.60 O   4.33 O
 14.76    331.4 B   314.8   112.9   1.87 B   3.12 B    390.7   2.63 O   4.39 O
 17.22    331.4 B   314.8   112.9   2.07 B   3.45 B    390.7   2.84 O   4.74 O
 19.68    331.4 B   314.8   112.9   2.85 B   4.76 B    390.7   3.79 O   6.32 O
 22.14    331.4 B   314.8   112.9   4.44 V   7.40 V    390.7   4.44 V   7.40 V
 24.60    331.4 B   314.8   112.9   3.67 V   6.12 V    390.7   3.67 V   6.12 V


                       ********************************
                       *  GIRDER  -  LIVE LOAD TK527  *
                       ********************************

                                MAXIMUM REACTIONS

                                                     REACTIONS     MOMENTS
    SUPPORT    DL1      DL2   +(LL+I)     -(LL+I)  +I.F. -I.F.  +I.F. -I.F.
       1       6.3      8.1     12.3         0.0     1.30


                         UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======
               DL1     DL2   +(LL+I)  -(LL+I)  I    DL1     DL2   +(LL+I) -(LL+I)
     X     MOMENT  MOMENT MOMENT   MOMENT  M   SHEAR  SHEAR  SHEAR   SHEAR  I.F.
    0.00      0.0      0.0    0.0      0.0         6.3    8.1   13.5     0.0  1.30
```

```
            SIMULT SHEAR    0.0       0.0       SIMULT MOM     0.0      0.0
    2.46    14.0    17.9   26.0       0.0        5.0     6.5   11.5     -0.5  1.30
            SIMULT SHEAR   11.5       0.0       SIMULT MOM    26.0     10.8
    4.92    24.8    31.9   43.5       0.0        3.8     4.9    9.7     -1.3  1.30
            SIMULT SHEAR    9.7       0.0       SIMULT MOM    43.5     22.8
    7.38    32.6    41.9   53.7       0.0        2.5     3.2    7.9     -2.3  1.30
            SIMULT SHEAR    7.9       0.0       SIMULT MOM    53.7     36.7
    9.84    37.3    47.9   61.7       0.0        1.3     1.6    6.4     -3.6  1.30
            SIMULT SHEAR    3.7       0.0       SIMULT MOM    57.2     48.0
   12.30    38.8    49.9   62.8       0.0        0.0     0.0    4.9     -4.9  1.30
            SIMULT SHEAR   -3.3       0.0       SIMULT MOM    54.7     54.7
   14.76    37.3    47.9   61.7       0.0       -1.3    -1.6    3.6     -6.4  1.30
            SIMULT SHEAR   -3.7       0.0       SIMULT MOM    48.0     57.2
   17.22    32.6    41.9   53.7       0.0       -2.5    -3.2    2.3     -7.9  1.30
            SIMULT SHEAR   -7.9       0.0       SIMULT MOM    36.7     53.7
   19.68    24.8    31.9   43.5       0.0       -3.8    -4.9    1.3     -9.7  1.30
            SIMULT SHEAR   -9.7       0.0       SIMULT MOM    22.8     43.5
   22.14    14.0    17.9   26.0       0.0       -5.0    -6.5    0.5    -11.5  1.30
            SIMULT SHEAR  -11.5       0.0       SIMULT MOM    10.8     26.0
   24.60     0.0     0.0    0.0       0.0       -6.3    -8.1    0.0    -13.5  1.30
            SIMULT SHEAR    0.0       0.0       SIMULT MOM     0.0      0.0
```

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 331.4 B | 314.8 | 112.9 | 3.23 V | 5.38 V | 390.7 | 3.23 V | 5.38 V |
| 2.46 | 331.4 B | 314.8 | 112.9 | 3.92 V | 6.53 V | 390.7 | 3.92 V | 6.53 V |
| 4.92 | 331.4 B | 314.8 | 112.9 | 2.52 B | 4.20 B | 390.7 | 3.35 O | 5.58 O |
| 7.38 | 331.4 B | 314.8 | 112.9 | 1.79 B | 2.98 B | 390.7 | 2.46 O | 4.10 O |
| 9.84 | 331.4 B | 314.8 | 112.9 | 1.42 B | 2.37 B | 390.7 | 2.01 O | 3.35 O |
| 12.30 | 331.4 B | 314.8 | 112.9 | 1.36 B | 2.26 B | 390.7 | 1.93 O | 3.22 O |
| 14.76 | 331.4 B | 314.8 | 112.9 | 1.42 B | 2.37 B | 390.7 | 2.01 O | 3.35 O |
| 17.22 | 331.4 B | 314.8 | 112.9 | 1.79 B | 2.98 B | 390.7 | 2.46 O | 4.10 O |
| 19.68 | 331.4 B | 314.8 | 112.9 | 2.52 B | 4.20 B | 390.7 | 3.35 O | 5.58 O |
| 22.14 | 331.4 B | 314.8 | 112.9 | 3.92 V | 6.53 V | 390.7 | 3.92 V | 6.53 V |
| 24.60 | 331.4 B | 314.8 | 112.9 | 3.23 V | 5.38 V | 390.7 | 3.23 V | 5.38 V |

```
********************************
*  GIRDER  -  LIVE LOAD  ML80  *
********************************
```

MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 6.3 | 8.1 | 13.1 | 0.0 | 1.30 | | | |

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | | 6.3 | 8.1 | 14.3 | 0.0 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 2.46 | 14.0 | 17.9 | 27.9 | 0.0 | | 5.0 | 6.5 | 12.4 | -0.5 | 1.30 |
| | SIMULT | SHEAR | 12.4 | 0.0 | | SIMULT | MOM | 27.9 | 10.8 | |
| 4.92 | 24.8 | 31.9 | 46.9 | 0.0 | | 3.8 | 4.9 | 10.4 | -1.3 | 1.30 |
| | SIMULT | SHEAR | 10.4 | 0.0 | | SIMULT | MOM | 46.9 | 22.8 | |
| 7.38 | 32.6 | 41.9 | 59.2 | 0.0 | | 2.5 | 3.2 | 8.6 | -2.3 | 1.30 |
| | SIMULT | SHEAR | 6.3 | 0.0 | | SIMULT | MOM | 58.0 | 36.7 | |
| 9.84 | 37.3 | 47.9 | 67.8 | 0.0 | | 1.3 | 1.6 | 7.0 | -3.8 | 1.30 |
| | SIMULT | SHEAR | 4.4 | 0.0 | | SIMULT | MOM | 62.9 | 51.2 | |
| 12.30 | 38.8 | 49.9 | 70.4 | 0.0 | | 0.0 | 0.0 | 5.4 | -5.4 | 1.30 |
| | SIMULT | SHEAR | 2.7 | 0.0 | | SIMULT | MOM | 60.7 | 60.7 | |
| 14.76 | 37.3 | 47.9 | 67.8 | 0.0 | | -1.3 | -1.6 | 3.8 | -7.0 | 1.30 |
| | SIMULT | SHEAR | -4.4 | 0.0 | | SIMULT | MOM | 51.2 | 62.9 | |
| 17.22 | 32.6 | 41.9 | 59.2 | 0.0 | | -2.5 | -3.2 | 2.3 | -8.6 | 1.30 |
| | SIMULT | SHEAR | -6.3 | 0.0 | | SIMULT | MOM | 36.7 | 58.0 | |
| 19.68 | 24.8 | 31.9 | 46.9 | 0.0 | | -3.8 | -4.9 | 1.3 | -10.4 | 1.30 |
| | SIMULT | SHEAR | -10.4 | 0.0 | | SIMULT | MOM | 22.8 | 46.9 | |
| 22.14 | 14.0 | 17.9 | 27.9 | 0.0 | | -5.0 | -6.5 | 0.5 | -12.4 | 1.30 |
| | SIMULT | SHEAR | -12.4 | 0.0 | | SIMULT | MOM | 10.8 | 27.9 | |
| 24.60 | 0.0 | 0.0 | 0.0 | 0.0 | | -6.3 | -8.1 | 0.0 | -14.3 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 331.4 B | 314.8 | 112.9 | 3.03 V | 5.06 V | 390.7 | 3.03 V | 5.06 V |
| 2.46 | 331.4 B | 314.8 | 112.9 | 3.65 V | 6.09 V | 390.7 | 3.65 V | 6.09 V |
| 4.92 | 331.4 B | 314.8 | 112.9 | 2.33 B | 3.89 B | 390.7 | 3.10 O | 5.17 O |
| 7.38 | 331.4 B | 314.8 | 112.9 | 1.62 B | 2.70 B | 390.7 | 2.23 O | 3.72 O |
| 9.84 | 331.4 B | 314.8 | 112.9 | 1.30 B | 2.16 B | 390.7 | 1.83 O | 3.04 O |
| 12.30 | 331.4 B | 314.8 | 112.9 | 1.21 B | 2.02 B | 390.7 | 1.72 O | 2.87 O |
| 14.76 | 331.4 B | 314.8 | 112.9 | 1.30 B | 2.16 B | 390.7 | 1.83 O | 3.04 O |
| 17.22 | 331.4 B | 314.8 | 112.9 | 1.62 B | 2.70 B | 390.7 | 2.23 O | 3.72 O |
| 19.68 | 331.4 B | 314.8 | 112.9 | 2.33 B | 3.89 B | 390.7 | 3.10 O | 5.17 O |
| 22.14 | 331.4 B | 314.8 | 112.9 | 3.65 V | 6.09 V | 390.7 | 3.65 V | 6.09 V |
| 24.60 | 331.4 B | 314.8 | 112.9 | 3.03 V | 5.06 V | 390.7 | 3.03 V | 5.06 V |

```
+++++++++++++++++++++++++++++++++++++++++++++
+                                           +
+        R A T I N G   S U M M A R Y        +
+                                           +
+++++++++++++++++++++++++++++++++++++++++++++
```

MEMBER: GIRDER

LOAD FACTOR RATING

| LOAD | | FACTOR | TONS | X | SPAN |
|---|---|---|---|---|---|
| H20 | IR (CRITICAL) | 2.60 O | 52.0 | 12.30 | 1 |
| | OR (CRITICAL) | 4.33 O | 86.7 | 12.30 | 1 |
| | IR ( POS MOM) | 2.60 O | 52.0 | 12.30 | 1 |

```
       OR ( POS MOM)     4.33 O    86.7    12.30  1
  HS20 IR (CRITICAL)     2.60 O    93.6    12.30  1
       OR (CRITICAL)     4.33 O   156.1    12.30  1
       IR ( POS MOM)     2.60 O    93.6    12.30  1
       OR ( POS MOM)     4.33 O   156.1    12.30  1
 TK527 IR (CRITICAL)     1.93 O    77.2    12.30  1
       OR (CRITICAL)     3.22 O   128.6    12.30  1
       IR ( POS MOM)     1.93 O    77.2    12.30  1
       OR ( POS MOM)     3.22 O   128.6    12.30  1
  ML80 IR (CRITICAL)     1.72 O    63.0    12.30  1
       OR (CRITICAL)     2.87 O   105.0    12.30  1
       IR ( POS MOM)     1.72 O    63.0    12.30  1
       OR ( POS MOM)     2.87 O   105.0    12.30  1

  NOTE: THIS RATING SUMMARY OF GIRDERS DOES FOLLOW THE ALL-OR-NONE
        COMPACT REQUIREMENTS.
        THIS GIRDER DOES HAVE ALL SECTIONS QUALIFYING AS COMPACT
        SECTIONS.  THEREFORE, THE RATING PRINTED IN THE SUMMARY
        IS THAT BASED ON COMPACT SECTIONS

      RATING FACTOR CODES: FIRST CHARACTER AFTER THE RATING FACTOR
      T - TOP STEEL STRESS/STRENGTH GOVERNS
      B - BOTTOM STEEL STRESS/STRENGTH GOVERNS
      C - CONCRETE STRESS/STRENGTH GOVERNS
      R - REINFORCEMENT STRESS/STRENGTH GOVERNS
      V - SHEAR STRESS/STRENGTH GOVERNS
      blank - COMPACT MOMENT STRENGTH GOVERNS
      O - OVERLOAD PROVISIONS GOVERN
      I - MOMENT-SHEAR INTERACTION GOVERNS

      RATING FACTOR CODES FOR FLANGE/WEB BUCKLING: SECOND CHARACTER AFTER THE RATING
FACTOR
      blank - SECTION DOES MEET FLANGE/WEB BUCKLING CRITERIA
          AND WILL NOT BUCKLE.
      F - SECTION DOES NOT MEET FLANGE PROJECTION/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE.
      W - SECTION DOES NOT MEET WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN WEB.
      Z - SECTION DOES NOT MEET BOTH FLANGE PROJECTION/THICKNESS RATIO
          CRITERIA AND WEB DEPTH/THICKNESS RATIO CRITERIA
          AND WILL BUCKLE IN FLANGE OR WEB.

      NON-COMPACT MOMENT STRENGTH CODES:
      B - SECTION IS BRACED
      U - SECTION IS UNBRACED

  NOTE: ALL RATINGS ARE BASED ON THE NUMBER OF DESIGN LANES OR THE ACTUAL
        TRAFFIC LANES AS DEFINED BY "D" OR "L" ENTERED FOR LANES IN THE
        PROJECT IDENTIFICATION.

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

## 3.11 STEEL TRUSSES

This Section covers the rating of steel truss bridges.

### 3.11.1 Policies and Guidelines

Most steel truss bridges can be rated by analytical methods based on design plans and field measurements.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.1 and MBE 6B.5.2.1 for guidance on structural steel, MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.11.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating. Connections for NSTM members must be included as part of the load rating analysis.

See Section 2.6.3 and 2.6.4 for discussion on applicable analysis methods of trusses and floorbeams and stringers respectively.

See Section 2.6.7.1 for discussion on analysis and load rating of gusset plate connections.

See Section 2.6.7.2 for discussion on analysis and load rating of bolted splice connections.

See Section 2.6.7.3 for discussion on analysis and load rating of pins.

The PennDOT program TRLRFD also analyzes and rates truss bridges; however, at this time, is only available for internal use by the Department.

#### 3.11.2.1 LFR or ASR Method

For steel truss bridges built prior to 2011 the typical load rating method will be ASR and LFR. PennDOT's BAR7 program can perform analysis on three different truss configurations: Truss-Floorbeam Type Bridge (Bridge Type TFF), Truss-Floorbeam-Stringer Type Bridge (Bridge Type TFS), Truss-Floorbeam-Stringer Type Bridge (Bridge Type TTT, same as TFS but does not analyze the Floorbeams and Stringers). BAR7 can rate the truss's floor system in both ASR and LFR, however the truss members will only be rated in ASR, while gusset plate and pony truss stability checks will only use LFR.

#### 3.11.2.2 LRFR Method

For steel truss bridges built after 2011 and those to be rated using LRFR, AASHTOWare BrR or other approved software should be used. In some cases, a more refined analysis may be required. See Table 1.2. See Section 2.6.3 and 2.6.4 for more LRFR discussion and references.

### 3.11.3 Live Load and Dead Load Distribution

3.11.3.1 LFR or ASR Method

See Section 3.9.3 for guidance on distribution of load to the floor system. The distribution of the live load on the truss is computed by the BAR7 program based on the design traffic lanes or the loaded traffic lanes as defined by the user of the program (See the BAR7 manual for additional information).

For stringers, a skew correction factor shall be applied to the live load shear distribution factor in accordance with 1993 DM-4, Table 3.23.2(A).

Dead loads placed with the deck and beams (DL1) are to be based on tributary width. Dead loads placed after the slab has cured (DL2) shall be distributed equally among all stringers (AASHTO Standard Specification 3.23.2.3.1.1).

3.11.3.2 LRFR Method

If a steel truss bridge requires a rating with LRFR, distribution factors may be computed in AASHTOWare BrR. For analysis outside of these programs, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.2 for the floor system and 4.6.2.4 for the truss.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Pub. 238 IP 3.3.3.1 for the exterior girder at the obtuse corners when utilizing the AASHTO distribution factors. See DM-4 Table 4.6.2.2.3c-1 for the formulas for the skew correction factor.

### 3.11.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen. There are no general details covering a variety of truss bridges since trusses were specifically designed for individual situations. Below are example drawings that may assist with determining assumed member sizes.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| S Series: S 66 | Standard 50’-0” C-C Pony Truss | 7/7/1913 | S 66 |
| S Series: S 134 | Standard 50’-0” C-C Pony Truss | 8/24/1914 | S 134 |
| S Series: S 143 | General Drawing for One 50’-0” Span Pony Truss | 6/21/1915 | S 143 |
| S Series: S 259 | Thru Truss Bridge (138’-0” Span) | 7/18/1919 | S 259 |
| S Series: S 267 | Deck Truss Bridge (3 – 135’-0” Spans) | 7/18/1919 | S 267 |
| S Series: S 754 | Thru Truss Bridge (158’-4 5/8” Span) | 4/29/1932 | S 754 |
| S Series: S 251 | 102’-1” Through Truss | -- | S 251 |
| Standards for Old Bridges 1931-1940 Volume 2 | Thru Truss Bridge (205’-4” Span) | 11/16/1932 | S-802 |

| | | | |
|---|---|---|---|
| Standards for Old Bridges 1961-1965 Volume 4 | Flame-Shortening Details for tightening loose Eyebars in Truss Bridges | 5/15/1963 | S-6409 |

### 3.11.5 Modeling Section Properties and Deterioration

See Section 2.6.1.2 for a discussion of modeling section loss.

### 3.11.6 Standard Practices

In some situations for Truss-Floorbeam-Stringer bridges, as discussed in Section 3.9.6, there will need to be a more in-depth evaluation of the stringers. When using bridge type 'TFS' for an LFR/ASR rating, BAR7 does not have the ability to evaluate intermittent section loss in the stringers since they must have a uniform cross-section throughout. In order to evaluate the effect of section loss, the stringers must be checked in a multi-girder run (Bridge Type 'GGG' in BAR7) in order to accurately account for section loss in the stringers.

When inputting section properties in BAR7, special care should be taken to ensure that the correct section properties are utilized, especially 'net section area' vs. 'gross section area'. When entering the gross area tension, the BAR7 Manual Section 5.15 indicates that the gross area shall not be used if the holes occupy more than 15% of the gross area. When they do, the excess above 15% of the holes not greater than 1.25 in, and all of area of larger holes, should be deducted from the gross area. Consider the location of section loss when determining the gross area tension. For example, if the member contains section loss that doesn't correspond to a location of bolt holes, the section loss can be considered independent of the bolt holes when determining the gross area tension.

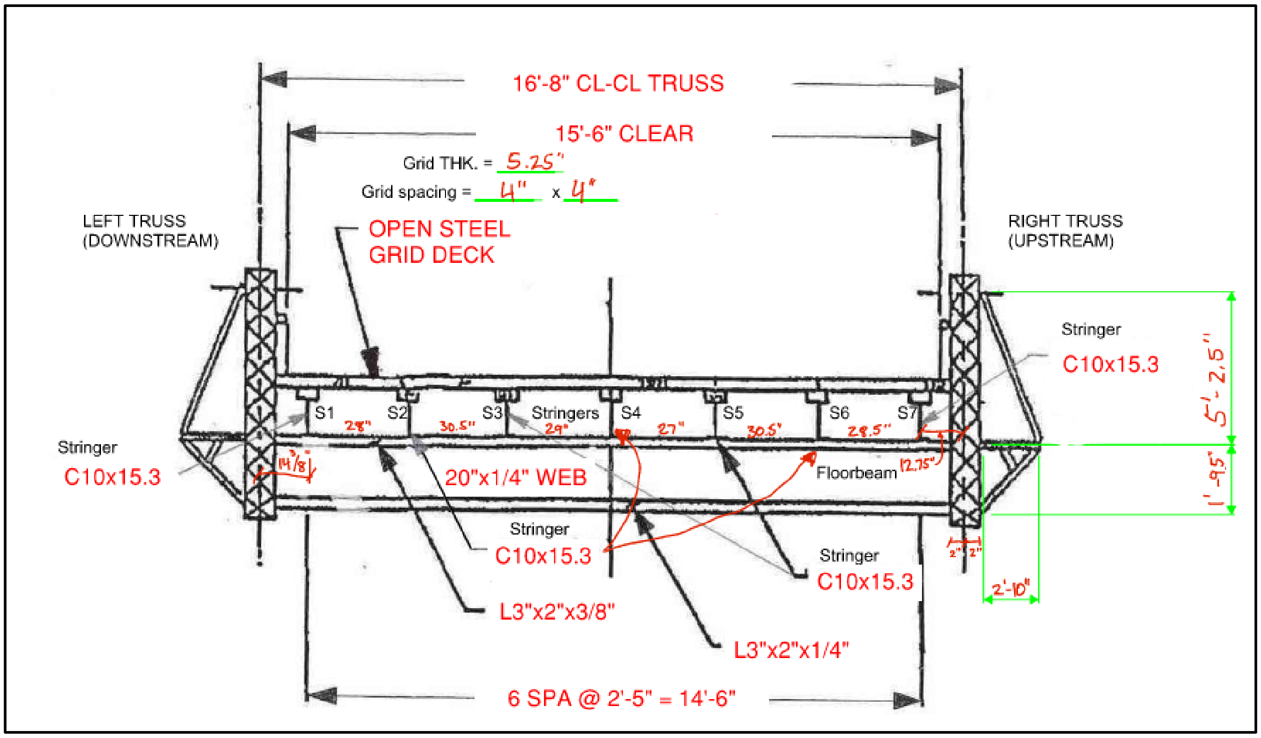
### 3.11.7 Common QA Findings

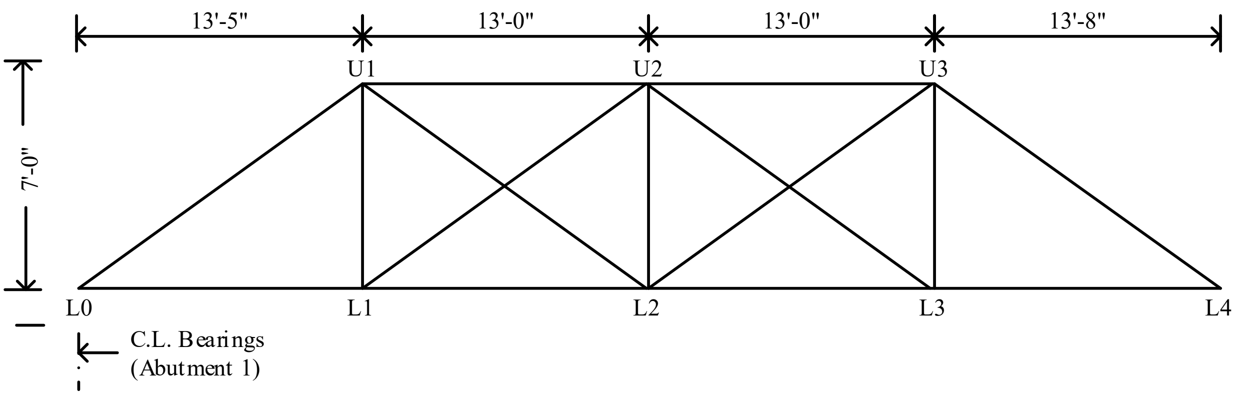
This Section is in development and will be provided in future editions of this manual.

### 3.11.8 Sample Load Rating

This Section contains sample load rating calculations for a typical floorbeam and interior stringer of a single span steel pony truss structure shown below. The analysis will be performed using PennDOT's Bridge Analysis and Rating (BAR7) Program, Version 7.15.0.0 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part-IV (DM-4).

The single span superstructure consists of a 5 ¼" open grid steel deck supported on rolled beam stringers and floorbeams. Details from the Contract Drawings are shown below.




## TRUSS ELEVATION

3.11.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

| | | | |
|---|---|---|---|
| Done By: | | Date: | |
| Checked By: | | Date: | |
| Structure ID (5A01): | 10722303480059 | Inspection Date (7A01): | Nov 2022 |
| Facility Carried (5A08): | T-348 Book Road over Muddy Creek | | |
| Feature Intersected (5A07): | Muddy Creek | | |
| Structure Type (6A26 - 6A29): | 18118 - ST Truss Thru | | |
| Spans / Members Analyzed: | Truss, Floorbeam, and Stringers | | |
| Analysis Method: | Truss - ASD, Floorbeam and Stringers - LFD | | |
| PennDOT Program / Version: | BAR7 Version 7.15.0.0 | | |

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 0.34 | 6.8 | 0.57 | 11.3 | 0.46 | 9.0 | * | * | O | O |
| HS20 | 0.32 | 11.6 | 0.54 | 19.3 | 0.43 | 15.4 | * | * | O | O |
| ML80 | 0.22 | 8.0 | 0.37 | 13.4 | 0.30 | 10.7 | * | * | O | O |
| TK527 | 0.24 | 9.8 | 0.41 | 16.3 | 0.33 | 13.0 | * | * | O | O |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 0.37 | 10.5 | 0.61 | 17.5 | 0.49 | 14.0 | * | * | O | O |
| EV3 | 0.21 | 9.1 | 0.35 | 15.2 | 0.28 | 12.2 | * | * | O | O |

Comments/Assumptions*:

Superstructure and substructure condition ratings are 4 and 4, respectively. Therefore, for an ADTT > 500, the SLC Factor equals 0.80 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is Floorbeam 2.

BAR7 analysis does not include any section loss.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

3.11.8.2 Truss, Floorbeam, and Stringer Load Rating Analysis

3.11.8.2.1 BAR7 Input Parameters

A floorbeam, stringer, and truss for the pony truss will be rated using BAR7. Many of the BAR7 input parameters can be left blank. Only the required input values are discussed below. Refer to the BAR7 User's Manual for additional information.

Project Identification

- Project Identification = "=BRRAT"
- Structure ID = 10722303480059
- Description = SINGLE SPAN PONY TRUSS
- Bridge Type = TFS
- Lanes = D for BAR7 to calculate based on the number of possible lanes.
- Live Load = Blank for H20, HS20, ML80, TK527 vehicles. Enter L for EV2 and EV3 vehicles.
- Output = Blank
- Concrete Deck = Blank
- S Over Factor = 6 for S/6 per AASHTO Table 3.23.1 for Steel Grid (4" or more), 1 Lane.

Bridge Cross Section and Loading

- Deck Width = 16'-8" = 16.67 ft
- Deck Overhang = 0 ft
- CL Girder to Edge of Curb = 16.67 ft – 15.5 ft = 0.58 ft per Contract Drawings
- Roadway Width = 15'-6" = 15.5 ft per Contract Drawings
- Distribution Factor – Shear
  Per AASHTO 3.23.1, the interior stringer distribution factor for shear is based on the Lever Rule.

  1 Lane Loaded, $DF_V$ = P(1) = 1.000 wheels / (2 wheels/axle) = <u>0.500 axles</u>

- Distribution Factor – Moment

  Per AASHTO Table 3.23.1, the interior stringer moment distribution factor for steel grid decks for one lane loaded, $DF_M$ = S/6

  Number of Lanes, $N_L$ = (Roadway Width/12' Lane Width)
  Roadway Width, W = 15.50 ft
  $N_L$ = 15.5 ft / 12 ft = 1.29 ==> Use 1 lane
  Beam Spacing, S = 2.41 ft

  $DF_M$ = S / 6 = 2.41 ft / 6 = 0.402 wheels x (2 wheels/axle) = <u>0.201 axles</u>

- Distribution Factor – Deflection
  Per AASHTO 10.6.4, the distribution factor for deflection is calculated as
  $DF_{Defl}$ = Number of Lanes x Reduction Factor / Number of Beams
  Reduction Factor = 1.00 for 1 lane per AASHTO 3.12.1

  $DF_{Defl}$ = 1 lane x 1.00 / 7 beams = <u>0.143 lanes/beam</u>

- Dead Loads – DL1
  For the bridge type "TFS", leave blank. The DL1 dead loads applied to the truss are entered separately under Truss Dead Loads. The DL1 dead loads applied to the floorbeams are entered separately under Floorbeam DL1. The DL1 dead loads applied to the stringers are entered separately under Stringer DL1.

  Total Dead Load 1 = Leave Blank

- Dead Loads – DL2
  DL2 are dead loads that are applied to the deck after the deck has hardened. DL2 includes dead loads due to the future wearing surface, weight of parapet railing, steel grid flooring, and other structures permanently attached to the deck.

  3" Pipe Railing = 0.0078 klf x 2 = 0.0156 klf
  Metal Grid Deck = 0.0185 ksf x 16 ft = 0.296 klf

  Total Dead Load 2 = (0.0156 klf + 0.296 klf) / 2 = 0.156 kips/ft

- Live Load Location = L for live loading moving across lower joints

- Number of Panels = 4

- End Connections = R for riveted connections

- End Bearing = L for the abutment bearing located at the lower joint

- Stringer Dead Load
  The stringer dead load includes the weight of haunch, permanent formwork, bracing, stiffeners and other hardware attached to the stringer. Stringer self-weight and deck slab weight are calculated by BAR7.

  Metal Grid Deck = 0.0185 ksf x 2.4167 ft = 0.045 kips/ft
  Stringer Dead Load = 0.045 kips/ft

- Floorbeam Dead Load
  The floorbeam dead load 1 includes the weight of the bracing, stiffeners and other hardware attached to the floorbeam. Floorbeam self-weight and deck slab weight are calculated by BAR7.

  Floorbeam Dead Load = 0 kips/ft

Span Lengths

- Continuity = S for Simple Span
- Span 1 Length = 53'-1" = 53.08 ft per Contract Drawings

Stringer Span Lengths

- Continuity = C for Continuous Span
- Span Length 1 = 13.42 ft
- Span Length 2 = 13.00 ft
- Span Length 3 = 13.00 ft
- Span Length 4 = 13.67 ft

Stringer Locations

- Continuity = C for Continuous Span
- Stringer 1 Location = 1.20 ft
- Stringer 2 Location = 3.53 ft
- Stringer 3 Location = 6.07 ft
- Stringer 4 Location = 8.49 ft
- Stringer 5 Location = 10.74 ft
- Stringer 6 Location = 13.28 ft
- Stringer 7 Location = 15.66 ft

Truss Geometry

- Panel Number 1
- Panel Width = 13.42 ft
- Vertical Post Height, H = 7.00 ft
- Panel Type = 1
- Panel Number 2
- Panel Width = 13.00 ft
- Vertical Post Height, H = 7.00 ft
- Panel Type = 13
- Panel Number 3
- Panel Width = 13.00 ft
- Vertical Post Height, H = 7.00 ft
- Panel Type = 13
- Panel Number 4
- Panel Width = 13.67 ft
- Vertical Post Height, H = 7.00 ft
- Panel Type = 2

Truss Dead Load

The truss dead loads acting on the truss joints include steel grid weight, stringers, floorbeams, stiffeners, knee braces, and splice plates…etc. The weight of the SIP forms and haunch weight are entered as floorbeam dead loads and then get applied to the girder. The remaining loads are calculated below.

Joint L0
L0-L1: 0.012 klf x 13.42 ft / 2 = 0.080 kips
L0-U1: 0.020 klf x 15.14 ft / 2 = 0.151 kips
Grid Deck: 0.0185 ksf x 13.42 ft / 2 x 16.33 ft / 2 = 1.014 kips
Stringers: 0.0153 kips/ft x 13.42 ft / 2 x 7 / 2 = 0.359 kips
Floorbeam: 0.037 kips/ft x 16.50 ft / 2 = 0.305 kips
Total = 1.91 x 1.15 (misc) = 2.20 kips

Joint L1
L0-L1: 0.012 klf x 13.42 ft / 2 = 0.080 kips
L1-U1: 0.011 klf x 7.00 ft / 2 = 0.039 kips
L1-U2: 0.001 klf x 14.76 ft / 2 = 0.008 kips
L1-L2: 0.012 klf x 13.00 ft / 2 = 0.078 kips
Grid Deck: 0.0185 ksf x (13.42 ft + 13.00 ft) / 2 x 16.33 ft / 2 = 1.995 kips
Stringers: 0.0153 kips/ft x (13.42 ft + 13.00 ft) / 2 x 7 / 2 = 0.707 kips
Floorbeam: 0.037 kips/ft x 16.50 ft / 2 = 0.305 kips
Total = 3.21 x 1.15 (misc) = 3.69 kips

Joint L2
L1-L1: 0.012 klf x 13.00 ft / 2 = 0.078 kips
L2-U1: 0.007 klf x 14.76 ft / 2 = 0.052 kips
L2-U2: 0.011 klf x 7.00 ft / 2 = 0.039 kips
L2-U3: 0.007 klf x 14.76 ft / 2 = 0.052 kips
L2-L3: 0.012 klf x 13.00 ft / 2 = 0.078 kips
Grid Deck: 0.0185 ksf x 13.00 ft x 16.33 ft / 2 = 1.964 kips
Stringers: 0.0153 kips/ft x 13.00 ft x 7 / 2 = 0.696 kips
Floorbeam: 0.037 kips/ft x 16.50 ft / 2 = 0.305 kips
Total = 3.26 x 1.15 (misc) = 3.75 kips

Joint L3
L3-L4: 0.012 klf x 13.42 ft / 2 = 0.081 kips
L3-U3: 0.011 klf x 7.00 ft / 2 = 0.039 kips
L3-L3: 0.001 klf x 14.76 ft / 2 = 0.008 kips
L3-U2: 0.012 klf x 13.00 ft / 2 = 0.078 kips
Grid Deck: 0.0185 ksf x (13.67 ft + 13.00 ft) / 2 x 16.33 ft / 2 = 2.014 kips
Stringers: 0.0153 kips/ft x (13.67 ft + 13.00 ft) / 2 x 7 / 2 = 0.714 kips
Floorbeam: 0.037 kips/ft x 16.50 ft / 2 = 0.305 kips
Total = 3.24 x 1.15 (misc) = 3.73 kips

Joint L4
L4-U3: 0.020 klf x 15.36 ft / 2 = 0.154 kips
L3-L4: 0.012 klf x 13.67 ft / 2 = 0.082 kips
Grid Deck: 0.0185 ksf x 13.67 ft / 2 x 16.33 ft / 2 = 1.032 kips
Stringers: 0.0153 kips/ft x 13.67 ft / 2 x 7 / 2 = 0.366 kips
Floorbeam: 0.037 kips/ft x 16.50 ft / 2 = 0.305 kips
Total = 1.94 x 1.15 (misc) = 2.23 kips

Joint U1
L0-U1: 0.020 klf x 15.14 ft / 2 = 0.151 kips
L1-U1: 0.011 klf x 7.00 ft / 2 = 0.039 kips
L2-U1: 0.007 klf x 14.76 ft / 2 = 0.052 kips
U1-U2: 0.020 klf x 13.00 ft / 2 = 0.130 kips
Total = 0.37 x 1.15 (misc) = 0.43 kips

Joint U2
U1-U2: 0.020 klf x 13.00 ft / 2 = 0.130 kips
L1-U2: 0.001 klf x 14.76 ft / 2 = 0.0074 kips
L2-U2: 0.011 klf x 7.00 ft / 2 = 0.039 kips
L3-U2: 0.001 klf x 14.76 ft / 2 = 0.008 kips
U2-U3: 0.020 klf x 13.00 ft / 2 = 0.130 kips
Total = 0.31 x 1.15 (misc) = 0.36 kips

Joint U3
L0-U1: 0.020 klf x 13.00 ft / 2 = 0.130 kips
L1-U1: 0.007 klf x 14.76 ft / 2 = 0.052 kips
L2-U1: 0.011 klf x 7.00 ft / 2 = 0.039 kips
U1-U2: 0.020 klf x 15.36 ft / 2 = 0.154 kips
Total = 0.38 x 1.15 (misc) = 0.43 kips

Steel Member Properties

The steel truss member section properties were calculated separately using AutoCAD. The floorbeam member consists of a 20” x ¼” web with L3x2x3/8” angles at the top and L3x2x1/4” angles at the bottom. The vertical legs of the angles are neglected for analysis purposes. The stringer section properties are based on a C10x15.3 Channel.

Lateral Brace Points

Lateral brace points for a floorbeam should be entered based on which flange is in compression. For a simple span bridge, the top flange is in compression for the entire span length while the bottom flange is in tension.

- Bracing or Stiffener Code = BF for brace points of a floorbeam
- Span Number = 2
- Bracing Point or Stiffener Code = T for top flange brace point
- Number of Spaces = 1, Spacing = 1.08 ft
- Number of Spaces = 6, Spacing = 2.42 ft

**PROJECT**

| | | | |
|---|---|---|---|
| Structure ID: | 10-7223-0348-0059 | | |
| Description: | Single Span Pony Truss | | |
| Bridge Type: | TFS | | Truss-Floorbeam-Stringer |
| SLC Level: | | | Super = 5 & Sub = 6; ADTT = 23 therefore SLC = 1.0 |
| Lanes: | D | | D for Design Lanes |
| Live Load: | | | H20, HS20, ML80, and TK527 |
| Output: | | | Blank for Normal Blank |
| Impact Factor: | | | Compute per AASHTO |
| Gage Distance: | | ft | Compute per AASHTO |
| Passing Distance: | | ft | Compute per AASHTO |
| Fatigue: | | | |
| Concrete Deck: | | | |
| Spec: | | | |
| Redist: | | | |
| Direction: | | | Leave blank to analyze loads in both directions |
| S over factor: | 6.0 | | S/6 |
| End Panel: | | | |
| Hyb: | | | |
| Skew Correction Factor: | | | |
| Pony Truss: | | | |
| PDF: | Y | | |
| Compact Req: | | | |

| Truss Geometry | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Panel Number | 1 | 2 | 3 | 4 | | | | |
| Panel Width | 13.42 | 13.0000 | 13.0000 | 13.67 | | | | |
| Vertical Post Height | 7.00 | 7.00 | 7.00 | 7.00 | | | | |
| Panel Type | 1 | 3 | 13 | 2 | | | | |

| Lateral Bracing | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| BAR7 Record # | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 |
| Bracing or Stiff Code | BF | | | | | | | |
| Span Number | 2 | | | | | | | |
| Code 1 | T | | | | | | | |
| Num of Spaces | 1 | | | | | | | |
| Spacing 1 | 1.08 | | | | | | | |
| Code 2 | T | | | | | | | |
| Num of Spaces | 6 | | | | | | | |
| Spacing 2 | 2.42 | | | | | | | |

**Cross Section**

| | | | |
|---|---|---|---|
| Deck Width: | 16.67 | ft | Per plans |
| Overhang or Spacing: | 0.00 | ft | Per plans |
| CL of Girder: | 0.58 | ft | See Hand Calcs |
| Roadway Width: | 15.50 | ft | Per plans |
| Distr Factor -Shear: | 0.500 | | See Hand Calcs |
| Distr Factor -Moment: | 0.201 | | See Hand Calcs |
| Distr Factor -Deflect: | 0.143 | | See Hand Calcs |
| Slab Thickness: | | in | |
| Haunch: | | in | |
| Bridge DL1: | | kip/ft | Leave Blank |
| Bridge DL2: | 0.156 | kip/ft | See Hand Calcs |
| F'c: | | ksi | |
| N: | | | |
| Symmetry: | | | |
| LL Location: | L | | Only applies to truss |
| Number of Panels: | 4 | | Only applies to truss |
| End Connections: | R | | Only applies to truss |
| CORS: | | | Only applies to truss |
| Hinge at U or L: | | | Only applies to truss |
| Hinge at Panels: | | | Only applies to truss |
| Temp Change: | | °F | Only applies to truss |
| End Bearing: | L | | Only applies to truss |
| Stringer DL: | 0.045 | kip/ft | See Hand Calcs |
| Floorbeam DL: | 0.000 | kip/ft | See Hand Calcs |

**Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | S | | Simple Spans |
| Span Length 1: | 53.08 | ft | See Hand Calcs |

**Stringer Span Lengths**

| | | | |
|---|---|---|---|
| Continuity: | C | | Continuous span |
| Span Length 1: | 13.42 | ft | Per plans/See Hand Calcs |
| Span Length 2: | 13.00 | ft | Per plans/See Hand Calcs |
| Span Length 3: | 13.00 | ft | Per plans/See Hand Calcs |
| Span Length 4: | 13.67 | ft | Per plans/See Hand Calcs |

**Stringer Locations**

| | | | |
|---|---|---|---|
| Continuity: | C | | Continuous span |
| Stringer 1 Location: | 1.20 | ft | Per plans/See Hand Calcs |
| Stringer 2 Location: | 3.53 | ft | Per plans/See Hand Calcs |
| Stringer 3 Location: | 6.07 | ft | Per plans/See Hand Calcs |
| Stringer 4 Location: | 8.49 | ft | Per plans/See Hand Calcs |
| Stringer 5 Location: | 10.74 | ft | Per plans/See Hand Calcs |
| Stringer 6 Location: | 13.28 | ft | Per plans/See Hand Calcs |
| Stringer 7 Location: | 15.66 | ft | Per plans/See Hand Calcs |

### 3.11.8.2.2 BAR7 Output

```
******************************************************************************
*                                                                            *
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*                                                                            *
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******************************************************************************
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                    BRIDGE ANALYSIS AND RATING (BAR7)                330427

PROGRAM P4353000                                                06/19/2024 17:14
VERSION 7.15.0.0          LAST UPDATED 02/15/2018        DOCUMENTATION 02/2018

INPUT: Truss Floorbeam Stringer BAR7 Analysis.dat



                            PROJECT IDENTIFICATION


STRUCTURE ID - 10722303480059 - SINGLE SPAN PONY TRUSS

 BRG SLC       LIVE OUT-  IMP GAGE PASS FAT- CONC      RE-      S OVER END
TYPE LEV LANES LOAD PUT  FACT DIST DIST IGUE DECK SPEC DIST DIR FACTOR PAN
 TFS       D             0.00  0.0  0.0                          0.00

     SKEW
     CORR   PONY
HYB FACTOR TRUSS PDF COMPACT
     0.000        Y


              BRIDGE CROSS SECTION AND LOADING


       OVERHANG     CL OF
DECK      OR      GIRDER OR   ROADWAY  DISTRIBUTION FACTORS
WIDTH  SPACING TRUSS TO CURB   WIDTH   SHEAR MOMENT DEFLECT
16.67    0.00       0.58       15.50   0.500  0.201  0.143

   SLAB               DEAD LOADS
THICKNESS  HAUNCH    DL1     DL2     F'C     N  SYMMETRY
   0.00      0.00   0.000   0.156   0.000    0.

LIVE LOAD  NO. OF   END         HINGE   TEMP    END
LOCATION   PANELS  COND.  CORS    AT   CHANGE   BRG.
    L         4      R           L 0      0.     L

STRINGER   FLOORBEAM  UNIT WEIGHT    GUSSET   PATCH LOAD
  DL1         DL1    DECK CONCRETE  ANALYSIS   ANALYSIS
 0.045       0.000        0.00

UNSYMM PIER
  SUPPORT



                      SPAN LENGTHS (SIMPLE)

SPAN #       1
LENGTH     53.09


             TRAFFIC LANE LOCATIONS

LANE #    1       2       3       4       5       6       7
DIST
WIDTH
```

```
% LL

             STRINGER SPAN LENGTHS (CONTINUOUS)

SPAN #       1       2       3       4
LENGTH     13.42   13.00   13.00   13.67


                            STRINGER LOCATIONS

STRINGER #   1      2      3      4      5      6      7
DISTANCE    1.20   3.53   6.07   8.49  10.74  13.28  15.66


              TRUSS GEOMETRY

PANEL  PANEL          VERTICAL POST      PANEL
 NO.   WIDTH         H1     H2      H3   TYPE
  1    13.42   Y    7.00   0.00    0.00     1
  2    13.00   Y    7.00   0.00    0.00    13
  3    13.00   Y    7.00   0.00    0.00    13
  4    13.67   Y    7.00   0.00    0.00     2


                        TRUSS DEAD LOADS

LOCAT    LOAD   LOCAT    LOAD   LOCAT    LOAD   LOCAT    LOAD
L   0    2.20   L   1    3.69   L   2    3.75   L   3    3.73
L   4    2.23   U   1    0.43   U   2    0.36   U   3    0.43


                           TRUSS MEMBER PROPERTIES


        GROSS   NET   MOMENT     UNBRACED                                GROSS
MEMBER   AREA   AREA    OF        LENGTH       FLANGE       FAT           AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH    C   CAT    FU    TENS
L 0U 1   5.94   5.94    24.1 30.0  0.00 0.00    0.00  0.00        0.0    5.94

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY        IY
         0.0    0.00      0.00     0.00     0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                GROSS
MEMBER   AREA   AREA    OF        LENGTH       FLANGE       FAT           AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH    C   CAT    FU    TENS
U 1U 2   5.94   5.94    24.1 30.0  0.00 0.00    0.00  0.00        0.0    5.94

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY        IY
         0.0    0.00      0.00     0.00     0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                GROSS
MEMBER   AREA   AREA    OF        LENGTH       FLANGE       FAT           AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH    C   CAT    FU    TENS
U 2U 3   5.94   5.94    24.1 30.0  0.00 0.00    0.00  0.00        0.0    5.94

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY        IY
         0.0    0.00      0.00     0.00     0.000        0.
```

```
        GROSS   NET   MOMENT     UNBRACED                                  GROSS
MEMBER   AREA   AREA    OF         LENGTH       FLANGE       FAT            AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH   C   CAT    FU      TENS
U 3L 4   5.94   5.94    24.1 30.0  0.00 0.00    0.00  0.00         0.0    5.94

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY         IY
         0.0    0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                  GROSS
MEMBER   AREA   AREA    OF         LENGTH       FLANGE       FAT            AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH   C   CAT    FU      TENS
L 0L 1   3.50   3.50     1.2 30.0  0.00 0.00    0.00  0.00         0.0    3.50

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY         IY
         0.0    0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                  GROSS
MEMBER   AREA   AREA    OF         LENGTH       FLANGE       FAT            AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH   C   CAT    FU      TENS
L 1L 2   3.50   3.50     1.2 30.0  0.00 0.00    0.00  0.00         0.0    3.50

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY         IY
         0.0    0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                  GROSS
MEMBER   AREA   AREA    OF         LENGTH       FLANGE       FAT            AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH   C   CAT    FU      TENS
L 2L 3   3.50   3.50     1.2 30.0  0.00 0.00    0.00  0.00         0.0    3.50

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY         IY
         0.0    0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                  GROSS
MEMBER   AREA   AREA    OF         LENGTH       FLANGE       FAT            AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH   C   CAT    FU      TENS
L 3L 4   3.50   3.50     1.2 30.0  0.00 0.00    0.00  0.00         0.0    3.50

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY         IY
         0.0    0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                  GROSS
MEMBER   AREA   AREA    OF         LENGTH       FLANGE       FAT            AREA
  ID    COMPR   TENS INERTIA  FY     L     ECC   WIDTH   C   CAT    FU      TENS
U 1L 2   2.00   2.00     0.2 30.0  0.00 0.00    0.00  0.00         0.0    2.00

               UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
               FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH  LENGTH     SXC      IYC       JXY         IY
         0.0    0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                                  GROSS
MEMBER   AREA   AREA    OF         LENGTH       FLANGE       FAT            AREA
```

```
  ID    COMPR   TENS INERTIA  FY     L    ECC   WIDTH   C   CAT   FU    TENS
L 2U 3   2.00   2.00     0.2 30.0  0.00 0.00    0.00  0.00        0.0   2.00

              UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
              FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH LENGTH     SXC      IYC       JXY         IY
         0.0   0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                             GROSS
MEMBER   AREA   AREA    OF         LENGTH        FLANGE      FAT       AREA
  ID    COMPR   TENS INERTIA  FY     L    ECC   WIDTH   C   CAT   FU    TENS
L 1U 2   0.44   0.44     0.0 30.0  0.00 0.00    0.00  0.00        0.0   0.44

              UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
              FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH LENGTH     SXC      IYC       JXY         IY
         0.0   0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                             GROSS
MEMBER   AREA   AREA    OF         LENGTH        FLANGE      FAT       AREA
  ID    COMPR   TENS INERTIA  FY     L    ECC   WIDTH   C   CAT   FU    TENS
U 2L 3   0.44   0.44     0.0 30.0  0.00 0.00    0.00  0.00        0.0   0.44

              UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
              FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH LENGTH     SXC      IYC       JXY         IY
         0.0   0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                             GROSS
MEMBER   AREA   AREA    OF         LENGTH        FLANGE      FAT       AREA
  ID    COMPR   TENS INERTIA  FY     L    ECC   WIDTH   C   CAT   FU    TENS
L 1U 1   3.25   3.25     1.8 30.0  0.00 0.00    0.00  0.00        0.0   3.25

              UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
              FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH LENGTH     SXC      IYC       JXY         IY
         0.0   0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                             GROSS
MEMBER   AREA   AREA    OF         LENGTH        FLANGE      FAT       AREA
  ID    COMPR   TENS INERTIA  FY     L    ECC   WIDTH   C   CAT   FU    TENS
L 2U 2   3.25   3.25     1.8 30.0  0.00 0.00    0.00  0.00        0.0   3.25

              UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
              FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH LENGTH     SXC      IYC       JXY         IY
         0.0   0.00      0.00     0.00      0.000        0.

        GROSS   NET   MOMENT     UNBRACED                             GROSS
MEMBER   AREA   AREA    OF         LENGTH        FLANGE      FAT       AREA
  ID    COMPR   TENS INERTIA  FY     L    ECC   WIDTH   C   CAT   FU    TENS
L 3U 3   3.25   3.25     1.8 30.0  0.00 0.00    0.00  0.00        0.0   3.25

              UNSUPP   SECTION MOMENT OF  TORSION   MOMENT OF
              FLANGE   MODULUS  INERTIA   INERTIA    INERTIA
        DEPTH LENGTH     SXC      IYC       JXY         IY
         0.0   0.00      0.00     0.00      0.000        0.


                          STEEL MEMBER PROPERTIES

  S         T  WF BM    WF BM   FLANGE             WF BM
G P         Y  M OF I    AREA     OR           V  OR WEB
```

```
F A         P  OR VRT   OR HRZ  ANGLE  FLANGE  A   PLATE    WEB
S N  RANGE E   LEG       LEG    THICK   WIDTH  R   DEPTH   THICK  SHAPE
F 2  16.67 P     0.00     0.00  0.0000  0.000      20.00  0.2500
               TPW   TPT    BPW   BPT    COMP  FY  FY TOP FY BOT CG TOP CG BOT
               6.00 0.3750  6.00 0.2500       30.0   0.0    0.0   0.000  0.000

     RANGE     M OF I    AREA   THICK   WIDTH  V   DEPTH   THICK  SHAPE
S      0.00 W    67.30    4.48  0.4360  2.600      10.00  0.2400
               TPW   TPT    BPW   BPT    COMP  FY  FY TOP FY BOT CG TOP CG BOT
               0.00 0.0000  0.00 0.0000   N  30.0   0.0    0.0   0.000  0.000



                LATERAL BRACE POINTS AND STIFFENER SPACINGS

              C                C                C                C
B OR S        O  NO.           O  NO.           O  NO.           O  NO.
G OR F        D  OF            D  OF            D  OF            D  OF
 CODE   SPAN  E SPCS SPACING   E SPCS SPACING   E SPCS SPACING   E SPCS SPACING
  BF      2   T   1    1.08    T   6    2.42        0    0.00        0    0.00
                  0    0.00        0    0.00        0    0.00        0    0.00


                                     DEFAULT VALUES

                                                          UNIT     END   GUSSET
  SLC               GAGE      PASSING              END    WEIGHT    BRG   PLATE
 LEVEL  LANES     DISTANCE    DISTANCE    CORS    COND     DECK     LOC     FU
   I     ---        6.0         4.0        S      ---      150.0     -      ---


                +++++++++++++++++++++++++++++++++++++++++++++
                +                                           +
                +         T R U S S   A N A L Y S I S       +
                +                                           +
                +++++++++++++++++++++++++++++++++++++++++++++


                        LIVE LOAD DISTRIBUTION FACTORS

                            BASED ON DESIGN LANES
                               TRUSS      TRUSS
                               FORCE   DEFLECTION
                               0.665(1)  0.500(1)


NOTE: IF THE LIVE LOAD STRESS IS ZERO AT ANY SECTION THE RATING
      FACTOR IS PRINTED AS 999.99 INDICATING THAT IT IS INFINITE.

NOTE: IF A SECTION DOES NOT MEET FLANGE OR WEB BUCKLING CRITERIA
      OF CURRENT AASHTO SPECIFICATIONS FOR LOAD FACTOR METHOD, THE
      RATING FACTORS ARE REPRINTED AS 888.88.  THIS INDICATES
      THAT THERE IS A POTENTIAL FATIGUE PROBLEM.


NOTE: THE ALLOWABLE STRESS IN AN ECCENTRICALLY LOADED COMPRESSION MEMBER
      IS COMPUTED BY THE SECANT FORMULA. SEE ARTICLE 5.4.2 OF THE 1978
      AASHTO MANUAL FOR MAINTENANCE INSPECTION OF BRIDGES.

    ***** WARNING *****

    **********************************************************
    * TRUSS GEOMETRY INDICATES THAT THERE ARE SOME PANELS    *
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    * WITH COUNTERS (TYPE 13, 14 OR 15) IN THIS TRUSS.  THE  *
    * PROGRAM ASSUMES THAT THE DIAGONALS IN THESE PANELS CAN *
    * RESIST TENSION ONLY, AND THE TRUSS IS INTERNALLY       *
    * DETERMINATE.  IF THESE DIAGONALS ARE STIFF, AND CAN    *
    * RESIST STRESS REVERSAL, THE TRUSS MAY BE INTERNALLY    *
    * INDETERMINATE.  SUCH TRUSSES SHOULD BE ANALYZED BY     *
    * OTHER APPROPRIATE METHODS.                             *
    **********************************************************


NOTE: THE COMPRESSION CAPACITY OF THE CORRESPONDING TOP CHORDS
      AND VERTICAL CHORDS AT EACH SPECIFIED FLOORBEAM LOCATION
      MAY NOT BE MODIFIED BECAUSE THIS IS NOT A PONY TRUSS OR
      THE LATERAL STABILITY ANALYSIS IS NOT REQUESTED.


                        ******************************
                        *  TRUSS  -  LIVE LOAD  H20   *
                        ******************************


                   MEMBER FORCES AND ALLOWABLE STRESS RATINGS
                                I.F. = IMPACT FACTOR

MEMBER MEMBER    DL          LL + IMPACT FORCE       ALLOWABLE FORCE
  ID   LENGTH   FORCE     COMP  I.F.   TENS  I.F.       COMP    TENS   RATING FACTORS

U 1U 2  13.00    -15.8   -57.8  1.28    0.0  1.28  IR    -76.6    95.0  IR   1.052
                                                   OR    -95.6   133.7  OR   1.379

U 2U 3  13.00    -15.8   -57.8  1.28    0.0  1.28  IR    -76.6    95.0  IR   1.052
                                                   OR    -95.6   133.7  OR   1.379

L 0L 1  13.42     11.9     0.0  1.28   45.4  1.28  IR    -11.1    56.0  IR   0.972
                                                   OR    -13.8    78.8  OR   1.473

L 1L 2  13.00     11.9     0.0  1.28   42.7L 1.28  IR    -11.8    56.0  IR   1.033
                                                   OR    -14.8    78.8  OR   1.565

L 2L 3  13.00     12.1     0.0  1.28   43.2L 1.28  IR    -11.8    56.0  IR   1.017
                                                   OR    -14.8    78.8  OR   1.543

L 3L 4  13.67     12.1     0.0  1.28   45.9  1.28  IR    -10.7    56.0  IR   0.957
                                                   OR    -13.3    78.8  OR   1.453

L 0U 1  15.14    -13.4   -59.2  1.28    0.0  1.00  IR    -74.0    95.0  IR   1.023
                                                   OR    -92.3   133.7  OR   1.332

L 1U 2  14.76      0.0     0.0  1.00   10.3  1.30  IR      0.0     7.0  IR   0.686
                                                   OR      0.0     9.9  OR   0.964

L 2U 3  14.76      4.3     0.0  1.00   33.7L 1.30  IR     -1.5    32.0  IR   0.822
                                                   OR     -1.9    45.0  OR   1.207

U 3L 4  15.36    -13.5   -59.4  1.28    0.0  1.00  IR    -73.7    95.0  IR   1.013
                                                   OR    -91.9   133.7  OR   1.320

U 1L 2  14.76      4.4     0.0  1.00   34.2L 1.30  IR     -1.5    32.0  IR   0.807
                                                   OR     -1.9    45.0  OR   1.188

U 2L 3  14.76      0.0     0.0  1.00   10.7  1.30  IR      0.0     7.0  IR   0.657
                                                   OR      0.0     9.9  OR   0.924
```

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| L 1U 1 | 7.00 | 3.7 | 0.0 | 1.00 | 29.8L | 1.30 | IR | -37.4 | 52.0 | IR | 1.622 |
| | | | | | | | OR | -46.6 | 73.1 | OR | 2.331 |
| L 2U 2 | 7.00 | -0.4 | -5.0 | 1.28 | 0.0 | 1.00 | IR | -37.4 | 52.0 | IR | 7.395 |
| | | | | | | | OR | -46.6 | 73.1 | OR | 9.240 |
| L 3U 3 | 7.00 | 3.7 | 0.0 | 1.00 | 29.5L | 1.30 | IR | -37.4 | 52.0 | IR | 1.636 |
| | | | | | | | OR | -46.6 | 73.1 | OR | 2.352 |

MEMBER U 2L 3 IS CRITICAL. SEE DETAILS BELOW.

| DL | LL + IMPACT FORCE | | | | ALLOWABLE FORCE | | | RATING | | |
|---|---|---|---|---|---|---|---|---|---|---|
| FORCE | COMP | I.F. | TENS | I.F. | | COMP | TENS | | FACTOR | TONS |
| 0.0 | 0.0 | 1.00 | 10.7 | 1.30 | IR | 0.0 | 7.0 | IR | 0.657 | 13.15 |
| | | | | | OR | 0.0 | 9.9 | OR | 0.924 | 18.49 |

SUPPORT REACTIONS

| SUPPORT | D.L. | LL+I | I.F. |
|---|---|---|---|
| L 0 | 8.4 | 36.6 L | 1.28 |
| L 4 | 8.4 | 36.6 L | 1.28 |

PANEL POINT DEFLECTIONS

| PANEL | DEAD LOAD + TEMP | | LIVE LOAD + IMPACT | | | | | |
|---|---|---|---|---|---|---|---|---|
| POINT | VERT | HOR | VERT+ | I.F. | VERT- | I.F. | HOR+ | HOR- |
| L 0 | 0.00 | 0.00 | 0.00 | 1.28 | 0.00 | 1.28 | 0.00 | 0.00 |
| L 1 | 0.14 | 0.02 | 0.41 | 1.28 | 0.00 | 1.28 | 0.05 | 0.00 |
| L 2 | 0.19 | 0.04 | 0.51 | 1.28 | 0.00 | 1.28 | 0.09 | 0.00 |
| L 3 | 0.14 | 0.06 | 0.42 | 1.28 | 0.00 | 1.28 | 0.12 | 0.00 |
| L 4 | 0.00 | 0.08 | 0.00 | 1.28 | 0.00 | 1.28 | 0.16 | 0.00 |

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********************************
*  TRUSS  -  LIVE LOAD  HS20  *
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MEMBER FORCES AND ALLOWABLE STRESS RATINGS
I.F. = IMPACT FACTOR

| MEMBER | MEMBER | DL | LL + IMPACT FORCE | | | | | ALLOWABLE FORCE | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ID | LENGTH | FORCE | COMP | I.F. | TENS | I.F. | | COMP | TENS | RATING FACTORS | |
| U 1U 2 | 13.00 | -15.8 | -82.3 | 1.28 | 0.0 | 1.28 | IR | -76.6 | 95.0 | IR | 0.739 |
| | | | | | | | OR | -95.6 | 133.7 | OR | 0.969 |
| U 2U 3 | 13.00 | -15.8 | -82.3 | 1.28 | 0.0 | 1.28 | IR | -76.6 | 95.0 | IR | 0.739 |
| | | | | | | | OR | -95.6 | 133.7 | OR | 0.969 |
| L 0L 1 | 13.42 | 11.9 | 0.0 | 1.28 | 67.2 | 1.28 | IR | -11.1 | 56.0 | IR | 0.656 |
| | | | | | | | OR | -13.8 | 78.8 | OR | 0.995 |
| L 1L 2 | 13.00 | 11.9 | 0.0 | 1.28 | 67.2 | 1.28 | IR | -11.8 | 56.0 | IR | 0.656 |
| | | | | | | | OR | -14.8 | 78.8 | OR | 0.995 |
| L 2L 3 | 13.00 | 12.1 | 0.0 | 1.28 | 67.9 | 1.28 | IR | -11.8 | 56.0 | IR | 0.647 |
| | | | | | | | OR | -14.8 | 78.8 | OR | 0.983 |

```
L 3L 4  13.67     12.1      0.0  1.28   67.9  1.28  IR    -10.7    56.0  IR   0.647
                                                    OR    -13.3    78.8  OR   0.983

L 0U 1  15.14    -13.4    -75.8  1.28    0.0  1.00  IR    -74.0    95.0  IR   0.799
                                                    OR    -92.3   133.7  OR   1.040

L 1U 2  14.76      0.0      0.0  1.00   10.3  1.30  IR      0.0     7.0  IR   0.686
                                                    OR      0.0     9.9  OR   0.964

L 2U 3  14.76      4.3      0.0  1.00   42.7  1.30  IR     -1.5    32.0  IR   0.650
                                                    OR     -1.9    45.0  OR   0.954

U 3L 4  15.36    -13.5    -76.3  1.28    0.0  1.00  IR    -73.7    95.0  IR   0.789
                                                    OR    -91.9   133.7  OR   1.027

U 1L 2  14.76      4.4      0.0  1.00   43.2  1.30  IR     -1.5    32.0  IR   0.638
                                                    OR     -1.9    45.0  OR   0.938

U 2L 3  14.76      0.0      0.0  1.00   10.7  1.30  IR      0.0     7.0  IR   0.657
                                                    OR      0.0     9.9  OR   0.924

L 1U 1   7.00      3.7      0.0  1.00   29.8L 1.30  IR    -37.4    52.0  IR   1.622
                                                    OR    -46.6    73.1  OR   2.331

L 2U 2   7.00     -0.4     -5.0  1.28    0.0  1.00  IR    -37.4    52.0  IR   7.395
                                                    OR    -46.6    73.1  OR   9.240

L 3U 3   7.00      3.7      0.0  1.00   29.5L 1.30  IR    -37.4    52.0  IR   1.636
                                                    OR    -46.6    73.1  OR   2.352


                   MEMBER U 1L 2 IS CRITICAL. SEE DETAILS BELOW.

      DL       LL + IMPACT FORCE      ALLOWABLE FORCE          RATING
    FORCE    COMP  I.F.   TENS  I.F.       COMP  TENS        FACTOR   TONS

     4.4      0.0  1.00   43.2  1.30  IR    -1.5   32.0    IR   0.638  22.96
                                      OR    -1.9   45.0    OR   0.938  33.79


                             SUPPORT REACTIONS

                      SUPPORT    D.L.      LL+I    I.F.
                        L 0       8.4      50.5    1.28
                        L 4       8.4      50.5    1.28


                    PANEL POINT DEFLECTIONS

PANEL  DEAD LOAD + TEMP             LIVE LOAD + IMPACT
POINT    VERT    HOR       VERT+  I.F.   VERT-  I.F.      HOR+    HOR-
 L 0     0.00    0.00       0.00  1.28    0.00  1.28      0.00    0.00
 L 1     0.14    0.02       0.64  1.28    0.00  1.28      0.08    0.00
 L 2     0.19    0.04       0.77  1.28    0.00  1.28      0.14    0.00
 L 3     0.14    0.06       0.66  1.28    0.00  1.28      0.20    0.00
 L 4     0.00    0.08       0.00  1.28    0.00  1.28      0.25    0.00


                         *******************************
                         *  TRUSS  -  LIVE LOAD TK527  *
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MEMBER FORCES AND ALLOWABLE STRESS RATINGS
I.F. = IMPACT FACTOR

| MEMBER ID | MEMBER LENGTH | DL FORCE | LL + IMPACT FORCE COMP | I.F. | TENS | I.F. | | ALLOWABLE FORCE COMP | TENS | RATING FACTORS | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| U 1U 2 | 13.00 | -15.8 | -94.8 | 1.28 | 0.0 | 1.28 | IR | -76.6 | 95.0 | IR | 0.642 |
| | | | | | | | OR | -95.6 | 133.7 | OR | 0.842 |
| U 2U 3 | 13.00 | -15.8 | -94.8 | 1.28 | 0.0 | 1.28 | IR | -76.6 | 95.0 | IR | 0.642 |
| | | | | | | | OR | -95.6 | 133.7 | OR | 0.842 |
| L 0L 1 | 13.42 | 11.9 | 0.0 | 1.28 | 75.3 | 1.28 | IR | -11.1 | 56.0 | IR | 0.585 |
| | | | | | | | OR | -13.8 | 78.8 | OR | 0.887 |
| L 1L 2 | 13.00 | 11.9 | 0.0 | 1.28 | 75.3 | 1.28 | IR | -11.8 | 56.0 | IR | 0.585 |
| | | | | | | | OR | -14.8 | 78.8 | OR | 0.887 |
| L 2L 3 | 13.00 | 12.1 | 0.0 | 1.28 | 76.3 | 1.28 | IR | -11.8 | 56.0 | IR | 0.576 |
| | | | | | | | OR | -14.8 | 78.8 | OR | 0.875 |
| L 3L 4 | 13.67 | 12.1 | 0.0 | 1.28 | 76.3 | 1.28 | IR | -10.7 | 56.0 | IR | 0.576 |
| | | | | | | | OR | -13.3 | 78.8 | OR | 0.875 |
| L 0U 1 | 15.14 | -13.4 | -84.9 | 1.28 | 0.0 | 1.00 | IR | -74.0 | 95.0 | IR | 0.713 |
| | | | | | | | OR | -92.3 | 133.7 | OR | 0.928 |
| L 1U 2 | 14.76 | 0.0 | 0.0 | 1.00 | 13.6 | 1.30 | IR | 0.0 | 7.0 | IR | 0.517 |
| | | | | | | | OR | 0.0 | 9.9 | OR | 0.727 |
| L 2U 3 | 14.76 | 4.3 | 0.0 | 1.00 | 48.6 | 1.30 | IR | -1.5 | 32.0 | IR | 0.571 |
| | | | | | | | OR | -1.9 | 45.0 | OR | 0.838 |
| U 3L 4 | 15.36 | -13.5 | -85.7 | 1.28 | 0.0 | 1.00 | IR | -73.7 | 95.0 | IR | 0.702 |
| | | | | | | | OR | -91.9 | 133.7 | OR | 0.914 |
| U 1L 2 | 14.76 | 4.4 | 0.0 | 1.00 | 49.3 | 1.30 | IR | -1.5 | 32.0 | IR | 0.560 |
| | | | | | | | OR | -1.9 | 45.0 | OR | 0.824 |
| U 2L 3 | 14.76 | 0.0 | 0.0 | 1.00 | 14.3 | 1.30 | IR | 0.0 | 7.0 | IR | 0.493 |
| | | | | | | | OR | 0.0 | 9.9 | OR | 0.693 |
| L 1U 1 | 7.00 | 3.7 | 0.0 | 1.00 | 38.5 | 1.30 | IR | -37.4 | 52.0 | IR | 1.254 |
| | | | | | | | OR | -46.6 | 73.1 | OR | 1.802 |
| L 2U 2 | 7.00 | -0.4 | -6.7 | 1.28 | 0.0 | 1.00 | IR | -37.4 | 52.0 | IR | 5.547 |
| | | | | | | | OR | -46.6 | 73.1 | OR | 6.931 |
| L 3U 3 | 7.00 | 3.7 | 0.0 | 1.00 | 38.6 | 1.30 | IR | -37.4 | 52.0 | IR | 1.250 |
| | | | | | | | OR | -46.6 | 73.1 | OR | 1.796 |

MEMBER U 2L 3 IS CRITICAL. SEE DETAILS BELOW.

| DL FORCE | LL + IMPACT FORCE COMP | I.F. | TENS | I.F. | | ALLOWABLE FORCE COMP | TENS | | RATING FACTOR | TONS |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | 0.0 | 1.00 | 14.3 | 1.30 | IR | 0.0 | 7.0 | IR | 0.493 | 19.72 |
| | | | | | OR | 0.0 | 9.9 | OR | 0.693 | 27.74 |

SUPPORT REACTIONS

| SUPPORT | D.L. | LL+I | I.F. |
|---|---|---|---|
| L 0 | 8.4 | 57.0 | 1.28 |
| L 4 | 8.4 | 57.0 | 1.28 |

PANEL POINT DEFLECTIONS

| PANEL | DEAD LOAD + TEMP | | LIVE LOAD + IMPACT | | | | | |
|---|---|---|---|---|---|---|---|---|
| POINT | VERT | HOR | VERT+ | I.F. | VERT- | I.F. | HOR+ | HOR- |
| L 0 | 0.00 | 0.00 | 0.00 | 1.28 | 0.00 | 1.28 | 0.00 | 0.00 |
| L 1 | 0.14 | 0.02 | 0.71 | 1.28 | 0.00 | 1.28 | 0.09 | 0.00 |
| L 2 | 0.19 | 0.04 | 0.86 | 1.28 | 0.00 | 1.28 | 0.16 | 0.00 |
| L 3 | 0.14 | 0.06 | 0.73 | 1.28 | 0.00 | 1.28 | 0.21 | 0.00 |
| L 4 | 0.00 | 0.08 | 0.00 | 1.28 | 0.00 | 1.28 | 0.28 | 0.00 |

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*  TRUSS  -  LIVE LOAD  ML80  *
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MEMBER FORCES AND ALLOWABLE STRESS RATINGS
I.F. = IMPACT FACTOR

| MEMBER ID | MEMBER LENGTH | DL FORCE | LL + IMPACT FORCE COMP | I.F. | TENS | I.F. | | ALLOWABLE FORCE COMP | TENS | RATING FACTORS | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| U 1U 2 | 13.00 | -15.8 | -100.3 | 1.28 | 0.0 | 1.28 | IR | -76.6 | 95.0 | IR | 0.607 |
| | | | | | | | OR | -95.6 | 133.7 | OR | 0.795 |
| U 2U 3 | 13.00 | -15.8 | -100.3 | 1.28 | 0.0 | 1.28 | IR | -76.6 | 95.0 | IR | 0.607 |
| | | | | | | | OR | -95.6 | 133.7 | OR | 0.795 |
| L 0L 1 | 13.42 | 11.9 | 0.0 | 1.28 | 76.9 | 1.28 | IR | -11.1 | 56.0 | IR | 0.573 |
| | | | | | | | OR | -13.8 | 78.8 | OR | 0.869 |
| L 1L 2 | 13.00 | 11.9 | 0.0 | 1.28 | 76.9 | 1.28 | IR | -11.8 | 56.0 | IR | 0.573 |
| | | | | | | | OR | -14.8 | 78.8 | OR | 0.869 |
| L 2L 3 | 13.00 | 12.1 | 0.0 | 1.28 | 77.8 | 1.28 | IR | -11.8 | 56.0 | IR | 0.565 |
| | | | | | | | OR | -14.8 | 78.8 | OR | 0.858 |
| L 3L 4 | 13.67 | 12.1 | 0.0 | 1.28 | 77.8 | 1.28 | IR | -10.7 | 56.0 | IR | 0.565 |
| | | | | | | | OR | -13.3 | 78.8 | OR | 0.858 |
| L 0U 1 | 15.14 | -13.4 | -86.8 | 1.28 | 0.0 | 1.00 | IR | -74.0 | 95.0 | IR | 0.698 |
| | | | | | | | OR | -92.3 | 133.7 | OR | 0.909 |
| L 1U 2 | 14.76 | 0.0 | 0.0 | 1.00 | 15.5 | 1.30 | IR | 0.0 | 7.0 | IR | 0.454 |
| | | | | | | | OR | 0.0 | 9.9 | OR | 0.638 |
| L 2U 3 | 14.76 | 4.3 | 0.0 | 1.00 | 51.6 | 1.30 | IR | -1.5 | 32.0 | IR | 0.538 |
| | | | | | | | OR | -1.9 | 45.0 | OR | 0.790 |
| U 3L 4 | 15.36 | -13.5 | -87.4 | 1.28 | 0.0 | 1.00 | IR | -73.7 | 95.0 | IR | 0.688 |
| | | | | | | | OR | -91.9 | 133.7 | OR | 0.897 |
| U 1L 2 | 14.76 | 4.4 | 0.0 | 1.00 | 52.2 | 1.30 | IR | -1.5 | 32.0 | IR | 0.528 |
| | | | | | | | OR | -1.9 | 45.0 | OR | 0.777 |
| U 2L 3 | 14.76 | 0.0 | 0.0 | 1.00 | 16.2 | 1.30 | IR | 0.0 | 7.0 | IR | 0.434 |
| | | | | | | | OR | 0.0 | 9.9 | OR | 0.611 |

```
L 1U 1   7.00      3.7      0.0  1.00    42.5  1.30  IR    -37.4    52.0  IR   1.138
                                                     OR    -46.6    73.1  OR   1.635

L 2U 2   7.00     -0.4     -7.6  1.28     0.0  1.00  IR    -37.4    52.0  IR   4.885
                                                     OR    -46.6    73.1  OR   6.104

L 3U 3   7.00      3.7      0.0  1.00    42.2  1.30  IR    -37.4    52.0  IR   1.145
                                                     OR    -46.6    73.1  OR   1.646


                   MEMBER U 2L 3 IS CRITICAL. SEE DETAILS BELOW.

       DL       LL + IMPACT FORCE      ALLOWABLE FORCE          RATING
     FORCE    COMP  I.F.   TENS  I.F.        COMP  TENS         FACTOR   TONS

      0.0     0.0  1.00   16.2  1.30  IR      0.0    7.0    IR   0.434  15.91
                                      OR      0.0    9.9    OR   0.611  22.37


                             SUPPORT REACTIONS

                      SUPPORT    D.L.      LL+I    I.F.
                        L 0       8.4      56.4    1.28
                        L 4       8.4      56.4    1.28


                    PANEL POINT DEFLECTIONS

PANEL  DEAD LOAD + TEMP             LIVE LOAD + IMPACT
POINT    VERT    HOR      VERT+  I.F.   VERT-  I.F.     HOR+    HOR-
 L 0     0.00    0.00      0.00  1.28    0.00  1.28     0.00    0.00
 L 1     0.14    0.02      0.72  1.28    0.00  1.28     0.09    0.00
 L 2     0.19    0.04      0.90  1.28    0.00  1.28     0.16    0.00
 L 3     0.14    0.06      0.74  1.28    0.00  1.28     0.21    0.00
 L 4     0.00    0.08      0.00  1.28    0.00  1.28     0.28    0.00


                +++++++++++++++++++++++++++++++++++++++++++++
                +                                           +
                +     S T R I N G E R   A N A L Y S I S     +
                +                                           +
                +++++++++++++++++++++++++++++++++++++++++++++

THE STRINGER LOCATED AT   6.07 FT. FROM THE CENTER LINE OF
GIRDER OR TRUSS IS CRITICAL.

                LIVE LOAD DISTRIBUTION FACTOR FOR MOMENT:  0.201

                LIVE LOAD DISTRIBUTION FACTOR FOR SHEAR:   0.500

                        DEAD LOADS ACTING ON THE STRINGER

                    STRINGER  SLAB    INPUT  TOTAL  TOTAL
                     WEIGHT  WEIGHT    DL1    DL1    DL2
                      0.015   0.000   0.045  0.060  0.045


                          STRINGER SECTION PROPERTIES
                           GROSS  MOMENT OF    C     SECTION    MODULUS
                    DEPTH   AREA   INERTIA  BOTTOM     TOP      BOTTOM
NON-COMPOSITE       10.00   4.48     67.30    5.00     13.46     13.46
```

DEFLECTIONS

SPAN 1 - LIVE LOAD IMPACT FACTOR FOR DEFLECTION: 1.30

| | DEAD LOAD | | ---------------- | LIVE LOAD + IMPACT | ---------------- | |
|---|---|---|---|---|---|---|
| X | DL1 | DL2 | H20 | HS20 | TK527 | ML80 |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.34 | 0.004 | 0.003 | 0.061 | 0.065 | 0.071 | 0.082 |
| 2.68 | 0.007 | 0.006 | 0.116 | 0.124 | 0.133 | 0.152 |
| 4.03 | 0.010 | 0.007 | 0.157 | 0.169 | 0.182 | 0.205 |
| 5.37 | 0.011 | 0.008 | 0.179 | 0.194 | 0.210 | 0.234 |
| 6.71 | 0.011 | 0.008 | 0.184 | 0.200 | 0.204 | 0.247 |
| 8.05 | 0.010 | 0.007 | 0.167 | 0.184 | 0.190 | 0.226 |
| 9.39 | 0.007 | 0.005 | 0.133 | 0.148 | 0.156 | 0.184 |
| 10.74 | 0.005 | 0.003 | 0.089 | 0.101 | 0.100 | 0.120 |
| 12.08 | 0.002 | 0.001 | 0.042 | 0.050 | 0.047 | 0.058 |
| 13.42 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SPAN 2 - LIVE LOAD IMPACT FACTOR FOR DEFLECTION: 1.30

| | DEAD LOAD | | ---------------- | LIVE LOAD + IMPACT | ---------------- | |
|---|---|---|---|---|---|---|
| X | DL1 | DL2 | H20 | HS20 | TK527 | ML80 |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.30 | -0.000 | -0.000 | 0.032 | 0.037 | 0.035 | 0.040 |
| 2.60 | 0.000 | 0.000 | 0.066 | 0.077 | 0.070 | 0.081 |
| 3.90 | 0.001 | 0.001 | 0.094 | 0.112 | 0.097 | 0.114 |
| 5.20 | 0.002 | 0.002 | 0.113 | 0.135 | 0.116 | 0.142 |
| 6.50 | 0.003 | 0.002 | 0.121 | 0.146 | 0.119 | 0.149 |
| 7.80 | 0.003 | 0.002 | 0.111 | 0.138 | 0.107 | 0.138 |
| 9.10 | 0.002 | 0.002 | 0.091 | 0.116 | 0.091 | 0.111 |
| 10.40 | 0.001 | 0.001 | 0.062 | 0.082 | 0.065 | 0.076 |
| 11.70 | 0.000 | 0.000 | 0.030 | 0.041 | 0.033 | 0.038 |
| 13.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SPAN 3 - LIVE LOAD IMPACT FACTOR FOR DEFLECTION: 1.30

| | DEAD LOAD | | ---------------- | LIVE LOAD + IMPACT | ---------------- | |
|---|---|---|---|---|---|---|
| X | DL1 | DL2 | H20 | HS20 | TK527 | ML80 |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.30 | 0.000 | 0.000 | 0.030 | 0.041 | 0.033 | 0.038 |
| 2.60 | 0.001 | 0.001 | 0.062 | 0.082 | 0.066 | 0.076 |
| 3.90 | 0.002 | 0.001 | 0.092 | 0.116 | 0.091 | 0.111 |
| 5.20 | 0.002 | 0.002 | 0.111 | 0.138 | 0.106 | 0.138 |
| 6.50 | 0.002 | 0.002 | 0.121 | 0.146 | 0.119 | 0.149 |
| 7.80 | 0.002 | 0.001 | 0.114 | 0.135 | 0.117 | 0.143 |
| 9.10 | 0.001 | 0.001 | 0.094 | 0.112 | 0.098 | 0.114 |
| 10.40 | 0.000 | 0.000 | 0.066 | 0.077 | 0.071 | 0.081 |
| 11.70 | -0.000 | -0.000 | 0.032 | 0.038 | 0.036 | 0.041 |
| 13.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SPAN 4 - LIVE LOAD IMPACT FACTOR FOR DEFLECTION: 1.30

| | DEAD LOAD | | ---------------- | LIVE LOAD + IMPACT | ---------------- | |
|---|---|---|---|---|---|---|
| X | DL1 | DL2 | H20 | HS20 | TK527 | ML80 |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.37 | 0.002 | 0.002 | 0.044 | 0.052 | 0.050 | 0.062 |
| 2.73 | 0.005 | 0.004 | 0.093 | 0.106 | 0.106 | 0.128 |
| 4.10 | 0.008 | 0.006 | 0.140 | 0.156 | 0.166 | 0.196 |
| 5.47 | 0.011 | 0.008 | 0.176 | 0.193 | 0.202 | 0.241 |
| 6.83 | 0.012 | 0.009 | 0.194 | 0.211 | 0.217 | 0.263 |
| 8.20 | 0.012 | 0.009 | 0.189 | 0.204 | 0.223 | 0.250 |
| 9.57 | 0.011 | 0.008 | 0.166 | 0.178 | 0.193 | 0.218 |
| 10.94 | 0.008 | 0.006 | 0.122 | 0.130 | 0.142 | 0.163 |

```
 12.30  0.004  0.003  0.064  0.069  0.075  0.087
 13.67  0.000  0.000  0.000  0.000  0.000  0.000


                      **********************************
                      *  STRINGER  -  LIVE LOAD  H20   *
                      **********************************

                                MAXIMUM REACTIONS

                                                   REACTIONS     MOMENTS
    SUPPORT    DL1      DL2   +(LL+I)    -(LL+I)  +I.F. -I.F.  +I.F. -I.F.
       1       0.3      0.2     20.8       -0.6 L  1.30  1.30
       2       0.9      0.7     19.7 L     -1.0 L  1.30  1.30   1.30  1.30
       3       0.7      0.5     20.7       -1.7 L  1.30  1.30   1.30  1.30
       4       0.9      0.7     19.8 L     -1.0 L  1.30  1.30   1.30  1.30
       5       0.3      0.2     20.8       -0.6 L  1.30  1.30


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
      ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
      SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.


                         UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
          DL1    DL2   +(LL+I)  -(LL+I)  I   DL1    DL2   +(LL+I) -(LL+I)
     X   MOMENT  MOMENT MOMENT   MOMENT  M  SHEAR  SHEAR  SHEAR   SHEAR  I.F.
    0.00    0.0     0.0    0.0      0.0  S    0.3    0.2   20.8    -0.6L 1.30
          SIMULT SHEAR    0.0      0.0      SIMULT MOM     0.0     0.0
    1.34    0.4     0.3    9.8     -0.8       0.2    0.2    7.3    -1.2  1.30
          SIMULT SHEAR    7.3     -0.6      SIMULT MOM     9.8     9.7
    2.68    0.6     0.5   16.8     -1.6       0.2    0.1    6.3    -2.2  1.30
          SIMULT SHEAR    6.3     -0.6      SIMULT MOM    16.8    16.4
    4.03    0.8     0.6   21.1     -2.4       0.1    0.1    5.2    -3.3  1.30
          SIMULT SHEAR    5.2     -0.6      SIMULT MOM    21.1    20.4
    5.37    0.8     0.6   22.8     -3.2      -0.0   -0.0    4.3    -4.3  1.30
          SIMULT SHEAR    4.3     -0.6      SIMULT MOM    22.8    22.0
    6.71    0.8     0.6   22.3     -3.9      -0.1   -0.1    3.3    -5.2  1.30
          SIMULT SHEAR   -5.0     -0.6      SIMULT MOM    22.3    21.4
    8.05    0.6     0.5   19.9     -4.7      -0.2   -0.1    2.5    -6.0  1.30
          SIMULT SHEAR   -5.9     -0.6      SIMULT MOM    19.9    19.1
    9.39    0.3     0.3   15.9     -5.5      -0.2   -0.2    1.7    -6.7  1.30
          SIMULT SHEAR   -6.7     -0.6      SIMULT MOM    15.9    15.3
   10.74   -0.0    -0.0   10.9     -6.6      -0.3   -0.2    1.0    -7.4  1.30
          SIMULT SHEAR   -7.3     -1.5      SIMULT MOM    10.9    10.5
   12.08   -0.5    -0.4    5.4     -8.9L     -0.4   -0.3    0.4    -7.9  1.30
          SIMULT SHEAR   -7.9     -3.6      SIMULT MOM     5.4     5.3
   13.42   -1.1    -0.8    2.1    -14.8L S   -0.5   -0.4    0.2L  -20.8  1.30
          SIMULT SHEAR    0.2     -5.1      SIMULT MOM     2.2     0.0

SPAN  2 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
          DL1    DL2   +(LL+I)  -(LL+I)  I   DL1    DL2   +(LL+I) -(LL+I)
     X   MOMENT  MOMENT MOMENT   MOMENT  M  SHEAR  SHEAR  SHEAR   SHEAR  I.F.
    0.00   -1.1    -0.8    2.1    -14.8L S    0.4    0.3   20.8    -0.9L 1.30
          SIMULT SHEAR   -0.8      4.8      SIMULT MOM     0.0     2.2
    1.30   -0.6    -0.5    6.1    -10.8       0.3    0.3    7.7    -0.9L 1.30
          SIMULT SHEAR    7.5      1.8      SIMULT MOM     5.5     1.2
    2.60   -0.2    -0.2   11.2     -8.7       0.3    0.2    7.0    -1.6  1.30
          SIMULT SHEAR    6.7      1.1      SIMULT MOM    10.4    11.2
```

```
   3.90     0.1     0.1   15.1     -7.3       0.2    0.1    6.2    -2.5  1.30
           SIMULT SHEAR    5.8      1.1      SIMULT MOM    14.3    15.1
   5.20     0.3     0.2   17.5     -6.0L      0.1    0.1    5.3    -3.5  1.30
           SIMULT SHEAR    4.9      0.2      SIMULT MOM    16.9    17.5
   6.50     0.4     0.3   18.1     -5.7L      0.0    0.0    4.4    -4.4  1.30
           SIMULT SHEAR   -4.4      0.2      SIMULT MOM    17.9    18.1
   7.80     0.4     0.3   17.1     -5.5L     -0.0   -0.0    3.4    -5.4  1.30
           SIMULT SHEAR   -4.9      0.2      SIMULT MOM    17.1    17.0
   9.10     0.2     0.2   14.7     -5.4      -0.1   -0.1    2.5    -6.3  1.30
           SIMULT SHEAR   -5.9     -0.8      SIMULT MOM    14.7    14.4
  10.40     0.0     0.0   10.8     -6.9      -0.2   -0.1    1.6    -7.1  1.30
           SIMULT SHEAR   -6.8     -1.5      SIMULT MOM    10.8    10.3
  11.70    -0.3    -0.2    5.8     -8.9      -0.3   -0.2    1.2L   -7.8  1.30
           SIMULT SHEAR   -7.6     -1.5      SIMULT MOM     2.2     5.5
  13.00    -0.7    -0.5    3.2    -13.5L S   -0.4   -0.3    1.2L  -20.8  1.30
           SIMULT SHEAR    0.3     -4.7      SIMULT MOM     3.6     0.3

SPAN  3 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
           DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
    X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
   0.00    -0.7    -0.5    3.2    -13.5L S    0.4    0.3   20.8     -1.2L 1.30
           SIMULT SHEAR   -1.2      4.7      SIMULT MOM     0.3     3.7
   1.30    -0.3    -0.2    5.8     -8.9       0.3    0.2    7.8     -1.2L 1.30
           SIMULT SHEAR    7.6      1.5      SIMULT MOM     5.4     2.2
   2.60     0.0     0.0   10.8     -6.9       0.2    0.1    7.1    -1.6  1.30
           SIMULT SHEAR    6.8      1.5      SIMULT MOM    10.3    10.8
   3.90     0.2     0.2   14.7     -5.4       0.1    0.1    6.3    -2.5  1.30
           SIMULT SHEAR    5.9      0.8      SIMULT MOM    14.4    14.7
   5.20     0.3     0.3   17.1     -5.5L      0.0    0.0    5.4    -3.4  1.30
           SIMULT SHEAR    4.9     -0.2      SIMULT MOM    17.1    17.1
   6.50     0.3     0.3   18.2     -5.8L     -0.0   -0.0    4.4    -4.4  1.30
           SIMULT SHEAR    4.4     -0.2      SIMULT MOM    18.2    17.9
   7.80     0.3     0.2   17.5     -6.1L     -0.1   -0.1    3.5    -5.3  1.30
           SIMULT SHEAR   -4.9     -0.2      SIMULT MOM    17.5    16.9
   9.10     0.1     0.0   15.2     -7.5      -0.2   -0.1    2.6    -6.2  1.30
           SIMULT SHEAR   -5.8     -1.2      SIMULT MOM    15.2    14.3
  10.40    -0.3    -0.2   11.3     -9.0      -0.3   -0.2    1.7    -7.0  1.30
           SIMULT SHEAR   -6.7     -1.2      SIMULT MOM    11.3    10.3
  11.70    -0.7    -0.5    6.1    -11.1      -0.3   -0.3    0.9L   -7.8  1.30
           SIMULT SHEAR   -7.5     -1.8      SIMULT MOM     1.2     5.4
  13.00    -1.2    -0.9    2.1    -15.0L S   -0.4   -0.3    0.9L  -20.8  1.30
           SIMULT SHEAR    0.8     -4.8      SIMULT MOM     2.2     0.0

SPAN  4 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
           DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
    X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
   0.00    -1.2    -0.9    2.1    -15.0L S    0.5    0.4   20.8     -0.2L 1.30
           SIMULT SHEAR   -0.2      5.1      SIMULT MOM     0.0     2.2
   1.37    -0.5    -0.4    5.5     -9.0L      0.4    0.3    7.9    -0.4  1.30
           SIMULT SHEAR    7.9      3.6      SIMULT MOM     5.4     5.5
   2.73    -0.0    -0.0   11.1     -6.6       0.3    0.2    7.4    -1.0  1.30
           SIMULT SHEAR    7.3      1.5      SIMULT MOM    10.7    11.1
   4.10     0.4     0.3   16.2     -5.5       0.2    0.2    6.7    -1.7  1.30
           SIMULT SHEAR    6.7      0.6      SIMULT MOM    15.5    16.2
   5.47     0.7     0.5   20.2     -4.7       0.2    0.1    6.0    -2.5  1.30
           SIMULT SHEAR    5.9      0.6      SIMULT MOM    19.4    20.2
   6.83     0.8     0.6   22.7     -3.9       0.1    0.1    5.2    -3.3  1.30
           SIMULT SHEAR    5.0      0.6      SIMULT MOM    21.8    22.7
   8.20     0.9     0.7   23.2     -3.1       0.0    0.0    4.3    -4.2  1.30
           SIMULT SHEAR   -4.2      0.6      SIMULT MOM    22.4    23.2
   9.57     0.8     0.6   21.4     -2.3      -0.1   -0.1    3.3    -5.2  1.30
```

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | SIMULT SHEAR | | -5.2 | 0.6 | | SIMULT MOM | | 20.8 | 21.4 | |
| 10.94 | 0.7 | 0.5 | 17.1 | -1.6 | | -0.2 | -0.1 | 2.2 | -6.2 | 1.30 |
| | SIMULT SHEAR | | -6.2 | 0.6 | | SIMULT MOM | | 16.7 | 17.1 | |
| 12.30 | 0.4 | 0.3 | 10.0 | -0.8 | | -0.2 | -0.2 | 1.2 | -7.3 | 1.30 |
| | SIMULT SHEAR | | -7.3 | 0.6 | | SIMULT MOM | | 9.9 | 10.0 | |
| 13.67 | 0.0 | 0.0 | 0.0 | 0.0 | S | -0.3 | -0.2 | 0.6L | -20.8 | 1.30 |
| | SIMULT SHEAR | | 0.0 | 0.0 | | SIMULT MOM | | 0.0 | 0.0 | |

FLEXURAL STRESSES - BEAM

SPAN 1
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.34 | -0.334 | -0.247 | -8.735 | 0.703 | 0.334 | 0.247 | 8.735 | -0.703 |
| 2.68 | -0.572 | -0.423 | -14.963 | 1.405 | 0.572 | 0.423 | 14.963 | -1.405 |
| 4.03 | -0.713 | -0.527 | -18.782 | 2.108 | 0.713 | 0.527 | 18.782 | -2.108 |
| 5.37 | -0.757 | -0.560 | -20.356 | 2.810 | 0.757 | 0.560 | 20.356 | -2.810 |
| 6.71 | -0.704 | -0.521 | -19.912 | 3.513 | 0.704 | 0.521 | 19.912 | -3.513 |
| 8.05 | -0.555 | -0.410 | -17.746 | 4.216 | 0.555 | 0.410 | 17.746 | -4.216 |
| 9.39 | -0.309 | -0.228 | -14.214 | 4.918 | 0.309 | 0.228 | 14.214 | -4.918 |
| 10.74 | 0.034 | 0.025 | -9.742 | 5.926 | -0.034 | -0.025 | 9.742 | -5.926 |
| 12.08 | 0.474 | 0.351 | -4.819 | 7.938 | -0.474 | -0.351 | 4.819 | -7.938 |
| 13.42 | 1.010 | 0.747 | -1.878 | 13.235 | -1.010 | -0.747 | 1.878 | -13.235 |

SPAN 2
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 1.010 | 0.747 | -1.878 | 13.235 | -1.010 | -0.747 | 1.878 | -13.235 |
| 1.30 | 0.562 | 0.416 | -5.427 | 9.644 | -0.562 | -0.416 | 5.427 | -9.644 |
| 2.60 | 0.204 | 0.151 | -9.996 | 7.796 | -0.204 | -0.151 | 9.996 | -7.796 |
| 3.90 | -0.062 | -0.046 | -13.476 | 6.474 | 0.062 | 0.046 | 13.476 | -6.474 |
| 5.20 | -0.238 | -0.176 | -15.581 | 5.313 | 0.238 | 0.176 | 15.581 | -5.313 |
| 6.50 | -0.324 | -0.239 | -16.159 | 5.092 | 0.324 | 0.239 | 16.159 | -5.092 |
| 7.80 | -0.318 | -0.235 | -15.263 | 4.872 | 0.318 | 0.235 | 15.263 | -4.872 |
| 9.10 | -0.222 | -0.164 | -13.098 | 4.781 | 0.222 | 0.164 | 13.098 | -4.781 |
| 10.40 | -0.035 | -0.026 | -9.629 | 6.157 | 0.035 | 0.026 | 9.629 | -6.157 |
| 11.70 | 0.243 | 0.180 | -5.183 | 7.932 | -0.243 | -0.180 | 5.183 | -7.932 |
| 13.00 | 0.612 | 0.453 | -2.857 | 12.054 | -0.612 | -0.453 | 2.857 | -12.054 |

SPAN 3
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.612 | 0.453 | -2.857 | 12.054 | -0.612 | -0.453 | 2.857 | -12.054 |
| 1.30 | 0.246 | 0.182 | -5.182 | 7.919 | -0.246 | -0.182 | 5.182 | -7.919 |
| 2.60 | -0.030 | -0.022 | -9.626 | 6.145 | 0.030 | 0.022 | 9.626 | -6.145 |
| 3.90 | -0.214 | -0.158 | -13.094 | 4.786 | 0.214 | 0.158 | 13.094 | -4.786 |
| 5.20 | -0.308 | -0.228 | -15.257 | 4.927 | 0.308 | 0.228 | 15.257 | -4.927 |
| 6.50 | -0.311 | -0.230 | -16.183 | 5.182 | 0.311 | 0.230 | 16.183 | -5.182 |
| 7.80 | -0.223 | -0.165 | -15.616 | 5.437 | 0.223 | 0.165 | 15.616 | -5.437 |
| 9.10 | -0.045 | -0.033 | -13.518 | 6.650 | 0.045 | 0.033 | 13.518 | -6.650 |
| 10.40 | 0.224 | 0.166 | -10.037 | 8.009 | -0.224 | -0.166 | 10.037 | -8.009 |
| 11.70 | 0.584 | 0.432 | -5.456 | 9.888 | -0.584 | -0.432 | 5.456 | -9.888 |
| 13.00 | 1.035 | 0.766 | -1.861 | 13.416 | -1.035 | -0.766 | 1.861 | -13.416 |

SPAN 4
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 1.035 | 0.766 | -1.861 | 13.416 | -1.035 | -0.766 | 1.861 | -13.416 |
| 1.37 | 0.480 | 0.355 | -4.873 | 7.999 | -0.480 | -0.355 | 4.873 | -7.999 |
| 2.73 | 0.025 | 0.019 | -9.870 | 5.859 | -0.025 | -0.019 | 9.870 | -5.859 |
| 4.10 | -0.329 | -0.244 | -14.420 | 4.869 | 0.329 | 0.244 | 14.420 | -4.869 |
| 5.47 | -0.583 | -0.432 | -18.022 | 4.173 | 0.583 | 0.432 | 18.022 | -4.173 |
| 6.83 | -0.737 | -0.545 | -20.239 | 3.478 | 0.737 | 0.545 | 20.239 | -3.478 |
| 8.20 | -0.790 | -0.585 | -20.703 | 2.782 | 0.790 | 0.585 | 20.703 | -2.782 |
| 9.57 | -0.743 | -0.550 | -19.113 | 2.087 | 0.743 | 0.550 | 19.113 | -2.087 |
| 10.94 | -0.596 | -0.441 | -15.232 | 1.391 | 0.596 | 0.441 | 15.232 | -1.391 |
| 12.30 | -0.348 | -0.258 | -8.895 | 0.696 | 0.348 | 0.258 | 8.895 | -0.696 |
| 13.67 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

| | | SHEAR STRESSES | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.146 | 0.108 | 9.495 | -0.284 | 1.000 | 1.03 V | 1.41 V |
| 1.34 | 0.109 | 0.081 | 3.332 | -0.530 | 1.000 | 1.82 T | 2.51 T |
| 2.68 | 0.072 | 0.053 | 2.854 | -1.026 | 1.000 | 1.04 T | 1.44 T |
| 4.03 | 0.035 | 0.026 | 2.389 | -1.499 | 1.000 | 0.81 T | 1.13 T |
| 5.37 | -0.002 | -0.001 | 1.942 | -1.944 | 1.000 | 0.75 T | 1.04 T |
| 6.71 | -0.039 | -0.029 | 1.519 | -2.358 | 1.000 | 0.77 T | 1.07 T |
| 8.05 | -0.075 | -0.056 | 1.128 | -2.737 | 1.000 | 0.88 T | 1.21 T |
| 9.39 | -0.112 | -0.083 | 0.775 | -3.075 | 1.000 | 1.12 T | 1.55 T |
| 10.74 | -0.149 | -0.110 | 0.465 | -3.370 | 1.000 | 1.70 T | 2.32 T |
| 12.08 | -0.186 | -0.138 | 0.204 | -3.616 | 0.801 | 1.56 B | 2.17 B |
| 13.42 | -0.223 | -0.165 | 0.076 | -9.495 | 0.801 | 0.87 B | 1.23 B |

SPAN 2
=======

| | | SHEAR STRESSES | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.194 | 0.144 | 9.495 | -0.399 | 0.727 | 0.77 B | 1.10 B |
| 1.30 | 0.159 | 0.117 | 3.537 | -0.402 | 0.836 | 1.33 B | 1.85 B |
| 2.60 | 0.123 | 0.091 | 3.209 | -0.752 | 1.000 | 1.69 T | 2.29 T |
| 3.90 | 0.087 | 0.065 | 2.834 | -1.162 | 1.000 | 1.22 T | 1.66 T |
| 5.20 | 0.051 | 0.038 | 2.425 | -1.591 | 1.000 | 1.03 T | 1.42 T |
| 6.50 | 0.016 | 0.012 | 1.997 | -2.025 | 1.000 | 0.99 T | 1.36 T |
| 7.80 | -0.020 | -0.015 | 1.562 | -2.450 | 1.000 | 1.04 T | 1.44 T |
| 9.10 | -0.056 | -0.041 | 1.136 | -2.854 | 1.000 | 1.23 T | 1.69 T |
| 10.40 | -0.092 | -0.068 | 0.731 | -3.224 | 1.000 | 1.71 T | 2.33 T |
| 11.70 | -0.127 | -0.094 | 0.541 | -3.545 | 0.792 | 1.59 B | 2.19 B |
| 13.00 | -0.163 | -0.121 | 0.538 | -9.483 | 0.684 | 0.85 B | 1.19 B |

SPAN 3
=======

| | | SHEAR STRESSES | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.162 | 0.120 | 9.483 | -0.552 | 0.684 | 0.85 B | 1.19 B |
| 1.30 | 0.126 | 0.093 | 3.546 | -0.556 | 0.791 | 1.59 B | 2.19 B |
| 2.60 | 0.091 | 0.067 | 3.225 | -0.731 | 1.000 | 1.71 T | 2.33 T |
| 3.90 | 0.055 | 0.041 | 2.857 | -1.136 | 1.000 | 1.23 T | 1.69 T |
| 5.20 | 0.019 | 0.014 | 2.453 | -1.563 | 1.000 | 1.05 T | 1.44 T |
| 6.50 | -0.017 | -0.012 | 2.028 | -1.998 | 1.000 | 0.99 T | 1.36 T |
| 7.80 | -0.052 | -0.039 | 1.594 | -2.427 | 1.000 | 1.03 T | 1.42 T |
| 9.10 | -0.088 | -0.065 | 1.165 | -2.836 | 1.000 | 1.21 T | 1.66 T |
| 10.40 | -0.124 | -0.092 | 0.754 | -3.213 | 1.000 | 1.68 T | 2.28 T |
| 11.70 | -0.160 | -0.118 | 0.401 | -3.541 | 0.838 | 1.30 B | 1.80 B |
| 13.00 | -0.195 | -0.145 | 0.397 | -9.495 | 0.729 | 0.76 B | 1.09 B |

SPAN 4
=======

| X | SHEAR STRESSES DL1 | DL2 | +(LL+I) | -(LL+I) | ALLOW COMPR REDUCTION | RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.227 | 0.168 | 9.495 | -0.074 | 0.785 | 0.83 B | 1.18 B |
| 1.37 | 0.189 | 0.140 | 3.618 | -0.203 | 0.785 | 1.51 B | 2.10 B |
| 2.73 | 0.152 | 0.112 | 3.372 | -0.462 | 1.000 | 1.68 T | 2.28 T |
| 4.10 | 0.114 | 0.084 | 3.078 | -0.772 | 1.000 | 1.10 T | 1.52 T |
| 5.47 | 0.076 | 0.056 | 2.740 | -1.125 | 1.000 | 0.86 T | 1.19 T |
| 6.83 | 0.039 | 0.029 | 2.361 | -1.516 | 1.000 | 0.75 T | 1.05 T |
| 8.20 | 0.001 | 0.001 | 1.946 | -1.939 | 1.000 | 0.73 T | 1.02 T |
| 9.57 | -0.036 | -0.027 | 1.499 | -2.386 | 1.000 | 0.80 T | 1.11 T |
| 10.94 | -0.074 | -0.055 | 1.024 | -2.853 | 1.000 | 1.02 T | 1.41 T |
| 12.30 | -0.112 | -0.083 | 0.526 | -3.332 | 1.000 | 1.79 T | 2.46 T |
| 13.67 | -0.149 | -0.110 | 0.275 | -9.495 | 1.000 | 1.03 V | 1.41 V |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 33.6 B | 26.9 | 38.1 | 0.83 V | 1.38 V | 39.6 | 0.83 V | 1.38 V |
| 1.34 | 33.6 B | 26.9 | 38.1 | 1.55 T | 2.58 T | 39.6 | 1.61 O | 2.68 O |
| 2.68 | 33.6 B | 26.9 | 38.1 | 0.89 T | 1.48 T | 39.6 | 0.92 O | 1.54 O |
| 4.03 | 33.6 B | 26.9 | 38.1 | 0.70 T | 1.16 T | 39.6 | 0.73 O | 1.21 O |
| 5.37 | 33.6 B | 26.9 | 38.1 | 0.64 T | 1.07 T | 39.6 | 0.67 O | 1.11 O |
| 6.71 | 33.6 B | 26.9 | 38.1 | 0.66 T | 1.10 T | 39.6 | 0.69 O | 1.14 O |
| 8.05 | 33.6 B | 26.9 | 38.1 | 0.75 T | 1.25 T | 39.6 | 0.78 O | 1.30 O |
| 9.39 | 33.6 B | 26.9 | 38.1 | 0.95 T | 1.59 T | 39.6 | 0.99 O | 1.65 O |
| 10.74 | 33.6 B | 26.9 | 38.1 | 1.42 T | 2.37 T | 39.6 | 1.48 O | 2.47 O |
| 12.08 | -27.0 U | -21.6 | 38.1 | 1.34 B | 2.23 B | | | |
| 13.42 | -27.0 U | -21.6 | 38.1 | 0.76 B | 1.26 B | | | |

SPAN 2
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -24.5 U | -19.6 | 38.1 | 0.68 B | 1.14 B | | | |
| 1.30 | -28.1 U | -22.5 | 38.1 | 1.14 B | 1.90 B | | | |
| 2.60 | 33.6 B | 26.9 | 38.1 | 1.41 T | 2.34 T | 39.6 | 1.46 O | 2.44 O |
| 3.90 | 33.6 B | 26.9 | 38.1 | 1.02 T | 1.70 T | 39.6 | 1.06 O | 1.77 O |
| 5.20 | 33.6 B | 26.9 | 38.1 | 0.87 T | 1.45 T | 39.6 | 0.91 O | 1.51 O |
| 6.50 | 33.6 B | 26.9 | 38.1 | 0.84 T | 1.39 T | 39.6 | 0.87 O | 1.45 O |
| 7.80 | 33.6 B | 26.9 | 38.1 | 0.89 T | 1.48 T | 39.6 | 0.92 O | 1.54 O |
| 9.10 | 33.6 B | 26.9 | 38.1 | 1.04 T | 1.73 T | 39.6 | 1.08 O | 1.80 O |
| 10.40 | 33.6 B | 26.9 | 38.1 | 1.43 T | 2.39 T | 39.6 | 1.49 O | 2.49 O |
| 11.70 | -26.7 U | -21.3 | 38.1 | 1.35 B | 2.25 B | | | |
| 13.00 | -23.0 U | -18.4 | 38.1 | 0.73 B | 1.22 B | | | |

SPAN 3
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|---|

```
  0.00    -23.0 U   -18.4    38.1   0.73 B   1.22 B
  1.30    -26.6 U   -21.3    38.1   1.35 B   2.25 B
  2.60     33.6 B    26.9    38.1   1.44 T   2.39 T     39.6   1.49 O   2.49 O
  3.90     33.6 B    26.9    38.1   1.04 T   1.73 T     39.6   1.08 O   1.80 O
  5.20     33.6 B    26.9    38.1   0.89 T   1.48 T     39.6   0.92 O   1.54 O
  6.50     33.6 B    26.9    38.1   0.84 T   1.39 T     39.6   0.87 O   1.45 O
  7.80     33.6 B    26.9    38.1   0.87 T   1.45 T     39.6   0.91 O   1.51 O
  9.10     33.6 B    26.9    38.1   1.02 T   1.70 T     39.6   1.06 O   1.77 O
 10.40     33.6 B    26.9    38.1   1.40 T   2.34 T     39.6   1.46 O   2.43 O
 11.70    -28.2 U   -22.6    38.1   1.11 B   1.85 B
 13.00    -24.5 U   -19.6    38.1   0.67 B   1.12 B

SPAN  4
=======
        NON-COMP OVERLOAD           NON-COMPACT     COMPACT      COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS    MOMENT   RATING FACTORS
   X    STRENGTH STRENGTH STRENGTH   IR       OR    STRENGTH    IR       OR
  0.00    -26.4 U   -21.1    38.1   0.73 B   1.22 B
  1.37    -26.4 U   -21.1    38.1   1.30 B   2.16 B
  2.73     33.6 B    26.9    38.1   1.41 T   2.34 T     39.6   1.46 O   2.44 O
  4.10     33.6 B    26.9    38.1   0.94 T   1.56 T     39.6   0.97 O   1.62 O
  5.47     33.6 B    26.9    38.1   0.73 T   1.22 T     39.6   0.77 O   1.28 O
  6.83     33.6 B    26.9    38.1   0.65 T   1.08 T     39.6   0.67 O   1.12 O
  8.20     33.6 B    26.9    38.1   0.63 T   1.05 T     39.6   0.66 O   1.09 O
  9.57     33.6 B    26.9    38.1   0.68 T   1.14 T     39.6   0.71 O   1.19 O
 10.94     33.6 B    26.9    38.1   0.87 T   1.45 T     39.6   0.90 O   1.51 O
 12.30     33.6 B    26.9    38.1   1.52 T   2.53 T     39.6   1.58 O   2.63 O
 13.67     33.6 B    26.9    38.1   0.83 V   1.38 V     39.6   0.83 V   1.38 V


                      **********************************
                      *  STRINGER  -  LIVE LOAD  HS20  *
                      **********************************

                                MAXIMUM REACTIONS

                                                       REACTIONS     MOMENTS
    SUPPORT    DL1      DL2    +(LL+I)     -(LL+I)   +I.F. -I.F.  +I.F. -I.F.
       1       0.3      0.2     21.0        -0.7      1.30  1.30
       2       0.9      0.7     19.7 L      -1.1      1.30  1.30   1.30  1.30
       3       0.7      0.5     20.8        -1.7 L    1.30  1.30   1.30  1.30
       4       0.9      0.7     19.8 L      -1.1      1.30  1.30   1.30  1.30
       5       0.3      0.2     20.9        -0.7      1.30  1.30


NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD
      ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER
      SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.


                         UNFACTORED MOMENTS AND SHEARS

SPAN  1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR   SHEAR  I.F.
    0.00     0.0     0.0    0.0      0.0   S    0.3    0.2   21.0     -0.7  1.30
           SIMULT SHEAR    0.0      0.0       SIMULT MOM     0.0      0.0
    1.34     0.4     0.3   10.0     -0.9        0.2    0.2    7.5     -1.7  1.30
           SIMULT SHEAR    7.5     -0.7       SIMULT MOM    10.0      9.0
    2.68     0.6     0.5   17.2     -1.8        0.2    0.1    6.4     -2.7  1.30
           SIMULT SHEAR    6.4     -0.7       SIMULT MOM    17.2     15.1
    4.03     0.8     0.6   21.8     -2.8        0.1    0.1    5.4     -3.8  1.30
```

```
            SIMULT SHEAR    5.4     -0.7      SIMULT MOM    21.8    18.5
    5.37     0.8     0.6   23.7     -3.7      -0.0   -0.0    4.4    -4.7  1.30
            SIMULT SHEAR    4.4     -0.7      SIMULT MOM    23.7    19.6
    6.71     0.8     0.6   23.5     -4.6      -0.1   -0.1    3.5    -5.6  1.30
            SIMULT SHEAR   -4.9     -0.7      SIMULT MOM    23.5    18.8
    8.05     0.6     0.5   21.3     -5.5      -0.2   -0.1    2.6    -6.3  1.30
            SIMULT SHEAR   -5.7     -0.7      SIMULT MOM    21.3    16.5
    9.39     0.3     0.3   17.5     -6.5      -0.2   -0.2    1.9    -7.0  1.30
            SIMULT SHEAR   -6.5     -0.7      SIMULT MOM    17.5    13.2
   10.74    -0.0    -0.0   12.8     -7.4      -0.3   -0.2    1.2    -7.5  1.30
            SIMULT SHEAR   -7.2     -0.7      SIMULT MOM    12.8     9.3
   12.08    -0.5    -0.4    7.5    -12.0      -0.4   -0.3    0.6    -8.0  1.30
            SIMULT SHEAR   -7.7     -4.7      SIMULT MOM     7.5     4.9
   13.42    -1.1    -0.8    2.3    -17.6  S   -0.5   -0.4    0.2   -20.9  1.30
            SIMULT SHEAR    0.2     -4.2      SIMULT MOM     2.3    -0.7

SPAN  2 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT    MOMENT   M  SHEAR   SHEAR   SHEAR    SHEAR  I.F.
    0.00    -1.1    -0.8    2.3    -17.6  S    0.4     0.3   21.1     -0.9  1.30
            SIMULT SHEAR   -0.9      6.0      SIMULT MOM    -0.7     2.3
    1.30    -0.6    -0.5    6.9    -12.7       0.3     0.3    8.1     -1.5  1.30
            SIMULT SHEAR    6.9      4.3      SIMULT MOM     5.1     6.7
    2.60    -0.2    -0.2   11.2     -8.8       0.3     0.2    7.4     -2.3  1.30
            SIMULT SHEAR    7.2      0.2      SIMULT MOM     7.9    11.2
    3.90     0.1     0.1   15.8     -8.5       0.2     0.1    6.7     -3.1  1.30
            SIMULT SHEAR    6.3      0.2      SIMULT MOM    11.1    14.3
    5.20     0.3     0.2   18.8     -8.1       0.1     0.1    6.0     -4.1  1.30
            SIMULT SHEAR    5.4      0.2      SIMULT MOM    13.5    16.0
    6.50     0.4     0.3   20.1     -7.8       0.0     0.0    5.2     -4.9  1.30
            SIMULT SHEAR    4.4      0.2      SIMULT MOM    14.9    16.2
    7.80     0.4     0.3   19.4     -7.5      -0.0   -0.0    4.3     -5.8  1.30
            SIMULT SHEAR   -4.9      0.2      SIMULT MOM    15.0    15.0
    9.10     0.2     0.2   17.0     -7.2      -0.1   -0.1    3.3     -6.5  1.30
            SIMULT SHEAR   -5.8      0.2      SIMULT MOM    13.7    12.6
   10.40     0.0     0.0   13.2     -7.1      -0.2   -0.1    2.5     -7.2  1.30
            SIMULT SHEAR   -6.5     -1.5      SIMULT MOM    10.9     9.2
   11.70    -0.3    -0.2    8.6    -10.8      -0.3   -0.2    1.5     -7.8  1.30
            SIMULT SHEAR   -7.3     -3.3      SIMULT MOM     5.4     5.2
   13.00    -0.7    -0.5    3.4    -15.0  S   -0.4   -0.3    1.2L  -20.8  1.30
            SIMULT SHEAR    0.5     -3.3      SIMULT MOM     3.6     0.0

SPAN  3 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT    MOMENT   M  SHEAR   SHEAR   SHEAR    SHEAR  I.F.
    0.00    -0.7    -0.5    3.4    -15.0  S    0.4     0.3   20.8     -1.2L 1.30
            SIMULT SHEAR   -1.0      5.8      SIMULT MOM     0.0     3.7
    1.30    -0.3    -0.2    8.5    -10.7       0.3     0.2    7.8     -1.6  1.30
            SIMULT SHEAR    7.3      3.2      SIMULT MOM     5.2     5.8
    2.60     0.0     0.0   13.1     -7.1       0.2     0.1    7.2     -2.5  1.30
            SIMULT SHEAR    6.5      1.5      SIMULT MOM     9.2    10.9
    3.90     0.2     0.2   17.0     -7.2       0.1     0.1    6.5     -3.4  1.30
            SIMULT SHEAR    5.9     -0.3      SIMULT MOM    12.6    13.7
    5.20     0.3     0.3   19.4     -7.6       0.0     0.0    5.8     -4.3  1.30
            SIMULT SHEAR    4.9     -0.3      SIMULT MOM    15.0    15.0
    6.50     0.3     0.3   20.1     -7.9      -0.0   -0.0    4.9     -5.2  1.30
            SIMULT SHEAR   -4.4     -0.3      SIMULT MOM    16.2    14.8
    7.80     0.3     0.2   18.9     -8.3      -0.1   -0.1    4.1     -6.0  1.30
            SIMULT SHEAR   -5.4     -0.3      SIMULT MOM    16.0    13.3
    9.10     0.1     0.0   15.9     -8.7      -0.2   -0.1    3.1     -6.8  1.30
            SIMULT SHEAR   -6.3     -0.3      SIMULT MOM    14.4    10.8
```

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 10.40 | -0.3 | -0.2 | 11.3 | -9.0 | | -0.3 | -0.2 | 2.3 | -7.4 | 1.30 |
| | SIMULT | SHEAR | -7.2 | -0.3 | | SIMULT | MOM | 11.2 | 7.5 | |
| 11.70 | -0.7 | -0.5 | 6.9 | -12.9 | | -0.3 | -0.3 | 1.5 | -8.1 | 1.30 |
| | SIMULT | SHEAR | -6.9 | -4.3 | | SIMULT | MOM | 6.5 | 5.1 | |
| 13.00 | -1.2 | -0.9 | 2.3 | -18.0 | S | -0.4 | -0.3 | 0.9 | -21.1 | 1.30 |
| | SIMULT | SHEAR | 0.9 | -6.0 | | SIMULT | MOM | 2.3 | -0.7 | |

SPAN 4 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30, NEG MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | -1.2 | -0.9 | 2.3 | -18.0 | S | 0.5 | 0.4 | 20.9 | -0.2 | 1.30 |
| | SIMULT | SHEAR | -0.2 | 4.3 | | SIMULT | MOM | -0.7 | 2.3 | |
| 1.37 | -0.5 | -0.4 | 7.5 | -12.2 | | 0.4 | 0.3 | 8.0 | -0.6 | 1.30 |
| | SIMULT | SHEAR | 7.8 | 4.7 | | SIMULT | MOM | 5.0 | 7.5 | |
| 2.73 | -0.0 | -0.0 | 12.9 | -7.3 | | 0.3 | 0.2 | 7.5 | -1.2 | 1.30 |
| | SIMULT | SHEAR | 7.2 | 0.7 | | SIMULT | MOM | 9.4 | 12.9 | |
| 4.10 | 0.4 | 0.3 | 17.8 | -6.4 | | 0.2 | 0.2 | 7.0 | -1.9 | 1.30 |
| | SIMULT | SHEAR | 6.5 | 0.7 | | SIMULT | MOM | 13.4 | 17.8 | |
| 5.47 | 0.7 | 0.5 | 21.6 | -5.5 | | 0.2 | 0.1 | 6.3 | -2.6 | 1.30 |
| | SIMULT | SHEAR | 5.7 | 0.7 | | SIMULT | MOM | 16.8 | 21.6 | |
| 6.83 | 0.8 | 0.6 | 23.8 | -4.5 | | 0.1 | 0.1 | 5.6 | -3.5 | 1.30 |
| | SIMULT | SHEAR | 4.9 | 0.7 | | SIMULT | MOM | 19.1 | 23.8 | |
| 8.20 | 0.9 | 0.7 | 24.1 | -3.6 | | 0.0 | 0.0 | 4.7 | -4.4 | 1.30 |
| | SIMULT | SHEAR | -4.4 | 0.7 | | SIMULT | MOM | 20.0 | 24.1 | |
| 9.57 | 0.8 | 0.6 | 22.1 | -2.7 | | -0.1 | -0.1 | 3.7 | -5.4 | 1.30 |
| | SIMULT | SHEAR | -5.4 | 0.7 | | SIMULT | MOM | 19.0 | 22.1 | |
| 10.94 | 0.7 | 0.5 | 17.5 | -1.8 | | -0.2 | -0.1 | 2.7 | -6.4 | 1.30 |
| | SIMULT | SHEAR | -6.4 | 0.7 | | SIMULT | MOM | 15.4 | 17.5 | |
| 12.30 | 0.4 | 0.3 | 10.2 | -0.9 | | -0.2 | -0.2 | 1.7 | -7.5 | 1.30 |
| | SIMULT | SHEAR | -7.5 | 0.7 | | SIMULT | MOM | 9.1 | 10.2 | |
| 13.67 | 0.0 | 0.0 | 0.0 | 0.0 | S | -0.3 | -0.2 | 0.7 | -20.9 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

FLEXURAL STRESSES - BEAM

SPAN 1
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.34 | -0.334 | -0.247 | -8.938 | 0.822 | 0.334 | 0.247 | 8.938 | -0.822 |
| 2.68 | -0.572 | -0.423 | -15.372 | 1.643 | 0.572 | 0.423 | 15.372 | -1.643 |
| 4.03 | -0.713 | -0.527 | -19.396 | 2.465 | 0.713 | 0.527 | 19.396 | -2.465 |
| 5.37 | -0.757 | -0.560 | -21.170 | 3.286 | 0.757 | 0.560 | 21.170 | -3.286 |
| 6.71 | -0.704 | -0.521 | -20.934 | 4.108 | 0.704 | 0.521 | 20.934 | -4.108 |
| 8.05 | -0.555 | -0.410 | -18.974 | 4.929 | 0.555 | 0.410 | 18.974 | -4.929 |
| 9.39 | -0.309 | -0.228 | -15.641 | 5.751 | 0.309 | 0.228 | 15.641 | -5.751 |
| 10.74 | 0.034 | 0.025 | -11.375 | 6.572 | -0.034 | -0.025 | 11.375 | -6.572 |
| 12.08 | 0.474 | 0.351 | -6.664 | 10.728 | -0.474 | -0.351 | 6.664 | -10.728 |
| 13.42 | 1.010 | 0.747 | -2.052 | 15.696 | -1.010 | -0.747 | 2.052 | -15.696 |

SPAN 2
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 1.010 | 0.747 | -2.052 | 15.696 | -1.010 | -0.747 | 2.052 | -15.696 |
| 1.30 | 0.562 | 0.416 | -6.166 | 11.309 | -0.562 | -0.416 | 6.166 | -11.309 |
| 2.60 | 0.204 | 0.151 | -10.011 | 7.823 | -0.204 | -0.151 | 10.011 | -7.823 |
| 3.90 | -0.062 | -0.046 | -14.103 | 7.538 | 0.062 | 0.046 | 14.103 | -7.538 |
| 5.20 | -0.238 | -0.176 | -16.800 | 7.253 | 0.238 | 0.176 | 16.800 | -7.253 |
| 6.50 | -0.324 | -0.239 | -17.891 | 6.968 | 0.324 | 0.239 | 17.891 | -6.968 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 7.80 | -0.318 | -0.235 | -17.306 | 6.684 | 0.318 | 0.235 | 17.306 | -6.684 |
| 9.10 | -0.222 | -0.164 | -15.173 | 6.405 | 0.222 | 0.164 | 15.173 | -6.405 |
| 10.40 | -0.035 | -0.026 | -11.767 | 6.311 | 0.035 | 0.026 | 11.767 | -6.311 |
| 11.70 | 0.243 | 0.180 | -7.674 | 9.590 | -0.243 | -0.180 | 7.674 | -9.590 |
| 13.00 | 0.612 | 0.453 | -2.995 | 13.362 | -0.612 | -0.453 | 2.995 | -13.362 |

SPAN 3
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.612 | 0.453 | -2.995 | 13.362 | -0.612 | -0.453 | 2.995 | -13.362 |
| 1.30 | 0.246 | 0.182 | -7.602 | 9.569 | -0.246 | -0.182 | 7.602 | -9.569 |
| 2.60 | -0.030 | -0.022 | -11.713 | 6.298 | 0.030 | 0.022 | 11.713 | -6.298 |
| 3.90 | -0.214 | -0.158 | -15.125 | 6.426 | 0.214 | 0.158 | 15.125 | -6.426 |
| 5.20 | -0.308 | -0.228 | -17.285 | 6.742 | 0.308 | 0.228 | 17.285 | -6.742 |
| 6.50 | -0.311 | -0.230 | -17.887 | 7.064 | 0.311 | 0.230 | 17.887 | -7.064 |
| 7.80 | -0.223 | -0.165 | -16.820 | 7.393 | 0.223 | 0.165 | 16.820 | -7.393 |
| 9.10 | -0.045 | -0.033 | -14.144 | 7.722 | 0.045 | 0.033 | 14.144 | -7.722 |
| 10.40 | 0.224 | 0.166 | -10.061 | 8.050 | -0.224 | -0.166 | 10.061 | -8.050 |
| 11.70 | 0.584 | 0.432 | -6.182 | 11.520 | -0.584 | -0.432 | 6.182 | -11.520 |
| 13.00 | 1.035 | 0.766 | -2.028 | 16.053 | -1.035 | -0.766 | 2.028 | -16.053 |

SPAN 4
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 1.035 | 0.766 | -2.028 | 16.053 | -1.035 | -0.766 | 2.028 | -16.053 |
| 1.37 | 0.480 | 0.355 | -6.694 | 10.862 | -0.480 | -0.355 | 6.694 | -10.862 |
| 2.73 | 0.025 | 0.019 | -11.487 | 6.490 | -0.025 | -0.019 | 11.487 | -6.490 |
| 4.10 | -0.329 | -0.244 | -15.827 | 5.678 | 0.329 | 0.244 | 15.827 | -5.678 |
| 5.47 | -0.583 | -0.432 | -19.233 | 4.867 | 0.583 | 0.432 | 19.233 | -4.867 |
| 6.83 | -0.737 | -0.545 | -21.252 | 4.056 | 0.737 | 0.545 | 21.252 | -4.056 |
| 8.20 | -0.790 | -0.585 | -21.511 | 3.245 | 0.790 | 0.585 | 21.511 | -3.245 |
| 9.57 | -0.743 | -0.550 | -19.716 | 2.434 | 0.743 | 0.550 | 19.716 | -2.434 |
| 10.94 | -0.596 | -0.441 | -15.636 | 1.622 | 0.596 | 0.441 | 15.636 | -1.622 |
| 12.30 | -0.348 | -0.258 | -9.095 | 0.811 | 0.348 | 0.258 | 9.095 | -0.811 |
| 13.67 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.146 | 0.108 | 9.567 | -0.313 | 1.000 | 1.02 V | 1.40 V |
| 1.34 | 0.109 | 0.081 | 3.410 | -0.769 | 1.000 | 1.78 T | 2.45 T |
| 2.68 | 0.072 | 0.053 | 2.932 | -1.248 | 1.000 | 1.01 T | 1.40 T |
| 4.03 | 0.035 | 0.026 | 2.467 | -1.714 | 1.000 | 0.79 T | 1.10 T |
| 5.37 | -0.002 | -0.001 | 2.019 | -2.151 | 1.000 | 0.72 T | 1.00 T |
| 6.71 | -0.039 | -0.029 | 1.597 | -2.541 | 1.000 | 0.73 T | 1.02 T |
| 8.05 | -0.075 | -0.056 | 1.207 | -2.881 | 1.000 | 0.82 T | 1.13 T |
| 9.39 | -0.112 | -0.083 | 0.852 | -3.174 | 1.000 | 1.02 T | 1.40 T |
| 10.74 | -0.149 | -0.110 | 0.542 | -3.422 | 1.000 | 1.46 T | 1.98 T |
| 12.08 | -0.186 | -0.138 | 0.282 | -3.632 | 0.801 | 1.16 B | 1.60 B |
| 13.42 | -0.223 | -0.165 | 0.078 | -9.519 | 0.801 | 0.73 B | 1.04 B |

SPAN 2
=======

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.194 | 0.144 | 9.623 | -0.409 | 0.692 | 0.62 B | 0.88 B |
| 1.30 | 0.159 | 0.117 | 3.676 | -0.688 | 0.670 | 0.89 B | 1.25 B |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 2.60 | 0.123 | 0.091 | 3.358 | -1.030 | 0.566 | 1.15 B | 1.58 B |
| 3.90 | 0.087 | 0.065 | 3.069 | -1.432 | 1.000 | 1.16 T | 1.59 T |
| 5.20 | 0.051 | 0.038 | 2.731 | -1.850 | 1.000 | 0.96 T | 1.31 T |
| 6.50 | 0.016 | 0.012 | 2.355 | -2.251 | 1.000 | 0.89 T | 1.23 T |
| 7.80 | -0.020 | -0.015 | 1.947 | -2.629 | 1.000 | 0.92 T | 1.27 T |
| 9.10 | -0.056 | -0.041 | 1.529 | -2.977 | 1.000 | 1.06 T | 1.46 T |
| 10.40 | -0.092 | -0.068 | 1.136 | -3.289 | 1.000 | 1.40 T | 1.91 T |
| 11.70 | -0.127 | -0.094 | 0.706 | -3.559 | 0.593 | 0.98 B | 1.35 B |
| 13.00 | -0.163 | -0.121 | 0.538 | -9.495 | 0.593 | 0.65 B | 0.92 B |

SPAN 3
=======

| | | SHEAR STRESSES | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.162 | 0.120 | 9.495 | -0.552 | 0.650 | 0.72 B | 1.02 B |
| 1.30 | 0.126 | 0.093 | 3.559 | -0.740 | 0.590 | 0.97 B | 1.34 B |
| 2.60 | 0.091 | 0.067 | 3.291 | -1.149 | 1.000 | 1.40 T | 1.92 T |
| 3.90 | 0.055 | 0.041 | 2.980 | -1.539 | 1.000 | 1.07 T | 1.46 T |
| 5.20 | 0.019 | 0.014 | 2.632 | -1.959 | 1.000 | 0.92 T | 1.27 T |
| 6.50 | -0.017 | -0.012 | 2.255 | -2.370 | 1.000 | 0.89 T | 1.23 T |
| 7.80 | -0.052 | -0.039 | 1.854 | -2.749 | 1.000 | 0.96 T | 1.31 T |
| 9.10 | -0.088 | -0.065 | 1.435 | -3.089 | 1.000 | 1.16 T | 1.59 T |
| 10.40 | -0.124 | -0.092 | 1.033 | -3.380 | 0.577 | 1.13 B | 1.56 B |
| 11.70 | -0.160 | -0.118 | 0.681 | -3.677 | 0.674 | 0.88 B | 1.23 B |
| 13.00 | -0.195 | -0.145 | 0.408 | -9.621 | 0.697 | 0.60 B | 0.86 B |

SPAN 4
=======

| | | SHEAR STRESSES | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.227 | 0.168 | 9.518 | -0.076 | 0.785 | 0.69 B | 0.99 B |
| 1.37 | 0.189 | 0.140 | 3.633 | -0.279 | 0.785 | 1.12 B | 1.55 B |
| 2.73 | 0.152 | 0.112 | 3.425 | -0.538 | 1.000 | 1.44 T | 1.96 T |
| 4.10 | 0.114 | 0.084 | 3.177 | -0.847 | 1.000 | 1.01 T | 1.39 T |
| 5.47 | 0.076 | 0.056 | 2.883 | -1.201 | 1.000 | 0.81 T | 1.12 T |
| 6.83 | 0.039 | 0.029 | 2.541 | -1.592 | 1.000 | 0.72 T | 1.00 T |
| 8.20 | 0.001 | 0.001 | 2.148 | -2.014 | 1.000 | 0.70 T | 0.98 T |
| 9.57 | -0.036 | -0.027 | 1.707 | -2.462 | 1.000 | 0.77 T | 1.08 T |
| 10.94 | -0.074 | -0.055 | 1.242 | -2.928 | 1.000 | 0.99 T | 1.37 T |
| 12.30 | -0.112 | -0.083 | 0.764 | -3.406 | 1.000 | 1.75 T | 2.41 T |
| 13.67 | -0.149 | -0.110 | 0.304 | -9.562 | 1.000 | 1.02 V | 1.40 V |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| | NON-COMP MOMENT | OVERLOAD MOMENT | SHEAR | NON-COMPACT RATING FACTORS | | COMPACT MOMENT | COMPACT RATING FACTORS | |
|---|---|---|---|---|---|---|---|---|
| X | STRENGTH | STRENGTH | STRENGTH | IR | OR | STRENGTH | IR | OR |
| 0.00 | 33.6 B | 26.9 | 38.1 | 0.82 V | 1.37 V | 39.6 | 0.82 V | 1.37 V |
| 1.34 | 33.6 B | 26.9 | 38.1 | 1.51 T | 2.52 T | 39.6 | 1.57 O | 2.62 O |
| 2.68 | 33.6 B | 26.9 | 38.1 | 0.86 T | 1.44 T | 39.6 | 0.90 O | 1.50 O |
| 4.03 | 33.6 B | 26.9 | 38.1 | 0.68 T | 1.13 T | 39.6 | 0.70 O | 1.17 O |
| 5.37 | 33.6 B | 26.9 | 38.1 | 0.62 T | 1.03 T | 39.6 | 0.64 O | 1.07 O |
| 6.71 | 33.6 B | 26.9 | 38.1 | 0.63 T | 1.04 T | 39.6 | 0.65 O | 1.09 O |
| 8.05 | 33.6 B | 26.9 | 38.1 | 0.70 T | 1.17 T | 39.6 | 0.73 O | 1.21 O |
| 9.39 | 33.6 B | 26.9 | 38.1 | 0.86 T | 1.44 T | 39.6 | 0.90 O | 1.50 O |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 10.74 | 33.6 B | 26.9 | 38.1 | 1.22 T | 2.03 T | 39.6 | 1.27 O | 2.12 O |
| 12.08 | -27.0 U | -21.6 | 38.1 | 0.99 B | 1.65 B | | | |
| 13.42 | -27.0 U | -21.6 | 38.1 | 0.64 B | 1.07 B | | | |

SPAN 2
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -23.3 U | -18.6 | 38.1 | 0.54 B | 0.91 B | | | |
| 1.30 | -22.6 U | -18.0 | 38.1 | 0.77 B | 1.28 B | | | |
| 2.60 | -19.0 U | -15.2 | 38.1 | 0.97 B | 1.62 B | | | |
| 3.90 | 33.6 B | 26.9 | 38.1 | 0.98 T | 1.63 T | 39.6 | 1.02 O | 1.69 O |
| 5.20 | 33.6 B | 26.9 | 38.1 | 0.81 T | 1.35 T | 39.6 | 0.84 O | 1.40 O |
| 6.50 | 33.6 B | 26.9 | 38.1 | 0.76 T | 1.26 T | 39.6 | 0.79 O | 1.31 O |
| 7.80 | 33.6 B | 26.9 | 38.1 | 0.78 T | 1.30 T | 39.6 | 0.81 O | 1.35 O |
| 9.10 | 33.6 B | 26.9 | 38.1 | 0.90 T | 1.50 T | 39.6 | 0.93 O | 1.56 O |
| 10.40 | 33.6 B | 26.9 | 38.1 | 1.17 T | 1.96 T | 39.6 | 1.22 O | 2.03 O |
| 11.70 | -19.9 U | -16.0 | 38.1 | 0.83 B | 1.38 B | | | |
| 13.00 | -19.9 U | -16.0 | 38.1 | 0.57 B | 0.94 B | | | |

SPAN 3
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -21.9 U | -17.5 | 38.1 | 0.63 B | 1.04 B | | | |
| 1.30 | -19.8 U | -15.9 | 38.1 | 0.83 B | 1.38 B | | | |
| 2.60 | 33.6 B | 26.9 | 38.1 | 1.18 T | 1.97 T | 39.6 | 1.23 O | 2.04 O |
| 3.90 | 33.6 B | 26.9 | 38.1 | 0.90 T | 1.50 T | 39.6 | 0.94 O | 1.56 O |
| 5.20 | 33.6 B | 26.9 | 38.1 | 0.78 T | 1.30 T | 39.6 | 0.81 O | 1.36 O |
| 6.50 | 33.6 B | 26.9 | 38.1 | 0.76 T | 1.26 T | 39.6 | 0.79 O | 1.31 O |
| 7.80 | 33.6 B | 26.9 | 38.1 | 0.81 T | 1.35 T | 39.6 | 0.84 O | 1.40 O |
| 9.10 | 33.6 B | 26.9 | 38.1 | 0.98 T | 1.63 T | 39.6 | 1.01 O | 1.69 O |
| 10.40 | -19.4 U | -15.5 | 38.1 | 0.96 B | 1.61 B | | | |
| 11.70 | -22.7 U | -18.1 | 38.1 | 0.76 B | 1.26 B | | | |
| 13.00 | -23.5 U | -18.8 | 38.1 | 0.53 B | 0.89 B | | | |

SPAN 4
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -26.4 U | -21.1 | 38.1 | 0.61 B | 1.02 B | | | |
| 1.37 | -26.4 U | -21.1 | 38.1 | 0.95 B | 1.59 B | | | |
| 2.73 | 33.6 B | 26.9 | 38.1 | 1.21 T | 2.01 T | 39.6 | 1.26 O | 2.09 O |
| 4.10 | 33.6 B | 26.9 | 38.1 | 0.85 T | 1.42 T | 39.6 | 0.89 O | 1.48 O |
| 5.47 | 33.6 B | 26.9 | 38.1 | 0.69 T | 1.15 T | 39.6 | 0.72 O | 1.20 O |
| 6.83 | 33.6 B | 26.9 | 38.1 | 0.62 T | 1.03 T | 39.6 | 0.64 O | 1.07 O |
| 8.20 | 33.6 B | 26.9 | 38.1 | 0.61 T | 1.01 T | 39.6 | 0.63 O | 1.05 O |
| 9.57 | 33.6 B | 26.9 | 38.1 | 0.66 T | 1.10 T | 39.6 | 0.69 O | 1.15 O |
| 10.94 | 33.6 B | 26.9 | 38.1 | 0.85 T | 1.41 T | 39.6 | 0.88 O | 1.47 O |
| 12.30 | 33.6 B | 26.9 | 38.1 | 1.48 T | 2.47 T | 39.6 | 1.54 O | 2.57 O |
| 13.67 | 33.6 B | 26.9 | 38.1 | 0.82 V | 1.37 V | 39.6 | 0.82 V | 1.37 V |

```
**********************************
*  STRINGER  -  LIVE LOAD TK527  *
**********************************
```

MAXIMUM REACTIONS

REACTIONS MOMENTS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | +I.F. | -I.F. | +I.F. | -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.3 | 0.2 | 17.3 | -0.7 | 1.30 | 1.30 | | |
| 2 | 0.9 | 0.7 | 13.3 | -1.1 | 1.30 | 1.30 | 1.30 | 1.30 |
| 3 | 0.7 | 0.5 | 20.5 | -1.7 | 1.30 | 1.30 | 1.30 | 1.30 |
| 4 | 0.9 | 0.7 | 13.4 | -1.1 | 1.30 | 1.30 | 1.30 | 1.30 |
| 5 | 0.3 | 0.2 | 17.4 | -0.6 | 1.30 | 1.30 | | |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30, NEG MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | S | 0.3 | 0.2 | 17.3 | -0.7 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 1.34 | 0.4 | 0.3 | 10.3 | -0.9 | | 0.2 | 0.2 | 7.6 | -0.7 | 1.30 |
| | SIMULT | SHEAR | 7.6 | -0.7 | | SIMULT | MOM | 10.3 | 6.3 | |
| 2.68 | 0.6 | 0.5 | 16.6 | -1.8 | | 0.2 | 0.1 | 6.2 | -1.4 | 1.30 |
| | SIMULT | SHEAR | 6.2 | -0.7 | | SIMULT | MOM | 16.6 | 10.8 | |
| 4.03 | 0.8 | 0.6 | 20.0 | -2.6 | | 0.1 | 0.1 | 5.0 | -2.0 | 1.30 |
| | SIMULT | SHEAR | 5.0 | -0.7 | | SIMULT | MOM | 20.0 | 13.6 | |
| 5.37 | 0.8 | 0.6 | 22.1 | -3.5 | | -0.0 | -0.0 | 3.6 | -3.3 | 1.30 |
| | SIMULT | SHEAR | 3.6 | -0.7 | | SIMULT | MOM | 22.1 | 18.3 | |
| 6.71 | 0.8 | 0.6 | 21.0 | -4.4 | | -0.1 | -0.1 | 2.5 | -4.7 | 1.30 |
| | SIMULT | SHEAR | -3.1 | -0.7 | | SIMULT | MOM | 16.6 | 19.4 | |
| 8.05 | 0.6 | 0.5 | 18.3 | -5.3 | | -0.2 | -0.1 | 1.6 | -6.0 | 1.30 |
| | SIMULT | SHEAR | -5.8 | -0.7 | | SIMULT | MOM | 12.6 | 16.6 | |
| 9.39 | 0.3 | 0.3 | 14.7 | -6.2 | | -0.2 | -0.2 | 0.8 | -7.2 | 1.30 |
| | SIMULT | SHEAR | -7.2 | -0.7 | | SIMULT | MOM | 7.7 | 11.5 | |
| 10.74 | -0.0 | -0.0 | 8.3 | -7.1 | | -0.3 | -0.2 | 0.3 | -8.5 | 1.30 |
| | SIMULT | SHEAR | -8.5 | -0.7 | | SIMULT | MOM | 2.8 | 8.3 | |
| 12.08 | -0.5 | -0.4 | 1.9 | -10.6 | | -0.4 | -0.3 | 0.2 | -9.7 | 1.30 |
| | SIMULT | SHEAR | 0.2 | -3.7 | | SIMULT | MOM | 1.9 | -0.0 | |
| 13.42 | -1.1 | -0.8 | 2.1 | -18.7 | S | -0.5 | -0.4 | 0.2 | -19.0 | 1.30 |
| | SIMULT | SHEAR | 0.2 | -8.2 | | SIMULT | MOM | 2.1 | -10.5 | |

SPAN 2 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30, NEG MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | -1.1 | -0.8 | 2.1 | -18.7 | S | 0.4 | 0.3 | 17.8 | -0.8 | 1.30 |
| | SIMULT | SHEAR | -0.8 | 4.5 | | SIMULT | MOM | -11.5 | 2.1 | |
| 1.30 | -0.6 | -0.5 | 2.5 | -12.9 | | 0.3 | 0.3 | 8.8 | -0.8 | 1.30 |
| | SIMULT | SHEAR | 8.4 | 4.4 | | SIMULT | MOM | -3.3 | 1.0 | |
| 2.60 | -0.2 | -0.2 | 9.0 | -9.8 | | 0.3 | 0.2 | 7.7 | -0.8 | 1.30 |
| | SIMULT | SHEAR | 6.9 | 1.3 | | SIMULT | MOM | 4.1 | -0.0 | |
| 3.90 | 0.1 | 0.1 | 13.0 | -8.1 | | 0.2 | 0.1 | 6.5 | -0.9 | 1.30 |
| | SIMULT | SHEAR | 5.5 | 1.3 | | SIMULT | MOM | 9.5 | 10.9 | |
| 5.20 | 0.3 | 0.2 | 14.5 | -6.5 | | 0.1 | 0.1 | 5.2 | -2.3 | 1.30 |
| | SIMULT | SHEAR | 4.2 | 1.3 | | SIMULT | MOM | 13.1 | 14.3 | |
| 6.50 | 0.4 | 0.3 | 15.2 | -4.8 | | 0.0 | 0.0 | 3.8 | -3.7 | 1.30 |
| | SIMULT | SHEAR | -3.7 | 1.3 | | SIMULT | MOM | 14.5 | 15.2 | |
| 7.80 | 0.4 | 0.3 | 13.9 | -4.7 | | -0.0 | -0.0 | 2.3 | -5.1 | 1.30 |
| | SIMULT | SHEAR | -4.2 | -0.9 | | SIMULT | MOM | 13.6 | 13.8 | |
| 9.10 | 0.2 | 0.2 | 12.4 | -5.9 | | -0.1 | -0.1 | 1.3 | -6.4 | 1.30 |
| | SIMULT | SHEAR | -5.5 | -0.9 | | SIMULT | MOM | -1.5 | 10.2 | |
| 10.40 | 0.0 | 0.0 | 8.4 | -7.1 | | -0.2 | -0.1 | 1.3 | -7.7 | 1.30 |

```
            SIMULT SHEAR   -6.9     -0.9      SIMULT MOM     0.2     4.7
   11.70    -0.3    -0.2    3.2    -10.4      -0.3   -0.2    1.3    -8.7  1.30
            SIMULT SHEAR    0.4     -3.6      SIMULT MOM     1.8    -2.1
   13.00    -0.7    -0.5    3.7    -15.4  S   -0.4   -0.3    1.3   -17.7  1.30
            SIMULT SHEAR    0.4     -4.1      SIMULT MOM     3.5    -6.6

SPAN  3 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
    0.00    -0.7    -0.5    3.7    -15.4  S    0.4    0.3   17.7    -1.3  1.30
            SIMULT SHEAR   -1.3      7.3      SIMULT MOM    -6.6     3.7
    1.30    -0.3    -0.2    3.1    -10.3       0.3    0.2    8.7    -1.3  1.30
            SIMULT SHEAR   -0.3      3.6      SIMULT MOM    -2.1     1.9
    2.60     0.0     0.0    8.4     -7.0       0.2    0.1    7.7    -1.3  1.30
            SIMULT SHEAR    6.9      0.9      SIMULT MOM     4.7     0.2
    3.90     0.2     0.2   12.4     -5.9       0.1    0.1    6.5    -1.3  1.30
            SIMULT SHEAR    5.5      0.9      SIMULT MOM    10.2    -1.6
    5.20     0.3     0.3   13.8     -4.7       0.0    0.0    5.1    -2.4  1.30
            SIMULT SHEAR    4.2      0.9      SIMULT MOM    13.8    13.6
    6.50     0.3     0.3   15.3     -5.0      -0.0   -0.0    3.7    -3.8  1.30
            SIMULT SHEAR    3.7     -1.3      SIMULT MOM    15.3    14.4
    7.80     0.3     0.2   14.5     -6.8      -0.1   -0.1    2.3    -5.2  1.30
            SIMULT SHEAR   -4.2     -1.3      SIMULT MOM    14.3    13.0
    9.10     0.1     0.0   13.0     -8.5      -0.2   -0.1    0.9    -6.5  1.30
            SIMULT SHEAR   -5.5     -1.3      SIMULT MOM    10.9     9.4
   10.40    -0.3    -0.2    9.1    -10.2      -0.3   -0.2    0.8    -7.7  1.30
            SIMULT SHEAR   -6.9     -1.3      SIMULT MOM    -0.0     3.9
   11.70    -0.7    -0.5    2.6    -13.3      -0.3   -0.3    0.8    -8.9  1.30
            SIMULT SHEAR   -8.4     -4.4      SIMULT MOM     1.0    -3.5
   13.00    -1.2    -0.9    2.1    -19.2  S   -0.4   -0.3    0.8   -17.9  1.30
            SIMULT SHEAR    0.8     -4.6      SIMULT MOM     2.1   -11.8

SPAN  4 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2   +(LL+I)  -(LL+I)  I   DL1     DL2   +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
    0.00    -1.2    -0.9    2.1    -19.2  S    0.5    0.4   19.1    -0.2  1.30
            SIMULT SHEAR   -0.2      8.2      SIMULT MOM   -10.8     2.1
    1.37    -0.5    -0.4    1.9    -10.6       0.4    0.3    9.8    -0.2  1.30
            SIMULT SHEAR   -0.2      4.0      SIMULT MOM    -0.2     1.9
    2.73    -0.0    -0.0    8.5     -7.0       0.3    0.2    8.6    -0.3  1.30
            SIMULT SHEAR    8.6      0.6      SIMULT MOM     8.5     2.9
    4.10     0.4     0.3   15.0     -6.1       0.2    0.2    7.3    -0.8  1.30
            SIMULT SHEAR    7.3      0.6      SIMULT MOM    15.0     8.0
    5.47     0.7     0.5   18.8     -5.2       0.2    0.1    6.1    -1.6  1.30
            SIMULT SHEAR    5.9      0.6      SIMULT MOM    17.1    13.0
    6.83     0.8     0.6   21.5     -4.4       0.1    0.1    4.7    -2.5  1.30
            SIMULT SHEAR    3.1      0.6      SIMULT MOM    20.1    17.1
    8.20     0.9     0.7   22.7     -3.5       0.0    0.0    3.4    -3.6  1.30
            SIMULT SHEAR   -3.6      0.6      SIMULT MOM    18.8    19.6
    9.57     0.8     0.6   20.6     -2.6      -0.1   -0.1    2.1    -5.0  1.30
            SIMULT SHEAR   -5.0      0.6      SIMULT MOM    14.1    20.6
   10.94     0.7     0.5   17.0     -1.7      -0.2   -0.1    1.4    -6.2  1.30
            SIMULT SHEAR   -6.2      0.6      SIMULT MOM    11.0    17.0
   12.30     0.4     0.3   10.6     -0.9      -0.2   -0.2    0.7    -7.7  1.30
            SIMULT SHEAR   -7.7      0.6      SIMULT MOM     6.4    10.6
   13.67     0.0     0.0    0.0      0.0  S   -0.3   -0.2    0.6   -17.4  1.30
            SIMULT SHEAR    0.0      0.0      SIMULT MOM     0.0     0.0


                           FLEXURAL STRESSES - BEAM
```

SPAN 1
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.34 | -0.334 | -0.247 | -9.150 | 0.787 | 0.334 | 0.247 | 9.150 | -0.787 |
| 2.68 | -0.572 | -0.423 | -14.787 | 1.573 | 0.572 | 0.423 | 14.787 | -1.573 |
| 4.03 | -0.713 | -0.527 | -17.872 | 2.360 | 0.713 | 0.527 | 17.872 | -2.360 |
| 5.37 | -0.757 | -0.560 | -19.672 | 3.147 | 0.757 | 0.560 | 19.672 | -3.147 |
| 6.71 | -0.704 | -0.521 | -18.712 | 3.934 | 0.704 | 0.521 | 18.712 | -3.934 |
| 8.05 | -0.555 | -0.410 | -16.346 | 4.720 | 0.555 | 0.410 | 16.346 | -4.720 |
| 9.39 | -0.309 | -0.228 | -13.090 | 5.507 | 0.309 | 0.228 | 13.090 | -5.507 |
| 10.74 | 0.034 | 0.025 | -7.441 | 6.294 | -0.034 | -0.025 | 7.441 | -6.294 |
| 12.08 | 0.474 | 0.351 | -1.702 | 9.436 | -0.474 | -0.351 | 1.702 | -9.436 |
| 13.42 | 1.010 | 0.747 | -1.891 | 16.657 | -1.010 | -0.747 | 1.891 | -16.657 |

SPAN 2
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 1.010 | 0.747 | -1.891 | 16.657 | -1.010 | -0.747 | 1.891 | -16.657 |
| 1.30 | 0.562 | 0.416 | -2.225 | 11.481 | -0.562 | -0.416 | 2.225 | -11.481 |
| 2.60 | 0.204 | 0.151 | -8.028 | 8.737 | -0.204 | -0.151 | 8.028 | -8.737 |
| 3.90 | -0.062 | -0.046 | -11.575 | 7.256 | 0.062 | 0.046 | 11.575 | -7.256 |
| 5.20 | -0.238 | -0.176 | -12.890 | 5.774 | 0.238 | 0.176 | 12.890 | -5.774 |
| 6.50 | -0.324 | -0.239 | -13.590 | 4.293 | 0.324 | 0.239 | 13.590 | -4.293 |
| 7.80 | -0.318 | -0.235 | -12.398 | 4.211 | 0.318 | 0.235 | 12.398 | -4.211 |
| 9.10 | -0.222 | -0.164 | -11.039 | 5.257 | 0.222 | 0.164 | 11.039 | -5.257 |
| 10.40 | -0.035 | -0.026 | -7.511 | 6.303 | 0.035 | 0.026 | 7.511 | -6.303 |
| 11.70 | 0.243 | 0.180 | -2.853 | 9.251 | -0.243 | -0.180 | 2.853 | -9.251 |
| 13.00 | 0.612 | 0.453 | -3.260 | 13.759 | -0.612 | -0.453 | 3.260 | -13.759 |

SPAN 3
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.612 | 0.453 | -3.260 | 13.759 | -0.612 | -0.453 | 3.260 | -13.759 |
| 1.30 | 0.246 | 0.182 | -2.727 | 9.218 | -0.246 | -0.182 | 2.727 | -9.218 |
| 2.60 | -0.030 | -0.022 | -7.508 | 6.282 | 0.030 | 0.022 | 7.508 | -6.282 |
| 3.90 | -0.214 | -0.158 | -11.016 | 5.242 | 0.214 | 0.158 | 11.016 | -5.242 |
| 5.20 | -0.308 | -0.228 | -12.343 | 4.202 | 0.308 | 0.228 | 12.343 | -4.202 |
| 6.50 | -0.311 | -0.230 | -13.609 | 4.488 | 0.311 | 0.230 | 13.609 | -4.488 |
| 7.80 | -0.223 | -0.165 | -12.916 | 6.038 | 0.223 | 0.165 | 12.916 | -6.038 |
| 9.10 | -0.045 | -0.033 | -11.619 | 7.587 | 0.045 | 0.033 | 11.619 | -7.587 |
| 10.40 | 0.224 | 0.166 | -8.088 | 9.137 | -0.224 | -0.166 | 8.088 | -9.137 |
| 11.70 | 0.584 | 0.432 | -2.294 | 11.825 | -0.584 | -0.432 | 2.294 | -11.825 |
| 13.00 | 1.035 | 0.766 | -1.879 | 17.076 | -1.035 | -0.766 | 1.879 | -17.076 |

SPAN 4
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 1.035 | 0.766 | -1.879 | 17.076 | -1.035 | -0.766 | 1.879 | -17.076 |
| 1.37 | 0.480 | 0.355 | -1.691 | 9.408 | -0.480 | -0.355 | 1.691 | -9.408 |
| 2.73 | 0.025 | 0.019 | -7.561 | 6.225 | -0.025 | -0.019 | 7.561 | -6.225 |
| 4.10 | -0.329 | -0.244 | -13.372 | 5.447 | 0.329 | 0.244 | 13.372 | -5.447 |
| 5.47 | -0.583 | -0.432 | -16.764 | 4.669 | 0.583 | 0.432 | 16.764 | -4.669 |
| 6.83 | -0.737 | -0.545 | -19.206 | 3.891 | 0.737 | 0.545 | 19.206 | -3.891 |
| 8.20 | -0.790 | -0.585 | -20.198 | 3.113 | 0.790 | 0.585 | 20.198 | -3.113 |
| 9.57 | -0.743 | -0.550 | -18.374 | 2.334 | 0.743 | 0.550 | 18.374 | -2.334 |
| 10.94 | -0.596 | -0.441 | -15.125 | 1.556 | 0.596 | 0.441 | 15.125 | -1.556 |
| 12.30 | -0.348 | -0.258 | -9.413 | 0.778 | 0.348 | 0.258 | 9.413 | -0.778 |
| 13.67 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

| X | DL1 | DL2 | +(LL+I) | -(LL+I) | ALLOW COMPR REDUCTION | RATING FACTORS IR | RATING FACTORS OR |
|---|---|---|---|---|---|---|---|
| | SHEAR STRESSES | | | | | | |
| 0.00 | 0.146 | 0.108 | 7.896 | -0.300 | 1.000 | 1.23 V | 1.69 V |
| 1.34 | 0.109 | 0.081 | 3.491 | -0.312 | 1.000 | 1.74 T | 2.40 T |
| 2.68 | 0.072 | 0.053 | 2.821 | -0.620 | 1.000 | 1.05 T | 1.45 T |
| 4.03 | 0.035 | 0.026 | 2.267 | -0.925 | 1.000 | 0.85 T | 1.19 T |
| 5.37 | -0.002 | -0.001 | 1.626 | -1.525 | 1.000 | 0.77 T | 1.08 T |
| 6.71 | -0.039 | -0.029 | 1.129 | -2.126 | 1.000 | 0.82 T | 1.14 T |
| 8.05 | -0.075 | -0.056 | 0.714 | -2.751 | 1.000 | 0.95 T | 1.32 T |
| 9.39 | -0.112 | -0.083 | 0.373 | -3.308 | 1.000 | 1.22 T | 1.68 T |
| 10.74 | -0.149 | -0.110 | 0.120 | -3.894 | 0.801 | 2.09 B | 2.86 B |
| 12.08 | -0.186 | -0.138 | 0.072 | -4.440 | 0.801 | 1.31 B | 1.82 B |
| 13.42 | -0.223 | -0.165 | 0.072 | -8.693 | 0.801 | 0.69 B | 0.98 B |

SPAN 2
=======

| X | DL1 | DL2 | +(LL+I) | -(LL+I) | ALLOW COMPR REDUCTION | RATING FACTORS IR | RATING FACTORS OR |
|---|---|---|---|---|---|---|---|
| | SHEAR STRESSES | | | | | | |
| 0.00 | 0.194 | 0.144 | 8.143 | -0.377 | 0.772 | 0.66 B | 0.94 B |
| 1.30 | 0.159 | 0.117 | 4.038 | -0.377 | 0.773 | 1.03 B | 1.43 B |
| 2.60 | 0.123 | 0.091 | 3.512 | -0.377 | 0.933 | 1.72 B | 2.36 B |
| 3.90 | 0.087 | 0.065 | 2.956 | -0.401 | 1.000 | 1.42 T | 1.93 T |
| 5.20 | 0.051 | 0.038 | 2.353 | -1.048 | 1.000 | 1.25 T | 1.71 T |
| 6.50 | 0.016 | 0.012 | 1.718 | -1.699 | 1.000 | 1.17 T | 1.61 T |
| 7.80 | -0.020 | -0.015 | 1.072 | -2.337 | 1.000 | 1.29 T | 1.77 T |
| 9.10 | -0.056 | -0.041 | 0.584 | -2.944 | 1.000 | 1.46 T | 2.00 T |
| 10.40 | -0.092 | -0.068 | 0.584 | -3.499 | 1.000 | 2.19 T | 2.99 T |
| 11.70 | -0.127 | -0.094 | 0.584 | -3.992 | 0.729 | 1.25 B | 1.73 B |
| 13.00 | -0.163 | -0.121 | 0.584 | -8.091 | 0.717 | 0.78 B | 1.09 B |

SPAN 3
=======

| X | DL1 | DL2 | +(LL+I) | -(LL+I) | ALLOW COMPR REDUCTION | RATING FACTORS IR | RATING FACTORS OR |
|---|---|---|---|---|---|---|---|
| | SHEAR STRESSES | | | | | | |
| 0.00 | 0.162 | 0.120 | 8.087 | -0.610 | 0.581 | 0.62 B | 0.87 B |
| 1.30 | 0.126 | 0.093 | 3.994 | -0.610 | 0.724 | 1.25 B | 1.72 B |
| 2.60 | 0.091 | 0.067 | 3.502 | -0.610 | 1.000 | 2.19 T | 2.99 T |
| 3.90 | 0.055 | 0.041 | 2.948 | -0.610 | 1.000 | 1.46 T | 2.01 T |
| 5.20 | 0.019 | 0.014 | 2.342 | -1.077 | 1.000 | 1.29 T | 1.78 T |
| 6.50 | -0.017 | -0.012 | 1.705 | -1.725 | 1.000 | 1.17 T | 1.61 T |
| 7.80 | -0.052 | -0.039 | 1.053 | -2.361 | 1.000 | 1.25 T | 1.71 T |
| 9.10 | -0.088 | -0.065 | 0.405 | -2.966 | 1.000 | 1.41 T | 1.93 T |
| 10.40 | -0.124 | -0.092 | 0.378 | -3.524 | 0.935 | 1.65 B | 2.26 B |
| 11.70 | -0.160 | -0.118 | 0.378 | -4.046 | 0.777 | 1.00 B | 1.39 B |
| 13.00 | -0.195 | -0.145 | 0.378 | -8.154 | 0.775 | 0.64 B | 0.92 B |

SPAN 4
=======

| X | DL1 | DL2 | +(LL+I) | -(LL+I) | ALLOW COMPR REDUCTION | RATING FACTORS IR | RATING FACTORS OR |
|---|---|---|---|---|---|---|---|
| | SHEAR STRESSES | | | | | | |
| 0.00 | 0.227 | 0.168 | 8.739 | -0.070 | 0.785 | 0.65 B | 0.93 B |
| 1.37 | 0.189 | 0.140 | 4.489 | -0.070 | 0.785 | 1.29 B | 1.79 B |
| 2.73 | 0.152 | 0.112 | 3.925 | -0.119 | 0.785 | 2.07 B | 2.83 B |
| 4.10 | 0.114 | 0.084 | 3.333 | -0.381 | 1.000 | 1.19 T | 1.64 T |
| 5.47 | 0.076 | 0.056 | 2.764 | -0.723 | 1.000 | 0.92 T | 1.28 T |
| 6.83 | 0.039 | 0.029 | 2.136 | -1.141 | 1.000 | 0.79 T | 1.10 T |
| 8.20 | 0.001 | 0.001 | 1.544 | -1.632 | 1.000 | 0.75 T | 1.05 T |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 9.57 | -0.036 | -0.027 | 0.944 | -2.270 | 1.000 | 0.83 T | 1.15 T |
| 10.94 | -0.074 | -0.055 | 0.621 | -2.832 | 1.000 | 1.02 T | 1.42 T |
| 12.30 | -0.112 | -0.083 | 0.312 | -3.526 | 1.000 | 1.69 T | 2.33 T |
| 13.67 | -0.149 | -0.110 | 0.291 | -7.938 | 1.000 | 1.23 V | 1.68 V |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 33.6 B | 26.9 | 38.1 | 1.00 V | 1.66 V | 39.6 | 1.00 V | 1.66 V |
| 1.34 | 33.6 B | 26.9 | 38.1 | 1.48 T | 2.46 T | 39.6 | 1.54 O | 2.56 O |
| 2.68 | 33.6 B | 26.9 | 38.1 | 0.90 T | 1.49 T | 39.6 | 0.93 O | 1.56 O |
| 4.03 | 33.6 B | 26.9 | 38.1 | 0.73 T | 1.22 T | 39.6 | 0.76 O | 1.27 O |
| 5.37 | 33.6 B | 26.9 | 38.1 | 0.66 T | 1.11 T | 39.6 | 0.69 O | 1.15 O |
| 6.71 | 33.6 B | 26.9 | 38.1 | 0.70 T | 1.17 T | 39.6 | 0.73 O | 1.22 O |
| 8.05 | 33.6 B | 26.9 | 38.1 | 0.81 T | 1.35 T | 39.6 | 0.85 O | 1.41 O |
| 9.39 | 33.6 B | 26.9 | 38.1 | 1.03 T | 1.72 T | 39.6 | 1.08 O | 1.79 O |
| 10.74 | -27.0 U | -21.6 | 38.1 | 1.76 B | 2.93 B | | | |
| 12.08 | -27.0 U | -21.6 | 38.1 | 1.12 B | 1.87 B | | | |
| 13.42 | -27.0 U | -21.6 | 38.1 | 0.60 B | 1.00 B | | | |

SPAN 2
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -26.0 U | -20.8 | 38.1 | 0.58 B | 0.96 B | | | |
| 1.30 | -26.0 U | -20.8 | 38.1 | 0.88 B | 1.47 B | | | |
| 2.60 | -31.4 U | -25.1 | 38.1 | 1.45 B | 2.42 B | | | |
| 3.90 | 33.6 B | 26.9 | 38.1 | 1.19 T | 1.98 T | 39.6 | 1.24 O | 2.06 O |
| 5.20 | 33.6 B | 26.9 | 38.1 | 1.05 T | 1.76 T | 39.6 | 1.10 O | 1.83 O |
| 6.50 | 33.6 B | 26.9 | 38.1 | 0.99 T | 1.66 T | 39.6 | 1.03 O | 1.72 O |
| 7.80 | 33.6 B | 26.9 | 38.1 | 1.09 T | 1.82 T | 39.6 | 1.13 O | 1.89 O |
| 9.10 | 33.6 B | 26.9 | 38.1 | 1.23 T | 2.06 T | 39.6 | 1.28 O | 2.14 O |
| 10.40 | 33.6 B | 26.9 | 38.1 | 1.84 T | 3.06 T | 39.6 | 1.91 O | 3.19 O |
| 11.70 | -24.5 U | -19.6 | 38.1 | 1.06 B | 1.77 B | | | |
| 13.00 | -24.1 U | -19.3 | 38.1 | 0.67 B | 1.12 B | | | |

SPAN 3
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -19.5 U | -15.6 | 38.1 | 0.54 B | 0.90 B | | | |
| 1.30 | -24.3 U | -19.5 | 38.1 | 1.06 B | 1.77 B | | | |
| 2.60 | 33.6 B | 26.9 | 38.1 | 1.84 T | 3.07 T | 39.6 | 1.91 O | 3.19 O |
| 3.90 | 33.6 B | 26.9 | 38.1 | 1.24 T | 2.06 T | 39.6 | 1.29 O | 2.14 O |
| 5.20 | 33.6 B | 26.9 | 38.1 | 1.10 T | 1.83 T | 39.6 | 1.14 O | 1.90 O |
| 6.50 | 33.6 B | 26.9 | 38.1 | 0.99 T | 1.66 T | 39.6 | 1.03 O | 1.72 O |
| 7.80 | 33.6 B | 26.9 | 38.1 | 1.05 T | 1.76 T | 39.6 | 1.10 O | 1.83 O |
| 9.10 | 33.6 B | 26.9 | 38.1 | 1.19 T | 1.98 T | 39.6 | 1.24 O | 2.06 O |
| 10.40 | -31.5 U | -25.2 | 38.1 | 1.39 B | 2.32 B | | | |
| 11.70 | -26.2 U | -20.9 | 38.1 | 0.86 B | 1.43 B | | | |
| 13.00 | -26.1 U | -20.9 | 38.1 | 0.57 B | 0.94 B | | | |

SPAN 4
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -26.4 U | -21.1 | 38.1 | 0.57 B | 0.95 B | | | |
| 1.37 | -26.4 U | -21.1 | 38.1 | 1.10 B | 1.84 B | | | |
| 2.73 | -26.4 U | -21.1 | 38.1 | 1.74 B | 2.90 B | | | |
| 4.10 | 33.6 B | 26.9 | 38.1 | 1.01 T | 1.68 T | 39.6 | 1.05 O | 1.75 O |
| 5.47 | 33.6 B | 26.9 | 38.1 | 0.79 T | 1.32 T | 39.6 | 0.82 O | 1.37 O |
| 6.83 | 33.6 B | 26.9 | 38.1 | 0.68 T | 1.13 T | 39.6 | 0.71 O | 1.18 O |
| 8.20 | 33.6 B | 26.9 | 38.1 | 0.64 T | 1.07 T | 39.6 | 0.67 O | 1.12 O |
| 9.57 | 33.6 B | 26.9 | 38.1 | 0.71 T | 1.19 T | 39.6 | 0.74 O | 1.24 O |
| 10.94 | 33.6 B | 26.9 | 38.1 | 0.87 T | 1.46 T | 39.6 | 0.91 O | 1.52 O |
| 12.30 | 33.6 B | 26.9 | 38.1 | 1.43 T | 2.39 T | 39.6 | 1.49 O | 2.49 O |
| 13.67 | 33.6 B | 26.9 | 38.1 | 0.99 V | 1.65 V | 39.6 | 0.99 V | 1.65 V |

**********************************
* STRINGER - LIVE LOAD ML80 *
**********************************

MAXIMUM REACTIONS

| SUPPORT | DL1 | DL2 | +(LL+I) | -(LL+I) | REACTIONS +I.F. | REACTIONS -I.F. | MOMENTS +I.F. | MOMENTS -I.F. |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.3 | 0.2 | 18.4 | -0.8 | 1.30 | 1.30 | | |
| 2 | 0.9 | 0.7 | 14.8 | -1.3 | 1.30 | 1.30 | 1.30 | 1.30 |
| 3 | 0.7 | 0.5 | 22.1 | -2.1 | 1.30 | 1.30 | 1.30 | 1.30 |
| 4 | 0.9 | 0.7 | 14.9 | -1.2 | 1.30 | 1.30 | 1.30 | 1.30 |
| 5 | 0.3 | 0.2 | 18.5 | -0.7 | 1.30 | 1.30 | | |

NOTE: ALL SUPPORT REACTIONS AND END SHEARS IN EACH SPAN DUE TO A LIVE LOAD ARE CALCULATED BASED ON AASHTO ARTICLE 3.23.1 AS INTERPRETED PER SOL 431-93-05 IN PENNDOT DM4 REV 12-94, PAGE C B.3-8.

UNFACTORED MOMENTS AND SHEARS

SPAN 1 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30, NEG MOM 1.30
=======

| X | DL1 MOMENT | DL2 MOMENT | +(LL+I) MOMENT | -(LL+I) MOMENT | I M | DL1 SHEAR | DL2 SHEAR | +(LL+I) SHEAR | -(LL+I) SHEAR | I.F. |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 0.0 | S | 0.3 | 0.2 | 18.4 | -0.8 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |
| 1.34 | 0.4 | 0.3 | 11.5 | -1.0 | | 0.2 | 0.2 | 8.6 | -0.8 | 1.30 |
| | SIMULT | SHEAR | 8.6 | -0.8 | | SIMULT | MOM | 11.5 | -1.0 | |
| 2.68 | 0.6 | 0.5 | 18.4 | -2.0 | | 0.2 | 0.1 | 6.9 | -1.4 | 1.30 |
| | SIMULT | SHEAR | 6.9 | -0.8 | | SIMULT | MOM | 18.4 | 5.7 | |
| 4.03 | 0.8 | 0.6 | 21.2 | -3.0 | | 0.1 | 0.1 | 5.3 | -2.1 | 1.30 |
| | SIMULT | SHEAR | 5.3 | -0.8 | | SIMULT | MOM | 21.2 | 5.9 | |
| 5.37 | 0.8 | 0.6 | 24.2 | -4.1 | | -0.0 | -0.0 | 3.9 | -3.5 | 1.30 |
| | SIMULT | SHEAR | 3.1 | -0.8 | | SIMULT | MOM | 20.7 | 17.4 | |
| 6.71 | 0.8 | 0.6 | 24.2 | -5.1 | | -0.1 | -0.1 | 2.6 | -4.8 | 1.30 |
| | SIMULT | SHEAR | -3.9 | -0.8 | | SIMULT | MOM | 17.5 | 18.2 | |
| 8.05 | 0.6 | 0.5 | 20.7 | -6.1 | | -0.2 | -0.1 | 1.6 | -6.1 | 1.30 |
| | SIMULT | SHEAR | -5.5 | -0.8 | | SIMULT | MOM | 12.6 | 16.2 | |
| 9.39 | 0.3 | 0.3 | 15.2 | -7.1 | | -0.2 | -0.2 | 0.7 | -7.9 | 1.30 |
| | SIMULT | SHEAR | -7.7 | -0.8 | | SIMULT | MOM | 6.9 | 12.8 | |
| 10.74 | -0.0 | -0.0 | 8.3 | -8.7 | | -0.3 | -0.2 | 0.2 | -9.6 | 1.30 |
| | SIMULT | SHEAR | -9.4 | -2.3 | | SIMULT | MOM | 2.2 | 5.8 | |

```
   12.08    -0.5    -0.4    2.4    -12.2      -0.4   -0.3    0.2   -11.1  1.30
            SIMULT SHEAR    0.2     -2.6      SIMULT MOM     2.4    -3.8
   13.42    -1.1    -0.8    2.7    -19.2  S   -0.5   -0.4    0.2   -20.5  1.30
            SIMULT SHEAR    0.2     -8.1      SIMULT MOM     2.7   -15.3

SPAN  2 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2  +(LL+I)  -(LL+I)  I   DL1     DL2  +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
    0.00    -1.1    -0.8    2.7    -19.2  S    0.4    0.3   19.5    -1.1  1.30
            SIMULT SHEAR   -1.1      3.4      SIMULT MOM   -12.5     2.7
    1.30    -0.6    -0.5    3.1    -15.7       0.3    0.3    9.9    -1.1  1.30
            SIMULT SHEAR    2.9      3.0      SIMULT MOM    -2.3     1.3
    2.60    -0.2    -0.2    9.1    -12.2       0.3    0.2    8.3    -1.5  1.30
            SIMULT SHEAR    7.5      1.6      SIMULT MOM     5.6     4.8
    3.90     0.1     0.1   13.5    -10.1       0.2    0.1    6.7    -2.1  1.30
            SIMULT SHEAR    5.9      1.6      SIMULT MOM    10.8     5.1
    5.20     0.3     0.2   16.6     -8.1       0.1    0.1    5.1    -2.6  1.30
            SIMULT SHEAR    3.9      1.6      SIMULT MOM    13.1     4.5
    6.50     0.4     0.3   17.7     -6.0       0.0    0.0    3.6    -3.6  1.30
            SIMULT SHEAR   -3.1      1.6      SIMULT MOM    12.7    12.9
    7.80     0.4     0.3   16.1     -5.5      -0.0   -0.0    3.0    -5.1  1.30
            SIMULT SHEAR   -4.0     -1.1      SIMULT MOM     3.6    13.3
    9.10     0.2     0.2   13.0     -6.9      -0.1   -0.1    2.4    -6.7  1.30
            SIMULT SHEAR   -6.0     -1.1      SIMULT MOM     4.7    11.0
   10.40     0.0     0.0    8.4     -9.1      -0.2   -0.1    1.8    -8.4  1.30
            SIMULT SHEAR   -7.6     -2.3      SIMULT MOM     4.8     5.8
   11.70    -0.3    -0.2    4.0    -12.0      -0.3   -0.2    1.6   -10.0  1.30
            SIMULT SHEAR    0.4     -2.3      SIMULT MOM     2.3    -2.1
   13.00    -0.7    -0.5    4.5    -15.4  S   -0.4   -0.3    1.6   -19.5  1.30
            SIMULT SHEAR    0.4     -3.0      SIMULT MOM     4.3   -12.3

SPAN  3 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2  +(LL+I)  -(LL+I)  I   DL1     DL2  +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
    0.00    -0.7    -0.5    4.5    -15.4  S    0.4    0.3   19.5    -1.7  1.30
            SIMULT SHEAR   -1.7      6.9      SIMULT MOM   -12.3     4.5
    1.30    -0.3    -0.2    3.8    -12.0       0.3    0.2   10.0    -1.7  1.30
            SIMULT SHEAR   -0.4      2.3      SIMULT MOM    -2.1     2.4
    2.60     0.0     0.0    8.4     -9.0       0.2    0.1    8.4    -1.8  1.30
            SIMULT SHEAR    7.6      2.3      SIMULT MOM     5.8     4.8
    3.90     0.2     0.2   12.9     -6.9       0.1    0.1    6.8    -2.5  1.30
            SIMULT SHEAR    6.0      1.1      SIMULT MOM    11.0     4.6
    5.20     0.3     0.3   16.1     -5.5       0.0    0.0    5.2    -3.0  1.30
            SIMULT SHEAR    4.0      1.1      SIMULT MOM    13.3     3.5
    6.50     0.3     0.3   17.8     -6.2      -0.0   -0.0    3.6    -3.6  1.30
            SIMULT SHEAR    3.1     -1.7      SIMULT MOM    12.9    12.7
    7.80     0.3     0.2   16.7     -8.4      -0.1   -0.1    2.6    -5.1  1.30
            SIMULT SHEAR   -3.9     -1.7      SIMULT MOM     4.5    13.1
    9.10     0.1     0.0   13.6    -10.5      -0.2   -0.1    2.1    -6.7  1.30
            SIMULT SHEAR   -5.9     -1.7      SIMULT MOM     5.1    10.8
   10.40    -0.3    -0.2    9.2    -12.7      -0.3   -0.2    1.5    -8.3  1.30
            SIMULT SHEAR   -7.5     -1.7      SIMULT MOM     4.8     5.6
   11.70    -0.7    -0.5    3.1    -16.2      -0.3   -0.3    1.1    -9.9  1.30
            SIMULT SHEAR   -2.9     -3.0      SIMULT MOM     1.3    -2.3
   13.00    -1.2    -0.9    2.7    -19.8  S   -0.4   -0.3    1.1   -19.5  1.30
            SIMULT SHEAR    1.1     -3.5      SIMULT MOM     2.7   -12.6

SPAN  4 - LIVE LOAD IMPACT FACTORS : POS MOM 1.30,  NEG MOM 1.30
=======
            DL1     DL2  +(LL+I)  -(LL+I)  I   DL1     DL2  +(LL+I) -(LL+I)
     X    MOMENT  MOMENT MOMENT   MOMENT   M  SHEAR  SHEAR  SHEAR    SHEAR  I.F.
```

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | -1.2 | -0.9 | 2.7 | -19.8 | S | 0.5 | 0.4 | 20.6 | -0.2 | 1.30 |
| | SIMULT | SHEAR | -0.2 | 8.1 | | SIMULT | MOM | -15.6 | 2.7 | |
| 1.37 | -0.5 | -0.4 | 2.4 | -12.1 | | 0.4 | 0.3 | 11.2 | -0.2 | 1.30 |
| | SIMULT | SHEAR | -0.2 | 2.6 | | SIMULT | MOM | -3.9 | 2.4 | |
| 2.73 | -0.0 | -0.0 | 8.4 | -8.6 | | 0.3 | 0.2 | 9.7 | -0.2 | 1.30 |
| | SIMULT | SHEAR | 9.5 | 2.4 | | SIMULT | MOM | 6.0 | 2.1 | |
| 4.10 | 0.4 | 0.3 | 15.6 | -7.0 | | 0.2 | 0.2 | 8.0 | -0.8 | 1.30 |
| | SIMULT | SHEAR | 7.8 | 0.7 | | SIMULT | MOM | 13.2 | 7.2 | |
| 5.47 | 0.7 | 0.5 | 21.3 | -6.0 | | 0.2 | 0.1 | 6.2 | -1.6 | 1.30 |
| | SIMULT | SHEAR | 5.5 | 0.7 | | SIMULT | MOM | 16.9 | 13.0 | |
| 6.83 | 0.8 | 0.6 | 24.9 | -5.0 | | 0.1 | 0.1 | 4.9 | -2.6 | 1.30 |
| | SIMULT | SHEAR | 4.0 | 0.7 | | SIMULT | MOM | 18.7 | 18.1 | |
| 8.20 | 0.9 | 0.7 | 25.0 | -4.0 | | 0.0 | 0.0 | 3.5 | -3.9 | 1.30 |
| | SIMULT | SHEAR | -3.1 | 0.7 | | SIMULT | MOM | 18.0 | 21.3 | |
| 9.57 | 0.8 | 0.6 | 21.9 | -3.0 | | -0.1 | -0.1 | 2.1 | -5.3 | 1.30 |
| | SIMULT | SHEAR | -5.3 | 0.7 | | SIMULT | MOM | 14.0 | 21.9 | |
| 10.94 | 0.7 | 0.5 | 18.9 | -2.0 | | -0.2 | -0.1 | 1.4 | -6.9 | 1.30 |
| | SIMULT | SHEAR | -6.9 | 0.7 | | SIMULT | MOM | 6.0 | 18.9 | |
| 12.30 | 0.4 | 0.3 | 11.8 | -1.0 | | -0.2 | -0.2 | 0.7 | -8.6 | 1.30 |
| | SIMULT | SHEAR | -8.6 | 0.7 | | SIMULT | MOM | -1.0 | 11.8 | |
| 13.67 | 0.0 | 0.0 | 0.0 | 0.0 | S | -0.3 | -0.2 | 0.7 | -18.5 | 1.30 |
| | SIMULT | SHEAR | 0.0 | 0.0 | | SIMULT | MOM | 0.0 | 0.0 | |

FLEXURAL STRESSES - BEAM

SPAN 1
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.34 | -0.334 | -0.247 | -10.244 | 0.904 | 0.334 | 0.247 | 10.244 | -0.904 |
| 2.68 | -0.572 | -0.423 | -16.396 | 1.808 | 0.572 | 0.423 | 16.396 | -1.808 |
| 4.03 | -0.713 | -0.527 | -18.926 | 2.712 | 0.713 | 0.527 | 18.926 | -2.712 |
| 5.37 | -0.757 | -0.560 | -21.612 | 3.615 | 0.757 | 0.560 | 21.612 | -3.615 |
| 6.71 | -0.704 | -0.521 | -21.598 | 4.519 | 0.704 | 0.521 | 21.598 | -4.519 |
| 8.05 | -0.555 | -0.410 | -18.441 | 5.423 | 0.555 | 0.410 | 18.441 | -5.423 |
| 9.39 | -0.309 | -0.228 | -13.540 | 6.327 | 0.309 | 0.228 | 13.540 | -6.327 |
| 10.74 | 0.034 | 0.025 | -7.377 | 7.753 | -0.034 | -0.025 | 7.377 | -7.753 |
| 12.08 | 0.474 | 0.351 | -2.168 | 10.838 | -0.474 | -0.351 | 2.168 | -10.838 |
| 13.42 | 1.010 | 0.747 | -2.409 | 17.105 | -1.010 | -0.747 | 2.409 | -17.105 |

SPAN 2
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 1.010 | 0.747 | -2.409 | 17.105 | -1.010 | -0.747 | 2.409 | -17.105 |
| 1.30 | 0.562 | 0.416 | -2.765 | 13.965 | -0.562 | -0.416 | 2.765 | -13.965 |
| 2.60 | 0.204 | 0.151 | -8.097 | 10.868 | -0.204 | -0.151 | 8.097 | -10.868 |
| 3.90 | -0.062 | -0.046 | -12.055 | 9.025 | 0.062 | 0.046 | 12.055 | -9.025 |
| 5.20 | -0.238 | -0.176 | -14.816 | 7.182 | 0.238 | 0.176 | 14.816 | -7.182 |
| 6.50 | -0.324 | -0.239 | -15.798 | 5.339 | 0.324 | 0.239 | 15.798 | -5.339 |
| 7.80 | -0.318 | -0.235 | -14.383 | 4.911 | 0.318 | 0.235 | 14.383 | -4.911 |
| 9.10 | -0.222 | -0.164 | -11.553 | 6.131 | 0.222 | 0.164 | 11.553 | -6.131 |
| 10.40 | -0.035 | -0.026 | -7.481 | 8.076 | 0.035 | 0.026 | 7.481 | -8.076 |
| 11.70 | 0.243 | 0.180 | -3.527 | 10.706 | -0.243 | -0.180 | 3.527 | -10.706 |
| 13.00 | 0.612 | 0.453 | -4.029 | 13.763 | -0.612 | -0.453 | 4.029 | -13.763 |

SPAN 3
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.612 | 0.453 | -4.029 | 13.763 | -0.612 | -0.453 | 4.029 | -13.763 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 1.30 | 0.246 | 0.182 | -3.393 | 10.686 | -0.246 | -0.182 | 3.393 | -10.686 |
| 2.60 | -0.030 | -0.022 | -7.477 | 8.058 | 0.030 | 0.022 | 7.477 | -8.058 |
| 3.90 | -0.214 | -0.158 | -11.539 | 6.139 | 0.214 | 0.158 | 11.539 | -6.139 |
| 5.20 | -0.308 | -0.228 | -14.371 | 4.921 | 0.308 | 0.228 | 14.371 | -4.921 |
| 6.50 | -0.311 | -0.230 | -15.833 | 5.548 | 0.311 | 0.230 | 15.833 | -5.548 |
| 7.80 | -0.223 | -0.165 | -14.861 | 7.464 | 0.223 | 0.165 | 14.861 | -7.464 |
| 9.10 | -0.045 | -0.033 | -12.109 | 9.380 | 0.045 | 0.033 | 12.109 | -9.380 |
| 10.40 | 0.224 | 0.166 | -8.167 | 11.295 | -0.224 | -0.166 | 8.167 | -11.295 |
| 11.70 | 0.584 | 0.432 | -2.768 | 14.429 | -0.584 | -0.432 | 2.768 | -14.429 |
| 13.00 | 1.035 | 0.766 | -2.387 | 17.670 | -1.035 | -0.766 | 2.387 | -17.670 |

SPAN 4
=======

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 1.035 | 0.766 | -2.387 | 17.670 | -1.035 | -0.766 | 2.387 | -17.670 |
| 1.37 | 0.480 | 0.355 | -2.148 | 10.792 | -0.480 | -0.355 | 2.148 | -10.792 |
| 2.73 | 0.025 | 0.019 | -7.491 | 7.661 | -0.025 | -0.019 | 7.491 | -7.661 |
| 4.10 | -0.329 | -0.244 | -13.876 | 6.264 | 0.329 | 0.244 | 13.876 | -6.264 |
| 5.47 | -0.583 | -0.432 | -18.961 | 5.369 | 0.583 | 0.432 | 18.961 | -5.369 |
| 6.83 | -0.737 | -0.545 | -22.219 | 4.474 | 0.737 | 0.545 | 22.219 | -4.474 |
| 8.20 | -0.790 | -0.585 | -22.277 | 3.579 | 0.790 | 0.585 | 22.277 | -3.579 |
| 9.57 | -0.743 | -0.550 | -19.497 | 2.684 | 0.743 | 0.550 | 19.497 | -2.684 |
| 10.94 | -0.596 | -0.441 | -16.870 | 1.790 | 0.596 | 0.441 | 16.870 | -1.790 |
| 12.30 | -0.348 | -0.258 | -10.531 | 0.895 | 0.348 | 0.258 | 10.531 | -0.895 |
| 13.67 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

SPAN 1
=======

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.146 | 0.108 | 8.392 | -0.345 | 1.000 | 1.16 V | 1.59 V |
| 1.34 | 0.109 | 0.081 | 3.908 | -0.345 | 1.000 | 1.55 T | 2.14 T |
| 2.68 | 0.072 | 0.053 | 3.128 | -0.660 | 1.000 | 0.95 T | 1.31 T |
| 4.03 | 0.035 | 0.026 | 2.407 | -0.959 | 1.000 | 0.81 T | 1.12 T |
| 5.37 | -0.002 | -0.001 | 1.758 | -1.604 | 1.000 | 0.70 T | 0.98 T |
| 6.71 | -0.039 | -0.029 | 1.191 | -2.213 | 1.000 | 0.71 T | 0.99 T |
| 8.05 | -0.075 | -0.056 | 0.712 | -2.788 | 1.000 | 0.84 T | 1.17 T |
| 9.39 | -0.112 | -0.083 | 0.335 | -3.612 | 1.000 | 1.18 T | 1.62 T |
| 10.74 | -0.149 | -0.110 | 0.092 | -4.377 | 0.801 | 1.70 B | 2.32 B |
| 12.08 | -0.186 | -0.138 | 0.092 | -5.073 | 0.801 | 1.14 B | 1.59 B |
| 13.42 | -0.223 | -0.165 | 0.092 | -9.348 | 0.801 | 0.67 B | 0.95 B |

SPAN 2
=======

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.194 | 0.144 | 8.883 | -0.480 | 0.829 | 0.70 B | 0.99 B |
| 1.30 | 0.159 | 0.117 | 4.526 | -0.480 | 0.825 | 0.90 B | 1.26 B |
| 2.60 | 0.123 | 0.091 | 3.790 | -0.687 | 0.941 | 1.40 B | 1.91 B |
| 3.90 | 0.087 | 0.065 | 3.046 | -0.973 | 1.000 | 1.36 T | 1.86 T |
| 5.20 | 0.051 | 0.038 | 2.322 | -1.189 | 1.000 | 1.09 T | 1.49 T |
| 6.50 | 0.016 | 0.012 | 1.640 | -1.652 | 1.000 | 1.01 T | 1.39 T |
| 7.80 | -0.020 | -0.015 | 1.354 | -2.347 | 1.000 | 1.11 T | 1.53 T |
| 9.10 | -0.056 | -0.041 | 1.117 | -3.078 | 1.000 | 1.39 T | 1.91 T |
| 10.40 | -0.092 | -0.068 | 0.802 | -3.821 | 0.780 | 1.60 B | 2.18 B |
| 11.70 | -0.127 | -0.094 | 0.726 | -4.552 | 0.780 | 1.16 B | 1.60 B |
| 13.00 | -0.163 | -0.121 | 0.726 | -8.899 | 0.783 | 0.86 B | 1.20 B |

SPAN 3

=======

| X | SHEAR STRESSES DL1 | DL2 | +(LL+I) | -(LL+I) | ALLOW COMPR REDUCTION | RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.162 | 0.120 | 8.903 | -0.754 | 0.552 | 0.58 B | 0.82 B |
| 1.30 | 0.126 | 0.093 | 4.556 | -0.754 | 0.779 | 1.16 B | 1.60 B |
| 2.60 | 0.091 | 0.067 | 3.826 | -0.818 | 0.779 | 1.60 B | 2.18 B |
| 3.90 | 0.055 | 0.041 | 3.082 | -1.138 | 1.000 | 1.40 T | 1.92 T |
| 5.20 | 0.019 | 0.014 | 2.351 | -1.380 | 1.000 | 1.11 T | 1.53 T |
| 6.50 | -0.017 | -0.012 | 1.653 | -1.640 | 1.000 | 1.01 T | 1.39 T |
| 7.80 | -0.052 | -0.039 | 1.188 | -2.323 | 1.000 | 1.08 T | 1.49 T |
| 9.10 | -0.088 | -0.065 | 0.972 | -3.048 | 1.000 | 1.36 T | 1.85 T |
| 10.40 | -0.124 | -0.092 | 0.686 | -3.793 | 0.942 | 1.34 B | 1.84 B |
| 11.70 | -0.160 | -0.118 | 0.480 | -4.530 | 0.829 | 0.88 B | 1.22 B |
| 13.00 | -0.195 | -0.145 | 0.480 | -8.889 | 0.834 | 0.68 B | 0.96 B |

SPAN 4
=======

| X | SHEAR STRESSES DL1 | DL2 | +(LL+I) | -(LL+I) | ALLOW COMPR REDUCTION | RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.227 | 0.168 | 9.391 | -0.089 | 0.785 | 0.63 B | 0.90 B |
| 1.37 | 0.189 | 0.140 | 5.120 | -0.089 | 0.785 | 1.12 B | 1.56 B |
| 2.73 | 0.152 | 0.112 | 4.426 | -0.089 | 0.785 | 1.68 B | 2.30 B |
| 4.10 | 0.114 | 0.084 | 3.662 | -0.344 | 1.000 | 1.15 T | 1.58 T |
| 5.47 | 0.076 | 0.056 | 2.838 | -0.726 | 1.000 | 0.82 T | 1.13 T |
| 6.83 | 0.039 | 0.029 | 2.228 | -1.209 | 1.000 | 0.68 T | 0.95 T |
| 8.20 | 0.001 | 0.001 | 1.617 | -1.781 | 1.000 | 0.68 T | 0.95 T |
| 9.57 | -0.036 | -0.027 | 0.963 | -2.434 | 1.000 | 0.78 T | 1.09 T |
| 10.94 | -0.074 | -0.055 | 0.634 | -3.159 | 1.000 | 0.92 T | 1.27 T |
| 12.30 | -0.112 | -0.083 | 0.335 | -3.944 | 1.000 | 1.51 T | 2.08 T |
| 13.67 | -0.149 | -0.110 | 0.335 | -8.432 | 1.000 | 1.16 V | 1.58 V |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

SPAN 1
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 33.6 B | 26.9 | 38.1 | 0.94 V | 1.56 V | 39.6 | 0.94 V | 1.56 V |
| 1.34 | 33.6 B | 26.9 | 38.1 | 1.32 T | 2.20 T | 39.6 | 1.37 O | 2.29 O |
| 2.68 | 33.6 B | 26.9 | 38.1 | 0.81 T | 1.35 T | 39.6 | 0.84 O | 1.40 O |
| 4.03 | 33.6 B | 26.9 | 38.1 | 0.69 T | 1.15 T | 39.6 | 0.72 O | 1.20 O |
| 5.37 | 33.6 B | 26.9 | 38.1 | 0.60 T | 1.01 T | 39.6 | 0.63 O | 1.05 O |
| 6.71 | 33.6 B | 26.9 | 38.1 | 0.61 T | 1.01 T | 39.6 | 0.63 O | 1.05 O |
| 8.05 | 33.6 B | 26.9 | 38.1 | 0.72 T | 1.20 T | 39.6 | 0.75 O | 1.25 O |
| 9.39 | 33.6 B | 26.9 | 38.1 | 1.00 T | 1.66 T | 39.6 | 1.04 O | 1.73 O |
| 10.74 | -27.0 U | -21.6 | 38.1 | 1.43 B | 2.38 B | | | |
| 12.08 | -27.0 U | -21.6 | 38.1 | 0.98 B | 1.63 B | | | |
| 13.42 | -27.0 U | -21.6 | 38.1 | 0.59 B | 0.98 B | | | |

SPAN 2
=======

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | -27.9 U | -22.3 | 38.1 | 0.61 B | 1.02 B | | | |
| 1.30 | -27.8 U | -22.2 | 38.1 | 0.78 B | 1.29 B | | | |

```
  2.60    -31.7 U   -25.3    38.1   1.18 B   1.96 B
  3.90     33.6 B    26.9    38.1   1.14 T   1.91 T     39.6   1.19 O   1.98 O
  5.20     33.6 B    26.9    38.1   0.92 T   1.53 T     39.6   0.96 O   1.59 O
  6.50     33.6 B    26.9    38.1   0.86 T   1.43 T     39.6   0.89 O   1.48 O
  7.80     33.6 B    26.9    38.1   0.94 T   1.57 T     39.6   0.98 O   1.63 O
  9.10     33.6 B    26.9    38.1   1.18 T   1.96 T     39.6   1.23 O   2.04 O
 10.40    -26.3 U   -21.0    38.1   1.34 B   2.24 B
 11.70    -26.3 U   -21.0    38.1   0.99 B   1.64 B
 13.00    -26.4 U   -21.1    38.1   0.74 B   1.24 B

SPAN  3
=======
        NON-COMP OVERLOAD          NON-COMPACT     COMPACT      COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS   MOMENT   RATING FACTORS
   X    STRENGTH STRENGTH STRENGTH   IR        OR   STRENGTH    IR        OR
  0.00    -18.6 U   -14.9    38.1   0.51 B   0.85 B
  1.30    -26.2 U   -21.0    38.1   0.99 B   1.64 B
  2.60    -26.2 U   -21.0    38.1   1.34 B   2.24 B
  3.90     33.6 B    26.9    38.1   1.18 T   1.97 T     39.6   1.23 O   2.05 O
  5.20     33.6 B    26.9    38.1   0.94 T   1.57 T     39.6   0.98 O   1.63 O
  6.50     33.6 B    26.9    38.1   0.85 T   1.42 T     39.6   0.89 O   1.48 O
  7.80     33.6 B    26.9    38.1   0.92 T   1.53 T     39.6   0.95 O   1.59 O
  9.10     33.6 B    26.9    38.1   1.14 T   1.90 T     39.6   1.19 O   1.98 O
 10.40    -31.7 U   -25.4    38.1   1.13 B   1.89 B
 11.70    -27.9 U   -22.3    38.1   0.75 B   1.25 B
 13.00    -28.1 U   -22.4    38.1   0.59 B   0.99 B

SPAN  4
=======
        NON-COMP OVERLOAD          NON-COMPACT     COMPACT      COMPACT
         MOMENT   MOMENT   SHEAR   RATING FACTORS   MOMENT   RATING FACTORS
   X    STRENGTH STRENGTH STRENGTH   IR        OR   STRENGTH    IR        OR
  0.00    -26.4 U   -21.1    38.1   0.55 B   0.92 B
  1.37    -26.4 U   -21.1    38.1   0.96 B   1.60 B
  2.73    -26.4 U   -21.1    38.1   1.41 B   2.36 B
  4.10     33.6 B    26.9    38.1   0.97 T   1.62 T     39.6   1.01 O   1.69 O
  5.47     33.6 B    26.9    38.1   0.70 T   1.16 T     39.6   0.73 O   1.21 O
  6.83     33.6 B    26.9    38.1   0.59 T   0.98 T     39.6   0.61 O   1.02 O
  8.20     33.6 B    26.9    38.1   0.58 T   0.97 T     39.6   0.61 O   1.02 O
  9.57     33.6 B    26.9    38.1   0.67 T   1.12 T     39.6   0.70 O   1.16 O
 10.94     33.6 B    26.9    38.1   0.78 T   1.31 T     39.6   0.82 O   1.36 O
 12.30     33.6 B    26.9    38.1   1.28 T   2.13 T     39.6   1.33 O   2.22 O
 13.67     33.6 B    26.9    38.1   0.93 V   1.56 V     39.6   0.93 V   1.56 V


                +++++++++++++++++++++++++++++++++++++++++++++
                +                                           +
                +    F L O O R B E A M   A N A L Y S I S    +
                +                                           +
                +++++++++++++++++++++++++++++++++++++++++++++


                         FLOORBEAM SPAN INFORMATION
                        ANALYZING HALF THE FLOORBEAM


LEFT CANTILEVER:     0.00 FT, FLOORBEAM MAIN SPAN:     16.66 FT
RIGHT CANTILEVER:    0.00 FT
TOTAL FLOORBEAM SPAN (INCLUDING ANY EXISTING CANTILEVER):      16.66 FT


NUMBER OF TRAFFIC LANES:  1, WHERE LANE WIDTH IS BASED ON DESIGN LANE:
DECK WIDTH:   16.67 FT, ROADWAY WIDTH:   15.50 FT
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CL OF GIRDER OR TRUSS TO CURB: 0.58 FT

MINIMAL X1 AT ROADWAY: 2.58 FT, MAXIMAL X2 AT ROADWAY: 14.08 FT

NOTE: THE WHEEL LOAD POSITIONS (X1 AND X2) MUST BE BETWEEN MINIMAL_X1_AT_ROADWAY AND MAXIMAL_X2_AT_ROADWAY

FLOORBEAM LIVE LOAD MOMENT AND SHEAR FACTORS

| | | BASED ON DESIGN LANE ----------- WHEEL LOAD POSITIONS ----------- | | | | | |
|---|---|---|---|---|---|---|---|
| | | LANE 1/4/7 | | LANE 2/5/8 | | LANE 3/6/9 | |
| X | FACTOR | X1 | X2 | X1 | X2 | X1 | X2 |
| 0.00 | 0.000 M | 0.00 | 0.00 | | | | |
| | 0.665 V | 2.58 | 8.58 | | | | |
| 1.67 | 1.108 M | 2.58 | 8.58 | | | | |
| | 0.665 V | 2.58 | 8.58 | | | | |
| 3.33 | 2.056 M | 3.38 | 9.38 | | | | |
| | 0.617 V | 3.38 | 9.38 | | | | |
| 5.00 | 2.595 M | 4.98 | 10.98 | | | | |
| | 0.515 V | 5.08 | 11.08 | | | | |
| 6.66 | 2.792 M | 6.68 | 12.68 | | | | |
| | 0.419 V | 6.68 | 12.68 | | | | |
| 8.33 | 2.665 M | 2.58 | 8.58 | | | | |
| | 0.165 V | 2.58 | 8.58 | | | | |

MOMENT/SHEAR FACTOR CODES: M - MOMENT, V - SHEAR

NOTE: THE EFFECT OF A SET OF WHEELS PLACED TRANSVERSELY ACROSS THE BRIDGE (ALONG THE FLOORBEAM SPAN) TO FIND THE MAXIMUM POS AND NEG MOMENTS OR SHEARS EXPRESSED IN TERMS OF THE NUMBER OF AXLES IS REFERRED TO AS MOMENT OR SHEAR FACTORS.

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<    FLOORBEAM  2    >
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FLOORBEAM SECTION PROPERTIES
(UNIFORM SECTION)

| | DEPTH | GROSS AREA | MOMENT OF INERTIA | C BOTTOM | SECTION MODULUS TOP | SECTION MODULUS BOTTOM |
|---|---|---|---|---|---|---|
| NON-COMPOSITE | 20.62 | 8.75 | 547.15 | 11.13 | 57.65 | 49.14 |

DEAD LOADS ACTING ON THE FLOORBEAM

| UNIFORM LOAD FLOORBEAM WEIGHT | UNIFORM LOAD INPUT DL1FB | CONCENTRATED LOADS DIST | CONCENTRATED LOADS DL1 | CONCENTRATED LOADS DL2 |
|---|---|---|---|---|
| 0.030 | 0.000 | 1.200 | 0.915 | 0.677 |
| | | 3.530 | 0.915 | 0.677 |
| | | 6.070 | 0.915 | 0.677 |
| | | 8.490 | 0.915 | 0.677 |
| | | 10.740 | 0.915 | 0.677 |
| | | 13.280 | 0.915 | 0.677 |
| | | 15.660 | 0.915 | 0.677 |

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*  FLOORBEAM  2 -  LIVE LOAD  H20   *
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LIVE LOAD REACTION FROM DECK (ONE LANE):  36.56

LIVE LOAD IMPACT FACTORS: POS MOM 1.30
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UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 3.4 | 2.3 | 31.6 | 1.30 |
| 1.67 | 5.2 | 3.6 | 52.7 | 2.4 | 1.7 | 31.6 | 1.30 |
| 3.33 | 9.3 | 6.4 | 97.7 | 2.4 | 1.7 | 29.3 | 1.30 |
| 5.00 | 11.9 | 8.1 | 123.3 | 1.4 | 1.0 | 24.5 | 1.30 |
| 6.66 | 13.7 | 9.4 | 132.7 | 0.5 | 0.3 | 19.9 | 1.30 |
| 8.33 | 14.4 | 9.9 | 126.7 | 0.4 | 0.3 | 7.8 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.67 | -1.086 | -0.746 | -10.962 | 0.000 | 1.274 | 0.876 | 12.860 | 0.000 |
| 3.33 | -1.927 | -1.324 | -20.342 | 0.000 | 2.260 | 1.553 | 23.863 | 0.000 |
| 5.00 | -2.471 | -1.694 | -25.675 | 0.000 | 2.898 | 1.988 | 30.119 | 0.000 |
| 6.66 | -2.847 | -1.953 | -27.624 | 0.000 | 3.339 | 2.291 | 32.405 | 0.000 |
| 8.33 | -3.001 | -2.061 | -26.367 | 0.000 | 3.521 | 2.418 | 30.931 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.683 | 0.468 | 6.322 | 0.000 | 1.000 | 1.40 V | 1.97 V |
| 1.67 | 0.490 | 0.333 | 6.322 | 0.000 | 1.000 | 1.12 B | 1.58 B |
| 3.33 | 0.480 | 0.333 | 5.866 | 0.000 | 1.000 | 0.53 B | 0.78 B |
| 5.00 | 0.287 | 0.198 | 4.896 | 0.000 | 1.000 | 0.39 B | 0.58 B |
| 6.66 | 0.094 | 0.062 | 3.983 | 0.000 | 1.000 | 0.34 B | 0.52 B |
| 8.33 | 0.084 | 0.062 | 1.569 | 0.000 | 1.000 | 0.34 B | 0.54 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 122.9 B | 98.3 | 84.2 | 1.12 V | 1.87 V | 156.3 | 1.12 V | 1.87 V |
| 1.67 | 122.9 B | 98.3 | 84.2 | 0.98 B | 1.63 B | 156.3 | 1.02 O | 1.70 O |
| 3.33 | 122.9 B | 98.3 | 84.2 | 0.48 B | 0.81 B | 156.3 | 0.51 O | 0.85 O |
| 5.00 | 122.9 B | 98.3 | 84.2 | 0.36 B | 0.60 B | 156.3 | 0.38 O | 0.63 O |
| 6.66 | 122.9 B | 98.3 | 84.2 | 0.32 B | 0.54 B | 156.3 | 0.34 O | 0.57 O |
| 8.33 | 122.9 B | 98.3 | 84.2 | 0.33 B | 0.55 B | 156.3 | 0.35 O | 0.58 O |

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*************************************
*  FLOORBEAM  2 -  LIVE LOAD  HS20  *
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LIVE LOAD REACTION FROM DECK (ONE TRUCK): 38.63

LIVE LOAD IMPACT FACTORS: POS MOM 1.30

UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 3.4 | 2.3 | 33.4 | 1.30 |
| 1.67 | 5.2 | 3.6 | 55.6 | 2.4 | 1.7 | 33.4 | 1.30 |
| 3.33 | 9.3 | 6.4 | 103.2 | 2.4 | 1.7 | 31.0 | 1.30 |
| 5.00 | 11.9 | 8.1 | 130.3 | 1.4 | 1.0 | 25.9 | 1.30 |
| 6.66 | 13.7 | 9.4 | 140.2 | 0.5 | 0.3 | 21.0 | 1.30 |
| 8.33 | 14.4 | 9.9 | 133.8 | 0.4 | 0.3 | 8.3 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.67 | -1.086 | -0.746 | -11.582 | 0.000 | 1.274 | 0.876 | 13.587 | 0.000 |
| 3.33 | -1.927 | -1.324 | -21.492 | 0.000 | 2.260 | 1.553 | 25.212 | 0.000 |
| 5.00 | -2.471 | -1.694 | -27.126 | 0.000 | 2.898 | 1.988 | 31.821 | 0.000 |
| 6.66 | -2.847 | -1.953 | -29.185 | 0.000 | 3.339 | 2.291 | 34.237 | 0.000 |
| 8.33 | -3.001 | -2.061 | -27.858 | 0.000 | 3.521 | 2.418 | 32.680 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.683 | 0.468 | 6.680 | 0.000 | 1.000 | 1.32 V | 1.86 V |
| 1.67 | 0.490 | 0.333 | 6.680 | 0.000 | 1.000 | 1.06 B | 1.50 B |
| 3.33 | 0.480 | 0.333 | 6.197 | 0.000 | 1.000 | 0.50 B | 0.74 B |
| 5.00 | 0.287 | 0.198 | 5.173 | 0.000 | 1.000 | 0.36 B | 0.55 B |
| 6.66 | 0.094 | 0.062 | 4.208 | 0.000 | 1.000 | 0.32 B | 0.49 B |
| 8.33 | 0.084 | 0.062 | 1.658 | 0.000 | 1.000 | 0.32 B | 0.51 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 122.9 B | 98.3 | 84.2 | 1.06 V | 1.77 V | 156.3 | 1.06 V | 1.77 V |
| 1.67 | 122.9 B | 98.3 | 84.2 | 0.92 B | 1.54 B | 156.3 | 0.96 O | 1.61 O |
| 3.33 | 122.9 B | 98.3 | 84.2 | 0.46 B | 0.76 B | 156.3 | 0.48 O | 0.80 O |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 5.00 | 122.9 B | 98.3 | 84.2 | 0.34 B | 0.57 B | 156.3 | 0.36 O | 0.60 O |
| 6.66 | 122.9 B | 98.3 | 84.2 | 0.31 B | 0.51 B | 156.3 | 0.32 O | 0.54 O |
| 8.33 | 122.9 B | 98.3 | 84.2 | 0.31 B | 0.52 B | 156.3 | 0.33 O | 0.55 O |

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************************************
*  FLOORBEAM  2 -  LIVE LOAD TK527  *
************************************
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LIVE LOAD REACTION FROM DECK (ONE TRUCK): 50.96

LIVE LOAD IMPACT FACTORS: POS MOM 1.30

UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 3.4 | 2.3 | 44.1 | 1.30 |
| 1.67 | 5.2 | 3.6 | 73.4 | 2.4 | 1.7 | 44.1 | 1.30 |
| 3.33 | 9.3 | 6.4 | 136.2 | 2.4 | 1.7 | 40.9 | 1.30 |
| 5.00 | 11.9 | 8.1 | 171.9 | 1.4 | 1.0 | 34.1 | 1.30 |
| 6.66 | 13.7 | 9.4 | 185.0 | 0.5 | 0.3 | 27.8 | 1.30 |
| 8.33 | 14.4 | 9.9 | 176.5 | 0.4 | 0.3 | 10.9 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.67 | -1.086 | -0.746 | -15.278 | 0.000 | 1.274 | 0.876 | 17.923 | 0.000 |
| 3.33 | -1.927 | -1.324 | -28.350 | 0.000 | 2.260 | 1.553 | 33.257 | 0.000 |
| 5.00 | -2.471 | -1.694 | -35.782 | 0.000 | 2.898 | 1.988 | 41.976 | 0.000 |
| 6.66 | -2.847 | -1.953 | -38.499 | 0.000 | 3.339 | 2.291 | 45.163 | 0.000 |
| 8.33 | -3.001 | -2.061 | -36.747 | 0.000 | 3.521 | 2.418 | 43.108 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.683 | 0.468 | 8.811 | 0.000 | 1.000 | 1.00 V | 1.41 V |
| 1.67 | 0.490 | 0.333 | 8.811 | 0.000 | 1.000 | 0.80 B | 1.14 B |
| 3.33 | 0.480 | 0.333 | 8.175 | 0.000 | 1.000 | 0.38 B | 0.56 B |
| 5.00 | 0.287 | 0.198 | 6.823 | 0.000 | 1.000 | 0.28 B | 0.42 B |
| 6.66 | 0.094 | 0.062 | 5.551 | 0.000 | 1.000 | 0.24 B | 0.37 B |
| 8.33 | 0.084 | 0.062 | 2.187 | 0.000 | 1.000 | 0.24 B | 0.38 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 122.9 B | 98.3 | 84.2 | 0.80 V | 1.34 V | 156.3 | 0.80 V | 1.34 V |
| 1.67 | 122.9 B | 98.3 | 84.2 | 0.70 B | 1.17 B | 156.3 | 0.73 O | 1.22 O |
| 3.33 | 122.9 B | 98.3 | 84.2 | 0.35 B | 0.58 B | 156.3 | 0.36 O | 0.61 O |
| 5.00 | 122.9 B | 98.3 | 84.2 | 0.26 B | 0.43 B | 156.3 | 0.27 O | 0.46 O |
| 6.66 | 122.9 B | 98.3 | 84.2 | 0.23 B | 0.39 B | 156.3 | 0.24 O | 0.41 O |
| 8.33 | 122.9 B | 98.3 | 84.2 | 0.24 B | 0.40 B | 156.3 | 0.25 O | 0.42 O |

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*************************************
*  FLOORBEAM  2 -  LIVE LOAD  ML80  *
*************************************
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LIVE LOAD REACTION FROM DECK (ONE TRUCK):  56.67

LIVE LOAD IMPACT FACTORS: POS MOM 1.30

UNFACTORED MOMENTS AND SHEARS

| X | DL1 MOMENT | DL2 MOMENT | LL+I MOMENT | DL1 SHEAR | DL2 SHEAR | LL+I SHEAR | I.F. |
|---|---|---|---|---|---|---|---|
| 0.00 | 0.0 | 0.0 | 0.0 | 3.4 | 2.3 | 49.0 | 1.30 |
| 1.67 | 5.2 | 3.6 | 81.6 | 2.4 | 1.7 | 49.0 | 1.30 |
| 3.33 | 9.3 | 6.4 | 151.5 | 2.4 | 1.7 | 45.5 | 1.30 |
| 5.00 | 11.9 | 8.1 | 191.2 | 1.4 | 1.0 | 37.9 | 1.30 |
| 6.66 | 13.7 | 9.4 | 205.7 | 0.5 | 0.3 | 30.9 | 1.30 |
| 8.33 | 14.4 | 9.9 | 196.3 | 0.4 | 0.3 | 12.2 | 1.30 |

NOTE: SYMBOL NEXT TO X
R: IT IS A RANGE POINT
B: IT IS BOTH A RANGE POINT AND ANALYSIS POINT

FLEXURAL STRESSES - BEAM

| | TOP FIBER STEEL STRESS | | | | BOTTOM FIBER STEEL STRESS | | | |
|---|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | DL1 | DL2 | +(LL+I) | -(LL+I) |
| 0.00 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 1.67 | -1.086 | -0.746 | -16.991 | 0.000 | 1.274 | 0.876 | 19.932 | 0.000 |
| 3.33 | -1.927 | -1.324 | -31.528 | 0.000 | 2.260 | 1.553 | 36.986 | 0.000 |
| 5.00 | -2.471 | -1.694 | -39.794 | 0.000 | 2.898 | 1.988 | 46.682 | 0.000 |
| 6.66 | -2.847 | -1.953 | -42.815 | 0.000 | 3.339 | 2.291 | 50.226 | 0.000 |
| 8.33 | -3.001 | -2.061 | -40.867 | 0.000 | 3.521 | 2.418 | 47.941 | 0.000 |

SHEAR STRESSES AND ALLOWABLE STRESS RATINGS

| | SHEAR STRESSES | | | | ALLOW COMPR | RATING FACTORS | |
|---|---|---|---|---|---|---|---|
| X | DL1 | DL2 | +(LL+I) | -(LL+I) | REDUCTION | IR | OR |
| 0.00 | 0.683 | 0.468 | 9.799 | 0.000 | 1.000 | 0.90 V | 1.27 V |
| 1.67 | 0.490 | 0.333 | 9.799 | 0.000 | 1.000 | 0.72 B | 1.02 B |
| 3.33 | 0.480 | 0.333 | 9.092 | 0.000 | 1.000 | 0.34 B | 0.51 B |
| 5.00 | 0.287 | 0.198 | 7.588 | 0.000 | 1.000 | 0.25 B | 0.38 B |
| 6.66 | 0.094 | 0.062 | 6.173 | 0.000 | 1.000 | 0.22 B | 0.34 B |
| 8.33 | 0.084 | 0.062 | 2.432 | 0.000 | 1.000 | 0.22 B | 0.35 B |

NOTE: THE SHEAR CAPACITIES CALCULATED HEREIN ARE BASED ON STIFFENED OR UNSTIFFENED EQUATIONS AS SPECIFIED BY INPUT REGARDLESS OF THE STIFFENER SPACINGS INPUT AND ARE NOT CHECKED AGAINST AASHTO CRITERIA.

STRENGTHS AND LOAD FACTOR RATINGS

| X | NON-COMP MOMENT STRENGTH | OVERLOAD MOMENT STRENGTH | SHEAR STRENGTH | NON-COMPACT RATING FACTORS IR | NON-COMPACT RATING FACTORS OR | COMPACT MOMENT STRENGTH | COMPACT RATING FACTORS IR | COMPACT RATING FACTORS OR |
|---|---|---|---|---|---|---|---|---|
| 0.00 | 122.9 B | 98.3 | 84.2 | 0.72 V | 1.21 V | 156.3 | 0.72 V | 1.21 V |
| 1.67 | 122.9 B | 98.3 | 84.2 | 0.63 B | 1.05 B | 156.3 | 0.66 O | 1.10 O |
| 3.33 | 122.9 B | 98.3 | 84.2 | 0.31 B | 0.52 B | 156.3 | 0.33 O | 0.55 O |
| 5.00 | 122.9 B | 98.3 | 84.2 | 0.23 B | 0.39 B | 156.3 | 0.25 O | 0.41 O |
| 6.66 | 122.9 B | 98.3 | 84.2 | 0.21 B | 0.35 B | 156.3 | 0.22 O | 0.37 O |
| 8.33 | 122.9 B | 98.3 | 84.2 | 0.21 B | 0.36 B | 156.3 | 0.23 O | 0.38 O |

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+++++++++++++++++++++++++++++++++++++++++++++
+                                           +
+        R A T I N G   S U M M A R Y        +
+                                           +
+++++++++++++++++++++++++++++++++++++++++++++
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MEMBER: TRUSS

| LOAD | | ALLOWABLE STRESS RATING FACTOR | TONS | MEMBER | LOAD FACTOR RATING FACTOR | TONS | MEMBER |
|---|---|---|---|---|---|---|---|
| H20 | IR (CRITICAL) | 0.66 | 13.1 | U 2L 3 | ***** | ***** | ****** |
| | OR (CRITICAL) | 0.92 | 18.5 | U 2L 3 | ***** | ***** | ****** |
| HS20 | IR (CRITICAL) | 0.64 | 23.0 | U 1L 2 | ***** | ***** | ****** |
| | OR (CRITICAL) | 0.92 | 33.3 | U 2L 3 | ***** | ***** | ****** |
| TK527 | IR (CRITICAL) | 0.49 | 19.7 | U 2L 3 | ***** | ***** | ****** |
| | OR (CRITICAL) | 0.69 | 27.7 | U 2L 3 | ***** | ***** | ****** |
| ML80 | IR (CRITICAL) | 0.43 | 15.9 | U 2L 3 | ***** | ***** | ****** |
| | OR (CRITICAL) | 0.61 | 22.4 | U 2L 3 | ***** | ***** | ****** |

NOTE: LOAD FACTOR RATING WAS NOT PERFORMED BECAUSE ADDITIONAL TRUSS MEMBER PROPERTIES WERE NOT INPUT BY THE USER.

MEMBER: STRINGER AT 6.070

| LOAD | | ALLOWABLE STRESS RATING FACTOR | TONS | X | SPAN | LOAD FACTOR RATING FACTOR | TONS | X | SPAN |
|---|---|---|---|---|---|---|---|---|---|
| H20 | IR (CRITICAL) | 0.73 T | 14.6 | 8.20 | 4 | 0.63 T | 12.6 | 8.20 | 4 |
| | OR (CRITICAL) | 1.02 T | 20.4 | 8.20 | 4 | 1.05 T | 21.0 | 8.20 | 4 |
| | IR ( POS MOM) | 0.73 T | 14.6 | 8.20 | 4 | 0.63 T | 12.6 | 8.20 | 4 |
| | OR ( POS MOM) | 1.02 T | 20.4 | 8.20 | 4 | 1.05 T | 21.0 | 8.20 | 4 |
| | IR ( NEG MOM) | 0.76 B | 15.3 | 13.00 | 3 | 0.67 B | 13.4 | 13.00 | 3 |
| | OR ( NEG MOM) | 1.09 B | 21.8 | 13.00 | 3 | 1.12 B | 22.4 | 13.00 | 3 |
| HS20 | IR (CRITICAL) | 0.60 B | 21.7 | 13.00 | 3 | 0.53 B | 19.2 | 13.00 | 3 |
| | OR (CRITICAL) | 0.86 B | 31.1 | 13.00 | 3 | 0.89 B | 32.0 | 13.00 | 3 |
| | IR ( POS MOM) | 0.70 T | 25.3 | 8.20 | 4 | 0.61 T | 21.8 | 8.20 | 4 |
| | OR ( POS MOM) | 0.98 T | 35.4 | 8.20 | 4 | 1.01 T | 36.3 | 8.20 | 4 |
| | IR ( NEG MOM) | 0.60 B | 21.7 | 13.00 | 3 | 0.53 B | 19.2 | 13.00 | 3 |
| | OR ( NEG MOM) | 0.86 B | 31.1 | 13.00 | 3 | 0.89 B | 32.0 | 13.00 | 3 |
| TK527 | IR (CRITICAL) | 0.62 B | 24.8 | 0.00 | 3 | 0.54 B | 21.5 | 0.00 | 3 |
| | OR (CRITICAL) | 0.87 B | 34.9 | 0.00 | 3 | 0.90 B | 35.9 | 0.00 | 3 |
| | IR ( POS MOM) | 0.75 T | 30.0 | 8.20 | 4 | 0.64 T | 25.8 | 8.20 | 4 |
| | OR ( POS MOM) | 1.05 T | 41.8 | 8.20 | 4 | 1.07 T | 43.0 | 8.20 | 4 |
| | IR ( NEG MOM) | 0.62 B | 24.8 | 0.00 | 3 | 0.54 B | 21.5 | 0.00 | 3 |
| | OR ( NEG MOM) | 0.87 B | 34.9 | 0.00 | 3 | 0.90 B | 35.9 | 0.00 | 3 |
| ML80 | IR (CRITICAL) | 0.58 B | 21.4 | 0.00 | 3 | 0.51 B | 18.6 | 0.00 | 3 |
| | OR (CRITICAL) | 0.82 B | 30.2 | 0.00 | 3 | 0.85 B | 31.1 | 0.00 | 3 |
| | IR ( POS MOM) | 0.68 T | 24.9 | 8.20 | 4 | 0.58 T | 21.4 | 8.20 | 4 |
| | OR ( POS MOM) | 0.95 T | 34.7 | 8.20 | 4 | 0.97 T | 35.7 | 8.20 | 4 |
| | IR ( NEG MOM) | 0.58 B | 21.4 | 0.00 | 3 | 0.51 B | 18.6 | 0.00 | 3 |
| | OR ( NEG MOM) | 0.82 B | 30.2 | 0.00 | 3 | 0.85 B | 31.1 | 0.00 | 3 |

NOTE: THIS RATING SUMMARY OF STRINGERS DOES FOLLOW THE ALL-OR-NONE COMPACT REQUIREMENTS.
THIS STRINGER DOES NOT HAVE ALL SECTIONS QUALIFYING AS COMPACT

SECTIONS. THEREFORE, EVEN THOUGH SOME SECTION WITH THE MINIMUM RATING QUALIFIES AS A COMPACT SECTION,THE RATING PRINTED IN THE SUMMARY IS THAT BASED ON A NON-COMPACT SECTION.

MEMBER: FLOORBEAM

| | | ALLOWABLE STRESS RATING | | | | LOAD FACTOR RATING | | | |
|---|---|---|---|---|---|---|---|---|---|
| LOAD | | FACTOR | TONS | X | FLBM | FACTOR | TONS | X | FLBM |
| H20 | IR (CRITICAL) | 0.34 B | 6.7 | 6.66 | 2 | 0.34 O | 6.8 | 6.66 | 2 |
| | OR (CRITICAL) | 0.52 B | 10.4 | 6.66 | 2 | 0.57 O | 11.3 | 6.66 | 2 |
| | IR ( POS MOM) | 0.34 B | 6.7 | 6.66 | 2 | 0.34 O | 6.8 | 6.66 | 2 |
| | OR ( POS MOM) | 0.52 B | 10.4 | 6.66 | 2 | 0.57 O | 11.3 | 6.66 | 2 |
| HS20 | IR (CRITICAL) | 0.32 B | 11.4 | 6.66 | 2 | 0.32 O | 11.6 | 6.66 | 2 |
| | OR (CRITICAL) | 0.49 B | 17.7 | 6.66 | 2 | 0.54 O | 19.3 | 6.66 | 2 |
| | IR ( POS MOM) | 0.32 B | 11.4 | 6.66 | 2 | 0.32 O | 11.6 | 6.66 | 2 |
| | OR ( POS MOM) | 0.49 B | 17.7 | 6.66 | 2 | 0.54 O | 19.3 | 6.66 | 2 |
| TK527 | IR (CRITICAL) | 0.24 B | 9.6 | 6.66 | 2 | 0.24 O | 9.8 | 6.66 | 2 |
| | OR (CRITICAL) | 0.37 B | 14.9 | 6.66 | 2 | 0.41 O | 16.3 | 6.66 | 2 |
| | IR ( POS MOM) | 0.24 B | 9.6 | 6.66 | 2 | 0.24 O | 9.8 | 6.66 | 2 |
| | OR ( POS MOM) | 0.37 B | 14.9 | 6.66 | 2 | 0.41 O | 16.3 | 6.66 | 2 |
| ML80 | IR (CRITICAL) | 0.22 B | 7.9 | 6.66 | 2 | 0.22 O | 8.0 | 6.66 | 2 |
| | OR (CRITICAL) | 0.34 B | 12.3 | 6.66 | 2 | 0.37 O | 13.4 | 6.66 | 2 |
| | IR ( POS MOM) | 0.22 B | 7.9 | 6.66 | 2 | 0.22 O | 8.0 | 6.66 | 2 |
| | OR ( POS MOM) | 0.34 B | 12.3 | 6.66 | 2 | 0.37 O | 13.4 | 6.66 | 2 |

NOTE: THIS RATING SUMMARY OF FLOORBEAMS DOES FOLLOW THE ALL-OR-NONE COMPACT REQUIREMENTS.
THIS FLOORBEAM DOES HAVE ALL SECTIONS QUALIFYING AS COMPACT SECTIONS. THEREFORE, THE RATING PRINTED IN THE SUMMARY IS THAT BASED ON COMPACT SECTIONS.

RATING FACTOR CODES: FIRST CHARACTER AFTER THE RATING FACTOR
T - TOP STEEL STRESS/STRENGTH GOVERNS
B - BOTTOM STEEL STRESS/STRENGTH GOVERNS
C - CONCRETE STRESS/STRENGTH GOVERNS
R - REINFORCEMENT STRESS/STRENGTH GOVERNS
V - SHEAR STRESS/STRENGTH GOVERNS
blank - COMPACT MOMENT STRENGTH GOVERNS
O - OVERLOAD PROVISIONS GOVERN
I - MOMENT-SHEAR INTERACTION GOVERNS

RATING FACTOR CODES FOR FLANGE/WEB BUCKLING: SECOND CHARACTER AFTER THE RATING FACTOR
blank - SECTION DOES MEET FLANGE/WEB BUCKLING CRITERIA AND WILL NOT BUCKLE.
F - SECTION DOES NOT MEET FLANGE PROJECTION/THICKNESS RATIO CRITERIA AND WILL BUCKLE IN FLANGE.
W - SECTION DOES NOT MEET WEB DEPTH/THICKNESS RATIO CRITERIA AND WILL BUCKLE IN WEB.
Z - SECTION DOES NOT MEET BOTH FLANGE PROJECTION/THICKNESS RATIO CRITERIA AND WEB DEPTH/THICKNESS RATIO CRITERIA AND WILL BUCKLE IN FLANGE OR WEB.

NON-COMPACT MOMENT STRENGTH CODES:
B - SECTION IS BRACED
U - SECTION IS UNBRACED

NOTE: ALL RATINGS ARE BASED ON THE NUMBER OF DESIGN LANES OR THE ACTUAL TRAFFIC LANES AS DEFINED BY "D" OR "L" ENTERED FOR LANES IN THE PROJECT IDENTIFICATION.

UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
< 0.1 * (DL1+DL2) AT SUPPORT 1 FOR LIVE LOAD H20

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    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  3 FOR LIVE LOAD  H20

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  5 FOR LIVE LOAD  H20

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  1 FOR LIVE LOAD  HS20

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  3 FOR LIVE LOAD  HS20

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  5 FOR LIVE LOAD  HS20

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  1 FOR LIVE LOAD TK527

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  3 FOR LIVE LOAD TK527

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  5 FOR LIVE LOAD TK527

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  1 FOR LIVE LOAD  ML80

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  3 FOR LIVE LOAD  ML80

    UPLIFT WARNING - TOTAL STRINGER REACTION < 0 OR
                     < 0.1 * (DL1+DL2) AT SUPPORT  5 FOR LIVE LOAD  ML80

BAR7 v7.15.0.0 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

## 3.12 BOX CULVERTS AND RIGID FRAMES

This Section covers the rating of reinforced concrete box culverts and rigid frames.

### 3.12.1 Policies and Guidelines

Most box culverts and rigid frames bridges can be rated by analytical methods based on design plans and field measurements. If design plans are not available, PennDOT standard plans can be utilized if field measured values are in reasonable agreement with one of the standard plans. Culverts and rigid frames constructed after 1990 were typically designed individually, and not taken from a design table. Therefore, these culverts can not be rated from field measurements.

If plans are not available and field measurements are not in reasonable agreement with PennDOT standard plans, the engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.12.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating.

#### 3.12.2.1 LFR or ASR Method

Box culverts and rigid frames designed with the LFD or ASD method should be rated using PennDOT's BOX5 program when the necessary information is available to complete an analytical rating.

#### 3.12.2.2 LRFR Method

Box culverts and rigid frames designed with the LRFD method on or after 2011, the bridge should be rated using PennDOT's BXLRFD program when the necessary information is available to complete an analytical rating.

### 3.12.3 Live Load and Dead Load Distribution

Regardless of the rating method or age, the amount of fill over the structure will determine the method in which the wheel loads are distributed through the fill to the top of the structure. See Section 2.5.7 for additional discussion on distribution of loads on culverts.

#### 3.12.3.1 LFR or ASR Method

Distribution factors may be calculated within the BOX5 program. Box culverts and rigid frames designed with the LFD or ASD method and are under less than 2 ft of fill (considered 'At Grade'), see AASHTO Std. Spec. 6.4.2 and Section 3.4.3 of this manual for distribution factor guidance for Slabs. For box culverts and rigid frames designed with the LFD or ASD method and the fill depth is 2 feet or greater, see AASHTO Std. Spec. 6.4. Per AASHTO Std. Spec. 6.4.1, concentrated loads shall be considered as uniformly distributed over a square with sides equal to 1.75 times the depth of fill. Per AASHTO Std. Spec. 6.4.2, the effect of live load may be neglected if:

- Single Spans: Depth of fill exceeds the greater of 8 ft and the span length.

- Multiple Spans: Depth of fill exceeds the distance between the faces of the end supports or abutments.

#### 3.12.3.2 LRFR Method

Distribution factors may be calculated within the BXLRFD program. For box culverts and rigid frames designed with the LRFD and to be rated using LRFD, see AASHTO LRFD 3.6.1.2.6 for distribution of wheel load through earth fills. Per AASHTO LRFD Table 3.6.1.2.6a-1, the Live Load Distribution Factor for all culverts (except concrete pipe) and buried structures is to be 1.15.

### 3.12.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1918-1930 Volume 1 | Standard Reinforced Concrete Boxes | 11/20/1929 | S-333 |
| Standards for Old Bridges 1961-1965 Volume 4 | Standard Reinforced Concrete Boxes | Various | S-2729, ST-131 |
| Standards for Old Bridges 1965-1972 Volume 5 | Standard Reinforced Concrete box Culverts Details | 12/17/1969 | ST-132 |
| BD-100 Series/BD-100 Series, 1970 Edition | Standard Reinforced Concrete Box Culverts | 9/1/1970 | BD-132 |

### 3.12.5 Modeling Section Properties and Deterioration

See Section 2.5.1.2 for a discussion of modeling section loss.

See Section 3.4.5 for a discussion of adjusting the model for bars that have been deemed ineffective.

### 3.12.6 Standard Practices

The provisions of this Section apply to culverts/rigid frames with clear spans 8’-0” and greater. If the span length is less than 8’-0”, the culvert/rigid frame is not considered a structure.

The BOX5 program is known to be conservative when analyzing and rating bottom slabs and the Department has issued guidance to ignore the bottom slab rating when using this program when the bottom slab exhibits no visible distress.

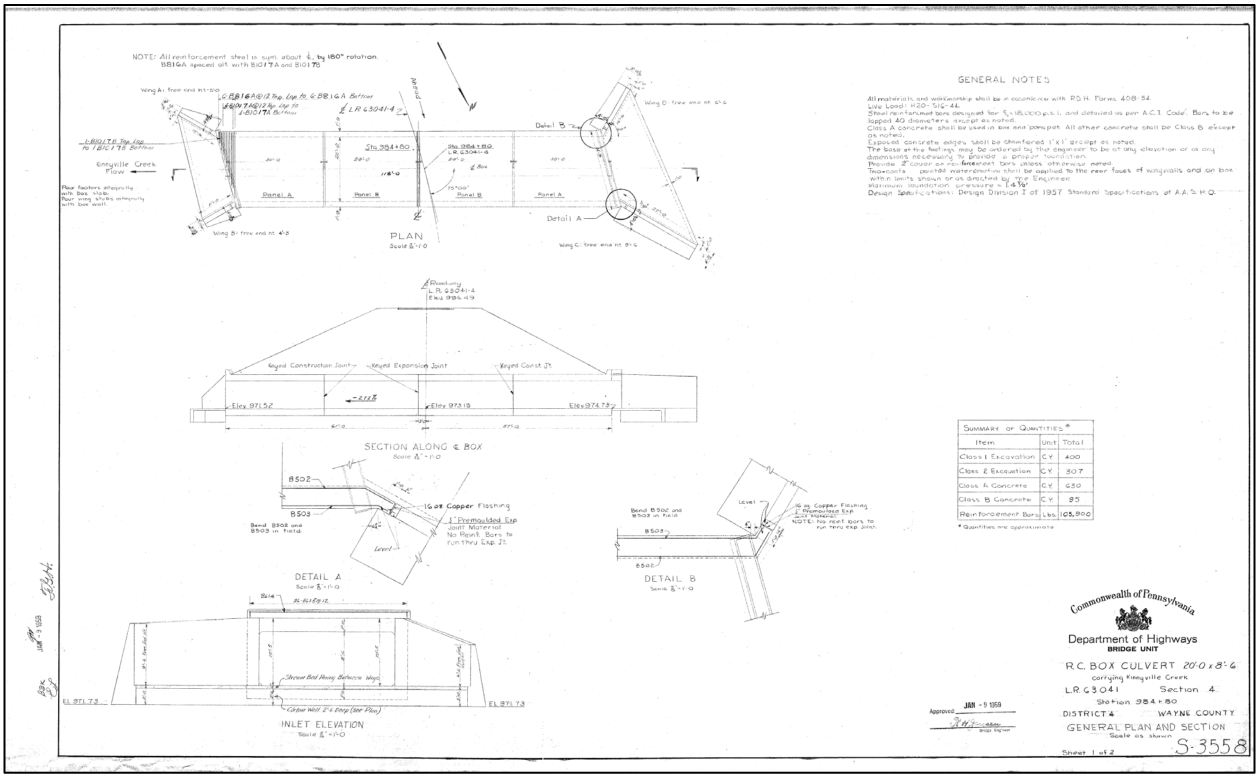
### 3.12.7 Common QA Findings

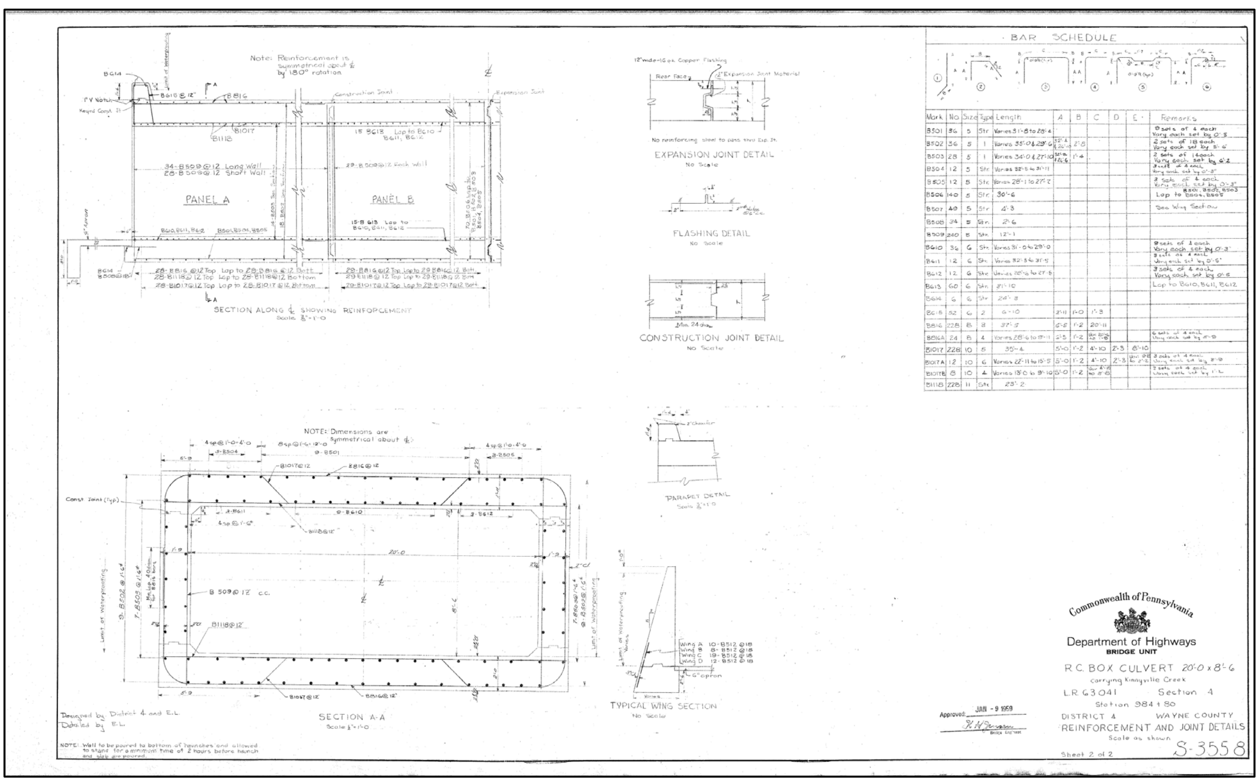
This Section is in development and will be provided in future editions of this manual.

### 3.12.8 Sample Load Rating

This Section contains sample load rating calculations for a cast-in-place box culvert shown below. The analysis will be performed using PennDOT’s Box Culvert Design and Rating (BOX5) Program, Version 7.9.0.1 and based on the Load Factor Design Method (LFD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part IV (DM-4).

The box culvert has an 8’-6” vertical opening by a 20’-0” horizontal opening. It supports two lanes of vehicular traffic with a fill height of 12.9 feet.


GENERAL NOTES
SUMMARY OF QUANTITIES
Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT
R.C. BOX CULVERT 20'-0 x 8'-6
GENERAL PLAN AND SECTION
S-3558


BAR SCHEDULE
Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT
R.C. BOX CULVERT 20'-0 x 8'-6
REINFORCEMENT AND JOINT DETAILS
S-3558

3.12.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

Done By: ABC Date:

Checked By: XYZ Date:

Structure ID (5A01): 63-0247-0420-1968 Inspection Date (7A01): 8/8/2021

Facility Carried (5A08): SR 247

Feature Intersected (5A07): SR 247 (White Rock Drive) over Kinneyville Creek

Structure Type (6A26 - 6A29): Single Cell Cast-In-Place Reinforced Concrete Box Culvert

Spans / Members Analyzed: Top Slab, Bottom Slab, Walls

Analysis Method: LFD

PennDOT Program / Version: BOX5 Version 5.9.0.1

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 7.97 | 159.5 | 13.31 | 266.3 | 11.97 | 239.7 | * | * | V | V |
| HS20 | 5.69 | 204.9 | 9.50 | 342.1 | 8.55 | 307.9 | * | * | V | V |
| ML80 | 4.66 | 170.9 | 7.79 | 285.3 | 7.01 | 256.7 | * | * | V | V |
| TK527 | 4.94 | 197.7 | 8.25 | 330.1 | 7.42 | 297.0 | * | * | V | V |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 6.42 | 184.7 | 10.73 | 308.4 | 9.66 | 277.5 | * | * | V | V |
| EV3 | 4.40 | 189.1 | 7.34 | 315.8 | 6.60 | 284.2 | * | * | V | V |

Comments/Assumptions*:

Culvert condition rating is 4. Therefore, for an ADTT < 500, the SLC Factor equals 0.9 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is in Slab 2 at 1.69 ft.
BOX5 analysis does not include any section loss or deterioration.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

#### 3.12.8.2 Box Culvert Load Rating Analysis

##### 3.12.8.2.1 BOX5 Input Parameters

The cast-in-place box culvert will be rated using BOX5. Many of the BOX5 input parameters can be left blank or are self-explanatory. Only the pertinent input values are discussed below. Refer to the BOX5 User's Manual for additional information.

Project Identification

- Project Identification = "=BOXCL"
- Structure ID = 63024704201968
- Description = CULVERT LOAD RATING

Specifications

- Method = LF for LFD
- Run Type = Z for cast-in-place culverts
- Live Load (when Z is specified for Run Type) = 0 for H20, HS20, ML80, TK527 and P-82 vehicles. Enter 2 for EV2 and EV3 vehicles.
- Top Slab = M for the top slab acting monolithically with the walls
- Unit Weight of Earth = 120 pcf (normal value per BOX5 User's Manual)
- Equivalent Fluid Pressure = 35 pcf (normal value per BOX5 User's Manual)
- Concrete Compressive Strength, $f'_c$ = 3000 psi for Class A Concrete per Contract Drawings, See Pub 238 Table IP 3.7.2.2-1.
- Rebar Grade = 40 ksi
- Rebar Diameter = 1.41 in (#11 bar max per Contract Drawings)
- Specs = 3 to use 1973 or earlier AASHTO Specifications
- Live Load Surcharge = 0 (Note: Ratings actually increased when a 3 ft surcharge was applied. Consider checking the minimum and maximum live load surcharge height.)
- Axial Force = Y to include the effect of axial force
- Output = 2 for rating summary with input echo

Culvert Data

- Clear Span = 20' – 0" per Contract Drawings
- Clear Height = 8'-6" per Contract Drawings
- Top Slab Thickness = 23 in per Contract Drawings
- Bottom Slab Thickness = 24 in per Contract Drawings
- Left Wall Thickness = 21 in per Contract Drawings
- Right Wall Thickness = 21 in per Contract Drawings
- Height of Fill = Elev 996.49 – (Elev 973.18 + 8.50 ft + 1.917 ft) = 12.9 ft
- Top Slab Bar Cover (Top and Bottom) = 2 in per Contract Drawings
- Bottom Slab Bar Cover (Top and Bottom) = 2.50 in per Contract Drawings
- Wall Bar Cover (Each Face) = 2.00 in per Contract Drawings

Haunch Data

- Per the Contract Drawings, corner haunch dimensions are 6 in x 6 in each.

Wall Reinforcement
The reinforcement sketch below is taken from the BOX5 User's Manual. The flexural reinforcement will be included for analysis in BOX5. However, the shear reinforcement will not be analyzed since the bent up bar is located farther than a "d" distance away from the inside face of the wall/haunch. Refer to the Contract Drawings and BOX5 User's Manual for additional information.

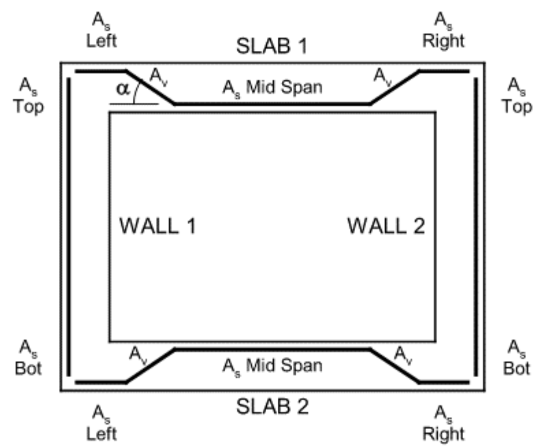



Figure 5.8.1 Wall and Slab Reinforcement

- Wall Reinforcement Area (Top & Bottom) = 1.27 in$^2$ (#10's @ 12") + 0.79 in$^2$ (#8's @ 12") = 2.06 in$^2$ in each corner.
- Slab Reinforcement Area (Left & Right) = 1.27 in$^2$ (#10's @ 12") + 0.79 in$^2$ (#8's @ 12") = 2.06 in$^2$ in each corner. The critical section for the end of slab reinforcement on the outside face is at the interface of the wall and slab/haunch. This is the largest negative moment region in the slab. The #8 bars and the #10 bars are developed and considered 100% effective.
- Slab Reinforcement Area (Midspan) = 1.56 in$^2$ (#11's @ 12") + 1.27 in$^2$ (#10's @ 12") = 2.83 in$^2$ at midpsan. The critical section for positive moment in the slab is at midspan. This is the largest positive moment in the slab. The #11 bars and the #10 bars are developed and considered 100% effective.

**PROJECT IDENTIFICATION**

| | | |
|---|---|---|
| County: | 63 | |
| SR: | 247 | |
| Segment: | 420 | |
| Offset: | 1968 | |
| Description: | Culvert Load Rating | |

**SPECIFICATIONS**

| | | | |
|---|---|---|---|
| Method: | LF | | Load Factor Specifications |
| Run Type: | Z | | Cast-in-place culvert |
| Live Load: | 0 | | H20, HS20, ML80, and TK527 |
| Top Slab: | M | | Monolithic with walls |
| Unit Weight: | 120.00 | pcf | Default = 120 pcf |
| Equiv Fluid Pressure: | 35.00 | pcf | Default = 35 pcf |
| Compressive Strength, $f'_c$: | 3000 | psi | Per Contract Drawings |
| Rebar Grade, $f_y$: | 40 | ksi | Per Contract Drawings |
| Rebar Diameter: | 1.41 | in | # 11 bar max per Contract Drawings |
| Specs: | 3 | | 3 for 1973 specs or earlier |
| Live Load Surcharge: | 0 | ft | |
| Axial Force:: | Y | | Y to include axial force effects |
| Output: | 0 | | Normal output |

**CULVERT DATA**

| | | | |
|---|---|---|---|
| Clear Span: | 20.00 | ft | Per Contract Drawings |
| Clear Height: | 8.50 | ft | Per Contract Drawings |
| Top Slab Thickness: | 23.00 | in | Per Contract Drawings |
| Bottom Slab Thickness: | 24.00 | in | Per Contract Drawings |
| Left Wall Thickness: | 21.00 | in | Per Contract Drawings |
| Right Wall Thickness: | 21.00 | in | Per Contract Drawings |
| Height of Fill: | 12.90 | ft | Per Contract Drawings |
| Top Slab Bar Cover: | 2.00 | in | Per Contract Drawings |
| Bottom Slab Bar Cover: | 2.50 | in | Per Contract Drawings |
| Wall Bar Cover: | 2.00 | in | Per Contract Drawings |

**HAUNCH DIMENSIONS**

| | | | |
|---|---|---|---|
| X Dimension: | 6.00 | in | For all haunches per Contract Drawings |
| Y Dimension: | 6.00 | in | For all haunches per Contract Drawings |

3.12.8.2.2 BOX5 Output

```
*****************************************************************************
*                                                                           *
*                   BOX CULVERT DESIGN AND RATING (BOX5)             330139 *
*                                                                           *
*                          COPYRIGHT (C) 1990-2017                          *
*                                                                           *
*                        COMMONWEALTH OF PENNSYLVANIA                       *
*                        DEPARTMENT OF TRANSPORTATION                       *
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*****************************************************************************
```

```
                    BOX CULVERT DESIGN AND RATING (BOX5)              330139

PROGRAM P4356010                                                06/12/2024 13:41
VERSION 5.9.0.1            LAST UPDATED 06/02/2017        DOCUMENTATION 06/2017

INPUT: CIP Concrete Box Culvert BOX5 Analysis.dat




   STRUCTURE IDENTIFICATION     SPAN
CNTY    SR     SEGMENT  OFFSET   ID          STRUCTURE DESCRIPTION
 63    0247     0420     1968         CULVERT LOAD RATING

                                  SPECIFICATIONS

          RUN    BOTTOM           FISH    LIVE   NO OF   TOP    NO OF
  METHOD   TYPE    SLAB   HAUNCH  CHANNEL  LOAD   CELLS   SLAB   LANES
   LFD      Z        Y       Y              0       1      M        2

            LOAD FACTORS                UNIT   EQUIV        f'c TOP
                      BETA E  BETA E   WEIGHT  FLUID        SLAB AT  REBAR
  GAMMA BETA D  BETA L   VERT    HORZ    E OR O  PRESS   f'c   GRADE   GRADE
  1.30   1.00    1.67    1.00    1.00     120.    35.0  3000.  3000.     40.

  REBAR                                                 FILL
   OR      P    W                   LIVE               HEIGHT    NO.
   WIRE   OR   OR                   LOAD    AXIAL      ADJ.     SPEC.
   DIA.    C    B   SPECS   ALPHA   SURCH.  FORCE     FACTOR     LL    OUTPUT
  1.410    C    B     3      45.     0.00     Y      0.0000      0       2

                                  CULVERT DATA

  CLEAR  CLEAR   SLAB THICKNESS  WALL THICKNESS    HEIGHT     %
  SPAN   HEIGHT    TOP  BOTTOM  LEFT   INT  RIGHT   OF FILL  GRADE
  20.00   8.50    23.00  24.00 21.00  0.00 21.00    12.9     0.00

                 BAR COVERS                                 PRECAST
     TOP SLAB       BOTTOM SLAB                OVERLAY    SEGMENT
  TOP BAR  BOT BAR  TOP BAR  BOT BAR   WALLS   THICKNESS    LENGTH
   2.000    2.000    2.500    2.500    2.000      0.00        0.00

                                  HAUNCH DIMENSIONS

                 BOTTOM                                     TOP
     LEFT       INTERIOR       RIGHT         LEFT       INTERIOR       RIGHT
    X     Y      X     Y      X     Y      X     Y      X     Y      X     Y
   6.00  6.00   0.00  0.00   6.00  6.00   6.00  6.00   0.00  0.00   6.00  6.00

                                  WALL REINFORCEMENT

                 WALL 1                                     WALL 2
      BOTTOM                 TOP                 BOTTOM                TOP
   AS    SIZE SPAC      AS    SIZE SPAC      AS    SIZE SPAC      AS    SIZE SPAC
  2.060   0   0.0     2.060   0   0.0     2.060   0   0.0     2.060   0   0.0

                                  SLAB REINFORCEMENT

SLAB     AT LEFT END OF SPAN          AT MID SPAN       AT RIGHT END OF SPAN
 NO   AS   SIZE SPAC   AV   SIZE SPAC   AS   SIZE SPAC  AS   SIZE SPAC  AV   SIZE SPAC
 1  2.060  0    0.0 0.000  0    0.0  2.830  0    0.0 2.060  0    0.0 0.000  0    0.0
```

```
 2  2.060  0   0.0 0.000  0   0.0  2.830  0   0.0 2.060  0   0.0 0.000  0   0.0


              ++++++++++++++++++++++++++++++++++++++++++++
              +                                          +
              +        R A T I N G   S U M M A R Y       +
              +                                          +
              ++++++++++++++++++++++++++++++++++++++++++++

           MINIMUM INVENTORY RATING          MINIMUM OPERATING RATING
LOAD    FACTOR   TONS  LOCATION   DIST     FACTOR   TONS  LOCATION   DIST
 H20      7.97 S 159.5   SLAB 2    1.69     13.31 S 266.3   SLAB 2    1.69
HS20      5.69 S 204.9   SLAB 2    1.69      9.50 S 342.1   SLAB 2    1.69
ML80      4.66 S 170.9   SLAB 2    1.69      7.79 S 285.3   SLAB 2    1.69
P-82      3.10 M 316.0   WALL 1    8.00      5.17 M 527.7   WALL 1    8.00
TK527     4.94 S 197.7   SLAB 2    1.69      8.25 S 330.1   SLAB 2    1.69


RATING FACTOR CODES
M - MAXIMUM MOMENT STRENGTH GOVERNS
S - MAXIMUM SHEAR STRENGTH GOVERNS

BOX5 V5.9.0.1 PROGRAM WAS EXECUTED COMPLETELY AND SUCCESSFULLY.
```

## 3.13 TIMBER BRIDGES

This Section covers the rating of timber bridges, including beams and deck comprised of wood materials. See Section 2.7 for additional guidance.

### 3.13.1 Policies and Guidelines

Most timber bridges can be rated by analytical methods based on design plans and field measurements. Like steel bridges, timber bridges can be rated using field measurements.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.3 for guidance on timber.

### 3.13.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2.

#### 3.13.2.1 LFR or ASR Method

LFR method is not applicable to timber bridges (per MBE Article C6B.5.3), therefore ASR method should be used if applicable. Timber bridges built prior to 2011 can be rated with an approved analysis program (BAR7 or other) with supplemental capacity and rating calculations, AASHTOWare Bridge Rating (BrR) or other approved software.

#### 3.13.2.2 LRFR Method

Timber bridges were built after 2011 should be rated using LRFR method with an approved analysis program (BAR7 or other) with supplemental capacity and rating calculations, AASHTOWare Bridge Rating (BrR) or other approved software.

### 3.13.3 Live Load and Dead Load Distribution

#### 3.13.3.1 LFR or ASR Method

Pub. 238 Section 3.3 provides guidance on the distribution of live load to longitudinal girders. Typically, the live load distribution factor is in accordance with the AASHTO Standard Specification.

The typical distribution factors are as follows when AASHTO distribution factors are applicable.

| Exterior | | Interior | |
|---|---|---|---|
| Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.3.1.2) | Shear (AASHTO 3.23.1.2) | Moment (AASHTO 3.23.2.2, Table 3.23.1) |
| Lever Rule* | Lever Rule* | Lever Rule* | Varies depending on deck type (See AASHTO Table 3.23.1) |

Note: Timber dimensions shown are for nominal thickness.

*Per Pub 238, IP 3.3 and 3.4, a reduction in load intensity as dictated in AASHTO Std. Spec. 3.12 shall be permitted. The reduction shall not be applied when using factors from Table 3.23.1.

A skew correction factor shall be applied to the live load shear distribution factor in accordance with Table 3.23.2(A) of the 1993 DM-4. See Section 2.2.3 and Pub. 238 IP 3.3.3.1 for additional discussion of skew.

For decks on timber stringers, dead loads placed with the deck and beams (DL1) are to be based on tributary width. In the case of concrete decks on timber stringers, dead loads placed after the concrete slab has cured (DL2) shall be distributed equally among all girders (AASHTO Standard Specification 3.23.2.3.1.1).

3.13.3.2 LRFR Method

For timber bridges designed with LRFD, distribution factors may be computed in AASHTOWare BrR. For analysis outside of the program, distribution factors should be computed as per AASHTO/PennDOT LRFD Section 4.6.2.2.

### 3.13.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1931-1940 Volume 2 | Timber Bridges (20' Roadway, Spans 8-20 Feet) | 4/10/1931 | S-711 |
| BLC-560M Series, July 1998 Edition | Various | 7/31/1998 | All |
| BLC-560M Series, July 2001 Edition | Various | 7/11/2001 | All |

### 3.13.5 Modeling Section Properties and Deterioration

See Section 2.7.1 for a discussion of modeling section loss.

### 3.13.6 Standard Practices

While AASHTOWare BrR can perform a complete analysis and rating of timber bridges, BAR7 or other approved software may be used to analyze the bridge to get loads. To use BAR7, section properties of the timber beams will need to be calculated and a steel section of similar weight will need to be selected for the run. The unfactored loads can then be taken and used with the calculated flexural and shear capacities of the beams to determine the bridge rating.

Timber decks, which exhibit excessive deflection under normal traffic, must also have load rating computations performed independent from the beams (AASHTO MBE 6.1.5.1). This is also similar for timber decks, which exhibit excessive deflection under normal traffic, on steel stringers.

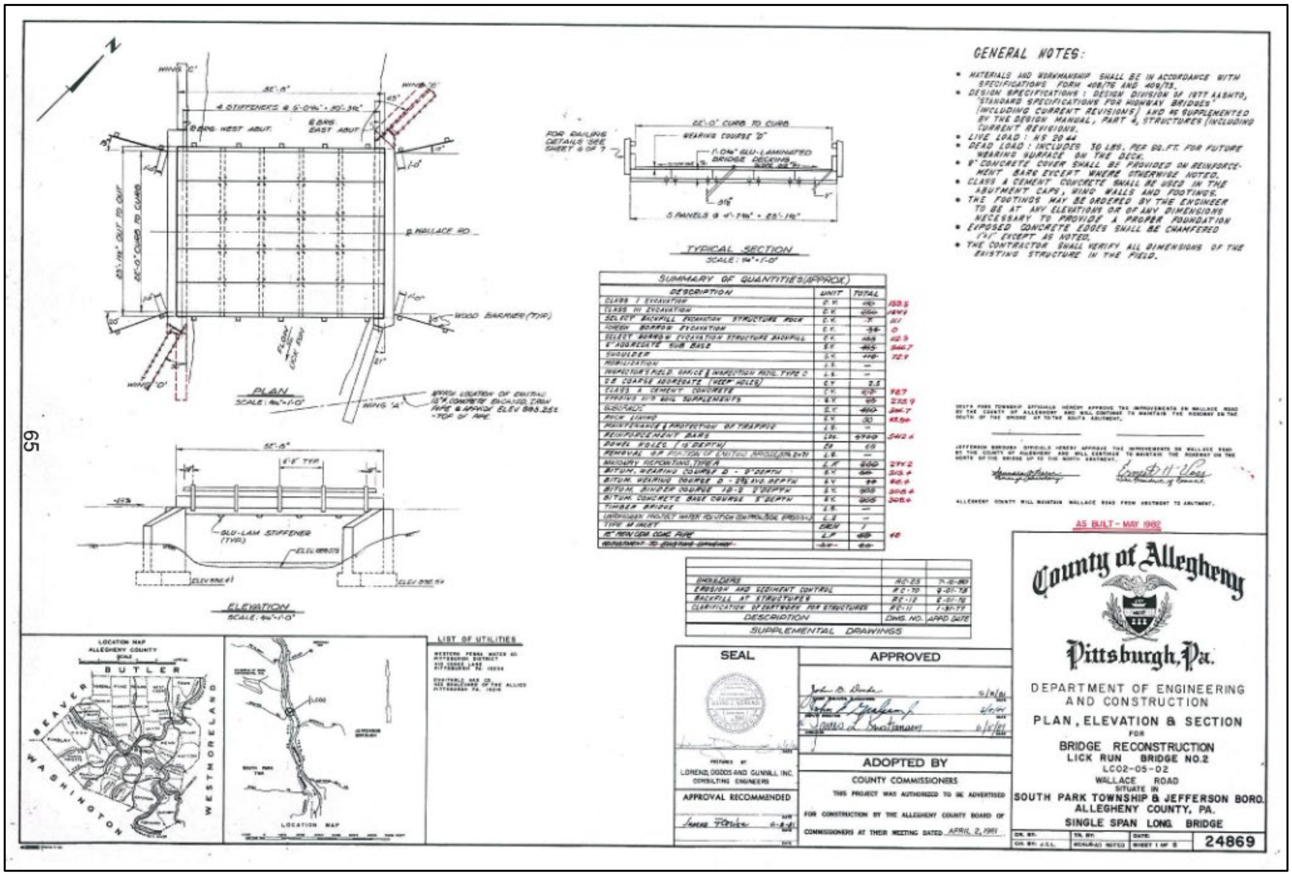
### 3.13.7 Common QA Findings

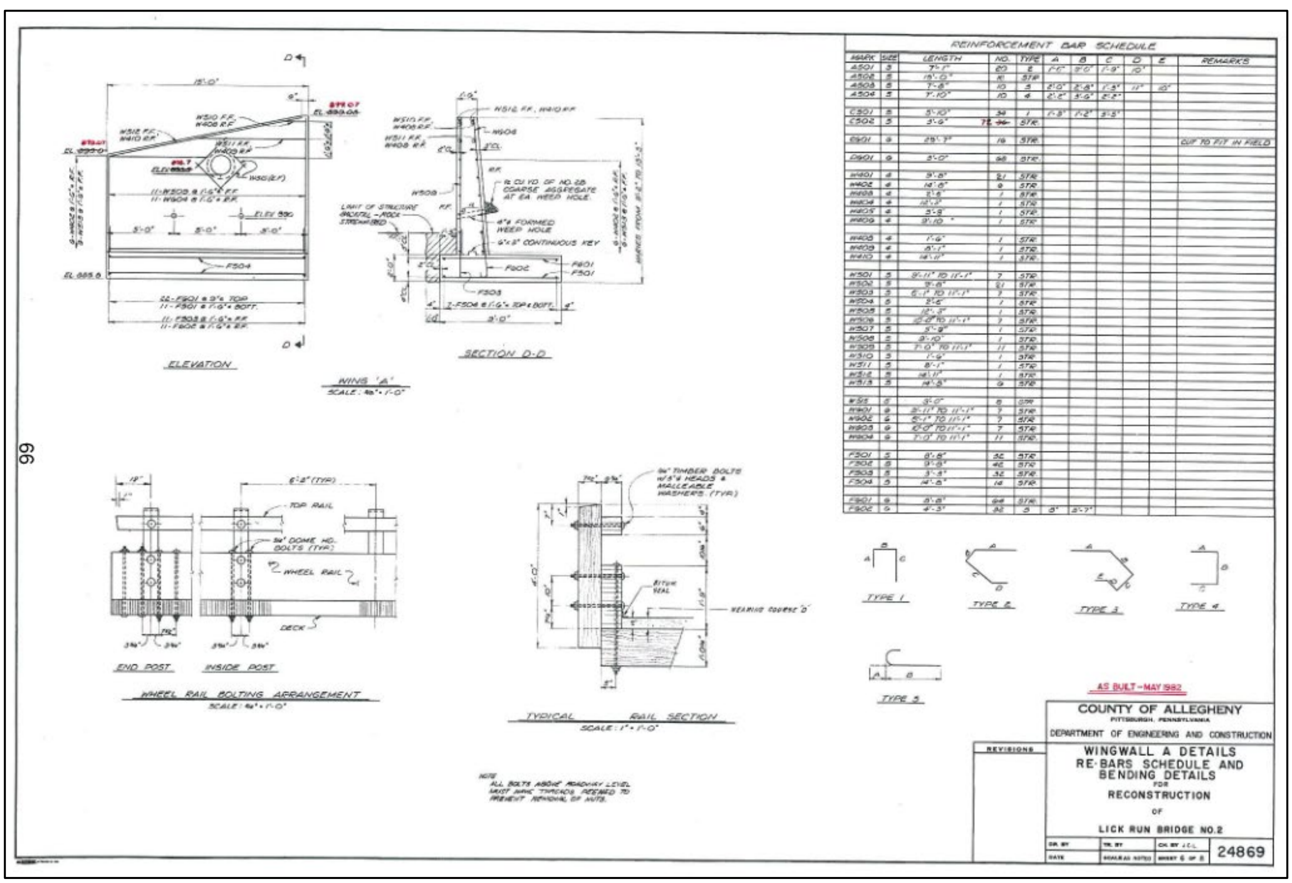
This Section is in development and will be provided in future editions of this manual.

### 3.13.8 Sample Load Rating

This Section contains sample load rating calculations for a Glu-Lam Timber Bridge Deck shown below. Hand calculations will be performed based on the Allowable Stress Design Method (ASD) in accordance with AASHTO Standard Specifications for Highway Bridges and PennDOT's 1993 Design Manual, Part IV (DM-4).

The glued laminated longitudinal panels have a span length of 31’-0’ with a skew of 90 degrees. The 4’-7 3/8” wide panels support a clear roadway width of approximately 22.3 ft.



3.13.8.1 Load Rating Summary Form

LOAD RATING SUMMARY FORM

Done By: Date:

Checked By: Date:

Structure ID (5A01): Inspection Date (7A01): 8/8/2021

Facility Carried (5A08): Wallace Road

Feature Intersected (5A07): Wallace Road Over Lick Run

Structure Type (6A26 - 6A29): Single Span Glued-Laminated Longitudinal Panel Timber Bridge

Spans / Members Analyzed: Longitudinal Panels

Analysis Method: ASD

PennDOT Program / Version: NA- Hand Calculated Ratings

| Vehicle | Inventory (IR) | | Operating (OR) | | SLC | | Controlling Member/Span | | Load Effect (Moment/Shear) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Factor | Tons | Factor | Tons | Factor | Tons | IR | OR | IR | OR |
| H20 | 0.62 | 12.5 | 0.97 | 19.4 | 0.97 | 19.4 | * | * | M | M |
| HS20 | 0.57 | 20.5 | 0.89 | 32.0 | 0.89 | 32.0 | * | * | M | M |
| ML80 | 0.39 | 14.3 | 0.61 | 22.2 | 0.61 | 22.2 | * | * | M | M |
| TK527 | 0.43 | 17.1 | 0.67 | 26.6 | 0.67 | 26.6 | * | * | M | M |
| PHL-93 | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| EV2 | 0.60 | 17.2 | 0.93 | 26.7 | 0.93 | 26.7 | * | * | M | M |
| EV3 | 0.37 | 16.0 | 0.59 | 25.4 | 0.59 | 25.4 | * | * | M | M |

Comments/Assumptions*:

Superstructure and substructure condition ratings are 6 and 5, repsectively. Therefore, the SLC Factor equals 1.0 per PennDOT Publication 238, Table IP 4.3.2-1. Per Pub 238 IP 3.2.2.5, EV2 and EV3 vehicles are included in the rating for compliance with FHWA's Memo HIBS-1.

* Controlling Member is the 1st Interior Panel at 15.5 ft (midspan).

Ratings calculated by hand and do not include any section loss or deterioration.

*Identify the amount of section loss and section remaining analyzed, for the member and location that controls, wearing surface thickness used in analysis, and other significant information. These comments should also be recorded in BMS2 item IR19.

3.13.8.2 Glued Laminated Longitudinal Panel Load Rating Analysis

3.13.8.2.1 Rating Using Hand Calculations

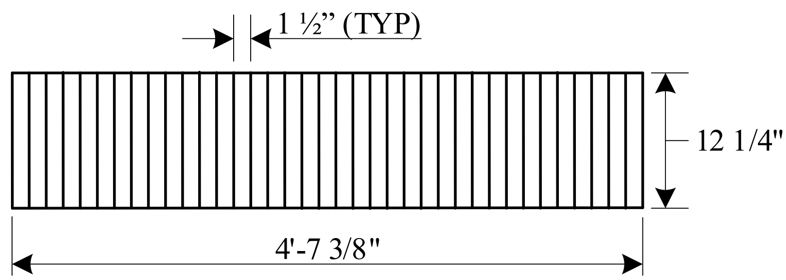
The longitudinal panels will be rated using hand calculations.

Bridge Geometry

Span Length, L = 31.00 ft based on field inspection

Width of Panel, $W_p$ = 4'- 7 3/8" = 4.6146 ft per Contract Drawings. Per AASHTO 3.25.3.1, the width of the panel should be 3.5 ft ≤ $W_p$ ≤ 4.5 ft. Since 4.6146 ft > 4.5 ft, Call ok.

Panel Depth, d = 1'-0 ¼" = 1.021 ft




CROSS SECTION

Moment Distribution Factors

Moment distribution factors for longitudinal glued laminated timber decks are calculated in accordance with AASHTO 3.25.3.1.

One Traffic Lane:

For one traffic lane, $DF_{M1}$ = larger of $W_p$/(4.25 + L/28) and $W_p$/5.50

$W_p$/(4.25 + L/28) = 4.6146 ft / (4.25 + 31 ft/28) = 0.861 wheels / (2 wheels/axle) = 0.431 axles

$W_p$/5.50 = 4.6146 ft / 5.50 = 0.839 wheels / (2 wheels/axle) = 0.420 axles

Two or More Traffic Lanes:

For two or more traffic lanes, $DF_{M2+}$ = larger of $W_p$/(3.75 + L/28) and $W_p$/5.00

$W_p$/(3.75 + L/28) = 4.6146 ft / (3.75 + 31 ft/28) = 0.950 wheels / (2 wheels/axle) = 0.475 axles

$W_p$/5.00 = 4.6146 ft / 5.00 = 0.923 wheels / (2 wheels/axle) = 0.461 axles

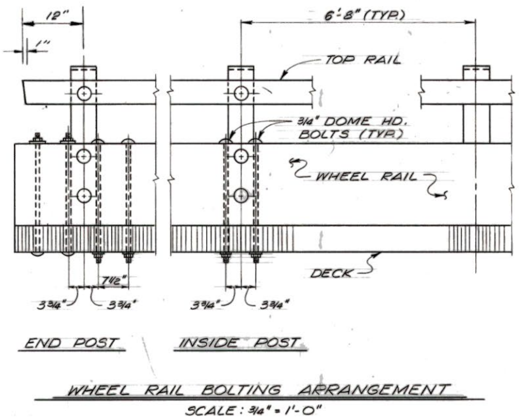
$DF_M$ = 0.475 axles (2 lanes controls)

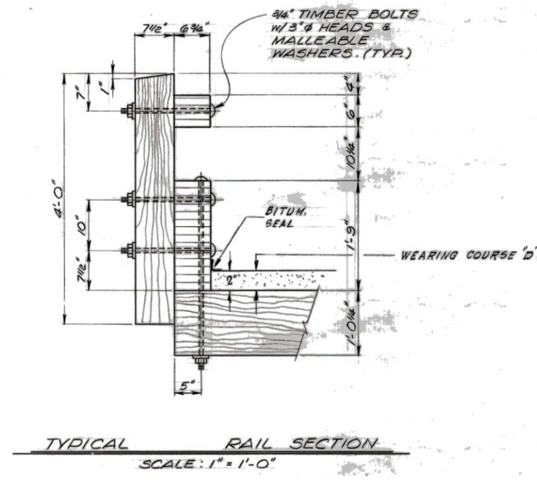
Shear Distribution Factors

Shear distribution factors for longitudinal glued laminated timber decks are calculated in accordance with AASHTO 3.25.3.2.

$DF_V$ = $W_p$/4.00 ≥ 1 = 4.6146 ft / 4.00 = 1.154 wheels / (2 wheels/axle) = 0.577 axles

Dead Loads




Unit Weight of Timber, $\gamma$ = 50 pcf = 0.050 kcf per AASHTO 3.3.6
Beam Self-Weight = 4.6146 ft x 1.021 ft x 0.050 kcf = 0.236 kips/ft

| | |
|---|---|
| Timber Railing Posts = 10 x 7.5 in x 7.5 in / (144 in$^2$/ft$^2$) x 4.0 ft x 0.050 kcf / 31 ft = | 0.025 kips/ft |
| Top Rails = 2 x 6.75 in x 6 in / (144 in$^2$/ft$^2$) x 0.050 kcf = | 0.028 kips/ft |
| Wheel Rails = 2 x 1.75 ft x 6.75 in / (12 in/ft) x 0.050 kcf = | 0.098 kips/ft |
| Transverse Stiffener = 4 x 8.5 in x 7.5 in / (144 in$^2$/ft$^2$) x 23.125 ft x 0.050 kcf / 31 ft = | 0.066 kips/ft |
| Total = | 0.217 kips/ft |

Distribute bridge component loads to exterior and first interior panels.
0.217 kips/ft / 4 beams = 0.054 kips/ft

Unit Weight of Macadam = 140 pcf per AASHTO 3.3.6
Asphalt Wearing Surface = (2 in + 3.5 in )/ 2 / (12 in/ft) x 22.0 ft x 0.140 kcf / 5 beams = 0.141 kips/ft

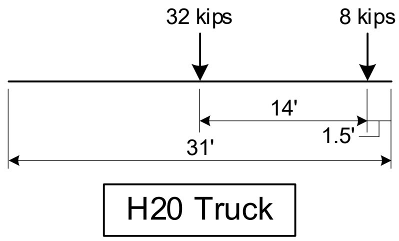
Total Uniform Dead Load, w = 0.236 kips/ft + 0.054 kips/ft + 0.141 kips/ft = 0.431 kips/ft

Dead Load Moment
Dead Load Moment at Midspan, $M_{DL} = wL^2/8$ = 0.431 kips/ft x (31 ft)$^2$/8 = 51.77 kip-ft

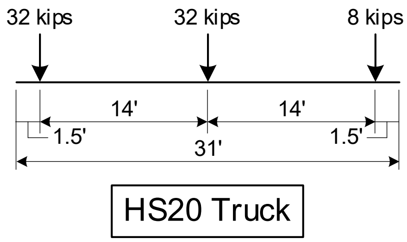
Live Load Moment
For simple span structures, the live load moment can be calculated using a line girder analysis program, such as PennDOT's CBA program, or by hand. For this load rating, the live load moment will be calculated by hand. Although the maximum live load moment does not occur at midspan, the magnitude of difference is small. In lieu of determining the reactions and drawing shear and moment diagrams, an easy formula can be used to calculate the undistributed midspan moments for point loads acting on a simple span.

$M_{LLU} = \Sigma(Pa)/2$ where distance "a" equals the distance from the point load to the nearest support.

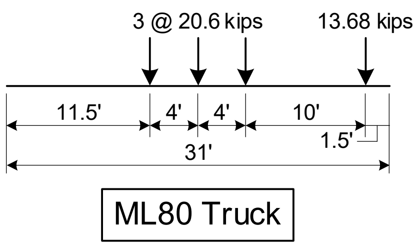
$M_{LL} = M_{LLU} \times DF_M \times IM$ where IM = 1.0 per AASHTO Section 13.1.3.



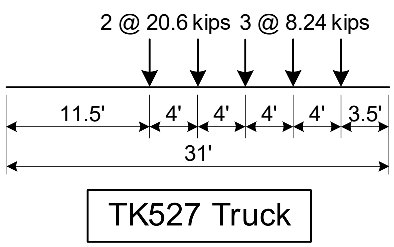

$M_{H20U}$ = (8 kips x 1.5 ft + 32 kips x 15.5 ft) / 2 = 254.00 kip-ft without DF and IM
$M_{H20}$ = 254.00 kip-ft x 0.475 x 1.0 = 120.65 kip-ft



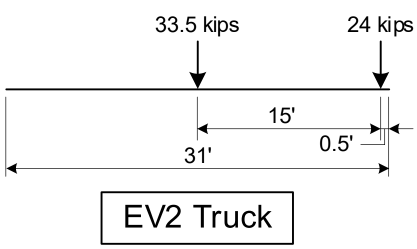

$M_{HS20U}$ = (8 kips x 1.5 ft + 32 kips x 15.5 ft + 32 kips x 1.5 ft) / 2 = 278.00 kip-ft without DF and IM
$M_{HS20}$ = 278.00 kip-ft x 0.475 x 1.0 = 132.05 kip-ft



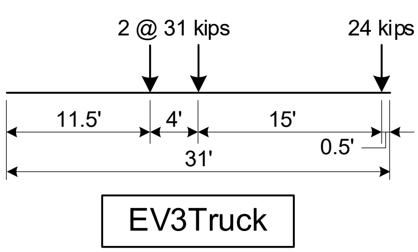

$M_{ML80U}$ = (13.68 kips x 1.5 ft + 20.6 kips x 11.5 ft + 20.6 kips x 15.5 ft + 20.6 kips x 11.5 ft) / 2 = 406.81 kip-ft w/out DF and IM
$M_{ML80}$ = 406.81 kip-ft x 0.475 x 1.0 = 193.23 kip-ft




$M_{TK527U}$ = (8.24 kips x 3.5 ft + 8.24 kips x 7.5 ft + 8.24 kips x 11.5 ft + 20.6 kips x 15.5 ft + 20.6 kips x 11.5 ft) / 2 = 370.80 kip-ft without DF and IM
$M_{TK527}$ = 370.80 kip-ft x 0.475 x 1.0 = 176.13 kip-ft




$M_{EV2U}$ = (24 kips x 0.5 ft + 33.5 kips x 15.5 ft) / 2 = 265.625 kip-ft without DF and IM
$M_{EV2}$ = 265.625 kip-ft x 0.475 x 1.0 = 126.17 kip-ft




$M_{EV3U}$ = (24 kips x 0.5 ft + 31 kips x 15.5 ft + 31 kips x 11.5 ft) / 2 = 424.50 kip-ft without DF and IM
$M_{EV3}$ = 424.50 kip-ft x 0.475 x 1.0 = 201.64 kip-ft

Allowable Bending Stress

The allowable bending stress, $F_b$', is calculated in accordance with AASHTO Section 13.6.4.1. Use Visually Graded Western Species "20F-V2" with Bending About Y-Y Axis (Loaded Parallel to Wide Faces of Laminations) in AASHTO Table 13.5.3A.

$F_b' = F_bC_MC_DC_FC_VC_LC_fC_{fu}C_r$ per AASHTO Equation 13-2 where:

Tabulated Unit Stress in Bending, $F_b = F_{by}$ =1200 psi per AASHTO Table 13.5.3A
Wet Service Factor, $C_M$ = 0.8 per AASHTO 13.5.5.1 and Table 13.5.3A, Footnote 14
Load Duration Factor, $C_D$ = 1.15 per AASHTO 13.5.5.2 and Table 13.5.5A for live load
Bending Size Factor, $C_F = (12/d)^{1/9} = 0.9977$ per AASHTO 13.6.4.2.2 Equation 13-3 (glued-laminated timber loaded parallel to wide face where d = 12.25 in)
Volume Factor, $C_V = (21/L)^{1/x}\,(12/d)^{1/x}(5.125/b)^{1/x} \leq 1.0$ per AASHTO 13.6.4.3.1 where:
Length of bending member between points of zero moment in feet, L = 31 ft,
Depth of bending member, d = 12.25 in,
Width of bending member, b = 1.5 in, and
X = 10 for other species (not Southern Pine)
$C_V$ = 1.085, Use $C_V$ = 1.0.
Beam Stability Factor, $C_L$ = 1.0 per AASHTO 13.6.4.4.2
Factors $C_f$, $C_{fu}$, and $C_r$ are not applicable to Glu-Laminated. Therefore, $C_f = C_{fu} = C_r = 1.0$.

$F_b$' = 1200 psi x 0.8 x 1.15 x 0.9977 x 1.0 x 1.0 x 1.0 x 1.0 x 1.0 = 1101 psi = 1.101 ksi

Bending Stresses

Section Modulus, $S = bd^2/6$ = 55.375 in x (12.25 in)$^2$ / 6 = 1385 in$^3$
Bending Stress, $f_b$ = M/S and is calculated at midspan.

$f_{bDL}$ = 51.77 kip-ft x (12 in/ft) / 1385 in$^3$ = 0.449 ksi
$f_{bH20}$ = 120.65 kip-ft x (12 in/ft) / 1385 in$^3$ = 1.045 ksi
$f_{bHS20}$ = 132.05 kip-ft x (12 in/ft) / 1385 in$^3$ = 1.144 ksi
$f_{bML80}$ = 193.23 kip-ft x (12 in/ft) / 1385 in$^3$ = 1.674 ksi
$f_{bTK527}$ = 176.13 kip-ft x (12 in/ft) / 1385 in$^3$ = 1.526 ksi
$f_{bEV2}$ = 126.17 kip-ft x (12 in/ft) / 1385 in$^3$ = 1.093 ksi
$f_{bEV3}$ = 201.64 kip-ft x (12 in/ft) / 1385 in$^3$ = 1.747 ksi

Inventory Ratings for Moment

Ratings are calculated using the Allowable Stress Design Method.
Inventory Moment Ratings, IR = (Allowable Stress – Dead Load Stress) / Live Load Stress
IR = ($F'_b - f_{bDL}$) / $f_{bLL}$

$IR_{H20}$ = (1.101 ksi – 0.449 ksi) / 1.045 ksi = 0.624 x 20 Tons = 12.5 Tons
$IR_{HS20}$ = (1.101 ksi – 0.449 ksi) / 1.144 ksi = 0.570 x 36 Tons = 20.5 Tons
$IR_{ML80}$ = (1.101 ksi – 0.449 ksi) / 1.674 ksi = 0.389 x 36.64 Tons = 14.3 Tons
$IR_{TK527}$ = (1.101 ksi – 0.449 ksi) / 1.526 ksi = 0.427 x 40 Tons = 17.1 Tons
$IR_{EV2}$ = (1.101 ksi – 0.449 ksi) / 1.093 ksi = 0.597 x 28.75 Tons = 17.2 Tons
$IR_{EV3}$ = (1.101 ksi – 0.449 ksi) / 1.747 ksi = 0.373 x 43 Tons = 16.0 Tons

Operating Ratings for Moment
Ratings are calculated using the Allowable Stress Design Method.
Operating Moment Ratings, OR = (1.33 x Allowable Stress – Dead Load Stress) / Live Load Stress
$OR = (1.33 \times F'_b - f_{bDL}) / f_{bLL}$

$OR_{H20}$ = (1.33 x 1.101 ksi – 0.449 ksi) / 1.045 ksi = 0.972 x 20 Tons = 19.4 Tons
$OR_{HS20}$ = (1.33 x 1.101 ksi – 0.449 ksi) / 1.144 ksi = 0.888 x 36 Tons = 32.0 Tons
$OR_{ML80}$ = (1.33 x 1.101 ksi – 0.449 ksi) / 1.674 ksi = 0.607 x 36.64 Tons = 22.2 Tons
$OR_{TK527}$ = (1.33 x 1.101 ksi – 0.449 ksi) / 1.526 ksi = 0.665 x 40 Tons = 26.6 Tons
$OR_{EV2}$ = (1.33 x 1.101 ksi – 0.449 ksi) / 1.093 ksi = 0.929 x 28.75 Tons = 26.7 Tons
$OR_{EV3}$ = (1.33 x 1.101 ksi – 0.449 ksi) / 1.747 ksi = 0.581 x 43 Tons = 25.0 Tons

Dead Load Shear
Per AASHTO Section 13.6.5.2, for uniformly distributed loads, the magnitude of the vertical shear used in AASHTO Equation 13-9 shall be the maximum shear calculated at a "d" distance from the support, where the member depth, d = 1.021 ft. The shear at any location along a simply supported beam subjected to a uniformly distributed load can be calculated using the formula, V = w(L/2 – x) where x is equal to the member depth, d.

Dead Load Shear, $V_{DL}$ = w(L/2 – x) = 0.431 kips/ft x (31 ft/2 – 1.021 ft) = 6.24 kips

Live Load Shear
Per AASHTO Section 13.6.5.2, the maximum vertical shear is calculated at a distance from the support equal to the smaller of 3d and L/4 where d is the member depth and L is the span length.

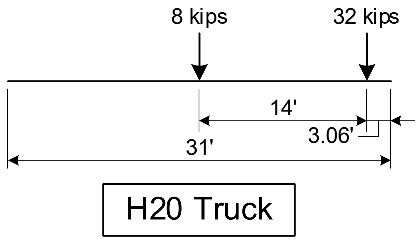
Critical Shear Location = Min(3d, L/4) = Min(3 x 1.021 ft = 3.0625 ft, 31 ft / 4 = 7.75 ft)
Critical Shear Location = 3.0625 ft from the support

For simple span structures, the live load shear can be calculated using a line girder analysis program, such as PennDOT's CBA program, or by hand. For this load rating, the live load shear will be calculated by hand. To determine the maximum shear force at the critical location for simple spans, the first axle is placed at the critical shear location. The support reaction is then calculated using the lever rule. The shear at the critical section will be equal to the reaction since there aren't any loads applied between the support and critical section.

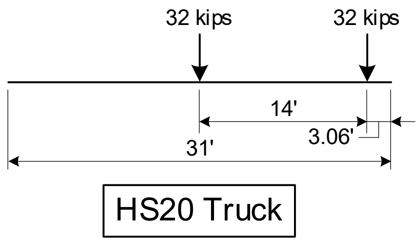
Distributed Live Load Vertical Shear, $V_{LL} = 0.50 (0.60 \times V_{LU} + V_{LD})$ per AASHTO Eq. 13-10 where:

$V_{LU}$ = Maximum vertical shear at the critical location for shear due to undistributed wheel loads. Per AASHTO 13.6.5.2, for undistributed wheel loads, one line of wheels is assumed to be carried by one bending member.

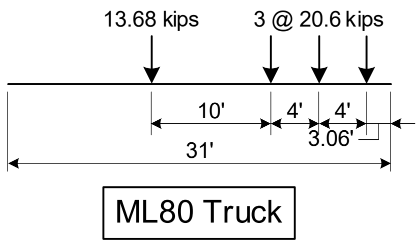
$V_{LD} = V_{LU} \times DF_V$ = Maximum vertical shear at the critical location for shear due to wheel loads distributed laterally for moment per AASHTO Section 3.23.



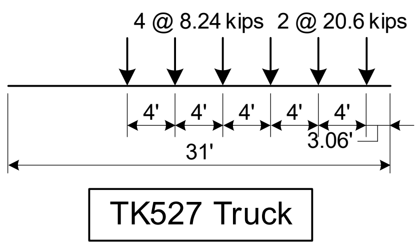

$V_{LUH20}$ = (32 kips x 27.94 ft + 8 kips x 13.94 ft) / 31 ft = 16.22 kips without DF and IM
$V_{LDH20}$ = 0.577 x 16.22 kips = 18.72 kips
$V_{LLH20}$ = 0.50 (0.60 x 16.22 kips + 18.72 kips) = 14.22 kips



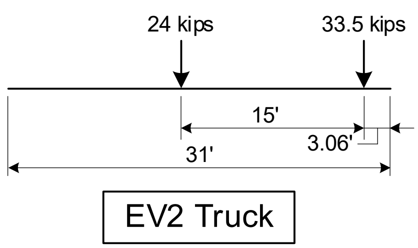

$V_{LUHS20}$ = (32 kips x (27.94 ft + 13.94 ft)) / 31 ft = 21.62 kips without DF and IM
$V_{LDHS20}$ = 0.577 x 21.62 kips = 24.94 kips
$V_{LLHS20}$ = 0.50 (0.60 x 21.62 kips + 24.94 kips) = 18.96 kips



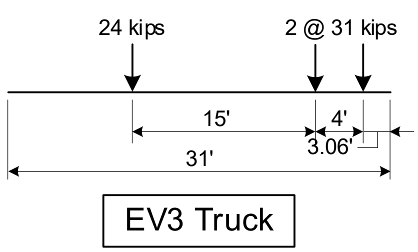

$V_{LUML80}$ = (20.6 kips x (27.94 ft + 23.94 ft + 19.94 ft) + 13.68 kips x 9.94 ft) / 31 ft = 26.06 kips without DF and IM
$V_{LDML80}$ = 0.577 x 26.06 kips = 30.07 kips
$V_{LLML80}$ = 0.50 (0.60 x 26.06 kips + 30.07 kips) = 22.85 kips




$V_{LUTK527}$ = (20.6 kips x (27.94 ft + 23.94 ft) + 8.24 kips x (19.94 ft + 15.94 ft + 11.94 ft + 7.94 ft)) / 31 ft = 24.65 kips without DF and IM
$V_{LDTK527}$ = 0.577 x 24.65 kips = 28.44 kips
$V_{LLTK527}$ = 0.50 (0.60 x 24.65 kips + 28.44 kips) = 21.62 kips




$V_{LUEV2}$ = (33.5 kips x 27.94 ft + 24 kips x 12.94 ft) / 31 ft = 20.10 kips without DF and IM
$V_{LDEV2}$ = 0.577 x 20.10 kips = 23.20 kips
$V_{LLEV2}$ = 0.50 (0.60 x 20.10 kips + 23.20 kips) = 17.63 kips




$V_{LUEV3}$ = (31.0 kips x (27.94 ft + 23.94 ft) + 24 kips x 8.94 ft) / 31 ft = 29.40 kips without DF and IM
$V_{LDEV3}$ = 0.577 x 29.40 kips = 33.93 kips
$V_{LLEV3}$ = 0.50 (0.60 x 29.40 kips + 33.93 kips) = 25.78 kips

<u>Allowable Shear Stress</u>

The allowable shear stress parallel to the grain, $F_v$', is calculated in accordance with AASHTO Section 13.6.5.3. Use Visually Graded Western Species with Bending About Y-Y Axis (Loaded Parallel to Wide Faces of Laminations) in AASHTO Table 13.5.3A.

$F_v' = F_v C_M C_D$ per AASHTO Equation 13-9 where:

Tabulated Unit Stress in Shear Parallel to the Grain, $F_v = F_{vy}$ =135 psi per AASHTO Table 13.5.3A
Wet Service Factor, $C_M$ = 0.875 per AASHTO 13.5.5.1 and Table 13.5.3A, Footnote 14
Load Duration Factor, $C_D$ = 1.15 per AASHTO 13.5.5.2 and Table 13.5.5A for live load

$F_v'$ = 135 psi x 0.875 x 1.15 = 135.8 psi = 0.136 ksi

Shear Stresses
Horizontal Shear Stress, $f_v$ = 3V/(2bd) per AASHTO Equation 13-9 with the critical location for shear at 3.0625 ft from the support.

$f_{vDL}$ = 3 x 6.24 kips / (2 x 55.375 in x 12.25 in) = 0.014 ksi
$f_{vH20}$ = 3 x 14.22 kips / (2 x 55.375 in x 12.25 in) = 0.031 ksi
$f_{vHS20}$ = 3 x 18.96 kips / (2 x 55.375 in x 12.25 in) = 0.042 ksi
$f_{vML80}$ = 3 x 22.85 kips / (2 x 55.375 in x 12.25 in) = 0.051 ksi
$f_{vTK527}$ = 3 x 21.62 kips / (2 x 55.375 in x 12.25 in) = 0.048 ksi
$f_{vEV2}$ = 3 x 17.63 kips / (2 x 55.375 in x 12.25 in) = 0.039 ksi
$f_{vEV3}$ = 3 x 25.78 kips / (2 x 55.375 in x 12.25 in) = 0.057 ksi

Inventory Ratings for Shear
Ratings are calculated using the Allowable Stress Design Method.
Inventory Shear Ratings, IR = (Allowable Stress – Dead Load Stress) / Live Load Stress
IR = ($F_v'$ – $f_{vDL}$) / $f_{vLL}$

$IR_{H20}$ = (0.136 ksi – 0.014 ksi) / 0.031 ksi = 3.935 x 20 Tons = 78.7 Tons
$IR_{HS20}$ = (0.136 ksi – 0.014 ksi) / 0.042 ksi = 2.935 x 36 Tons = 105.6 Tons
$IR_{ML80}$ = (0.136 ksi – 0.014 ksi) / 0.051 ksi = 2.392 x 36.64 Tons = 87.6 Tons
$IR_{TK527}$ = (0.136 ksi – 0.014 ksi) / 0.049 ksi = 2.490 x 40 Tons = 99.6 Tons
$IR_{EV2}$ = (0.136 ksi – 0.014 ksi) / 0.039 ksi = 3.128 x 28.75 Tons = 89.9 Tons
$IR_{EV3}$ = (0.136 ksi – 0.014 ksi) / 0.057 ksi = 2.140 x 43 Tons = 92.0 Tons

Operating Ratings for Shear
Ratings are calculated using the Allowable Stress Design Method.
Operating Shear Ratings, OR = (1.33 x Allowable Stress – Dead Load Stress) / Live Load Stress
OR = (1.33 x $F'_v$ – $f_{vDL}$) / $f_{vLL}$

$OR_{H20}$ = (1.33 x 0.136 ksi – 0.014 ksi) / 0.031 ksi = 5.383 x 20 Tons = 107.6 Tons
$OR_{HS20}$ = (1.33 x 0.136 ksi – 0.014 ksi) / 0.042 ksi = 3.973 x 36 Tons = 143.0 Tons
$OR_{ML80}$ = (1.33 x 0.136 ksi – 0.014 ksi) / 0.051 ksi = 3.272 x 36.64 Tons = 119.8 Tons
$OR_{TK527}$ = (1.33 x 0.136 ksi – 0.014 ksi) / 0.049 ksi = 3.406 x 40 Tons = 136.2 Tons
$OR_{EV2}$ = (1.33 x 0.136 ksi – 0.014 ksi) / 0.039 ksi = 4.279 x 28.75 Tons = 123.0 Tons
$OR_{EV3}$ = (1.33 x 0.136 ksi – 0.014 ksi) / 0.057 ksi = 2.928 x 43 Tons = 125.9 Tons

## 3.14 REINFORCED CONCRETE ARCH CULVERTS

This Section covers the rating of reinforced concrete arch culverts.

### 3.14.1 Policies and Guidelines

Most reinforced concrete arch bridges can be rated by analytical methods based on design plans and field measurements. If design plans are not available, PennDOT standard plans can be utilized if field measured values are in reasonable agreement with one of the standard plans.

If plans are not available and field measurements are not in reasonable agreement with PennDOT standard plans, engineering judgment can be utilized. A new engineering judgement procedure is in development for bridges not currently covered in Pub. 238 Appendix IP 03-B. Until the new Pub. 238 is released, the current procedure or sound engineering judgment should be used.

If material properties are available, see Pub. 238 IP 3.7.2 for hierarchy of choosing the properties if there is conflicting information. If material information is not available, see Pub. 238 IP 3.7.2.2 and MBE 6B5.2.4 for guidance on concrete and MBE 6B.5.2.3 for guidance on reinforcing steel.

### 3.14.2 Analysis Method and Software

For further discussion on load rating software, refer to Section 1.2. Refer to Table 1.2 for a list of the recommended PennDOT software/spreadsheets as well as Acceptable Non-PennDOT Software for load rating. The program CANDE can be downloaded at: <CANDE> (candeforculverts.com)

#### 3.14.2.1 LFR or ASR Method

For reinforced concrete arch bridges designed in LFD or ASD, the typical load rating method will be LFR. PennDOT's ARCH program, the program CANDE or other approved software should be utilized to analyze these bridges when the necessary information is available to complete an analysis. ARCH/CANDE does not perform a load rating, therefore, SpColumn or another approved program outside of ARCH/CANDE can be used to complete the rating.

#### 3.14.2.2 LRFR Method

For reinforced concrete arch bridges designed with LRFD, the typical load rating method will be LRFR. ARCH does not perform rating runs, therefore, the program CANDE may be used to determine forces and stresses in the arch. CANDE also does not perform rating computations, therefore, SpColumn or another approved program outside of CANDE can be used to complete the rating. If an analytical rating will be completed, the BIS can be consulted for guidance.

### 3.14.3 Live Load and Dead Load Distribution

Regardless of the rating method or age, the amount of fill over the structure will determine the method which the wheel loads are distributed through the fill to the top of the structure. See Section 2.5.6 for additional discussion on distribution of loads on arch culverts.

#### 3.14.3.1 LFR or ASR Method

PennDOT's ARCH program uniformly distributes live load over the length of the arch. Dead loads, in addition to self-weight, are computed in the program based on a height of fill over the crown and the unit weight of the cover.

3.14.3.2 LRFR Method

Per AASHTO LRFD Table 3.6.1.2.6a-1, the Live Load Distribution Factor for all arches is to be 1.15.

### 3.14.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| Standards for Old Bridges 1941-1960 Volume 3 | Concrete Arches | 2/1/1941 | S-705 Series, S-706 Series, S-707 Series |

### 3.14.5 Modeling Section Properties and Deterioration

See Section 2.5.1.2 for a discussion of modeling section loss.

See Section 3.4.5 for a discussion of adjusting the model for bars that have been deemed ineffective.

### 3.14.6 Standard Practices

PennDOT's ARCH program is unable to provide separate results in terms of dead load and live load reactions. If separate results are desired, two input forms should be completed, one with FILL = actual height and ULD = 0, and the second with FILL = 0 and ULD = desired load.

### 3.14.7 Common QA Findings

This Section is in development and will be provided in future editions of this manual.

### 3.14.8 Sample Load Rating

Sample load ratings are in development and will be provided in future editions of this manual.

## 3.15 STONE MASONRY ARCHES

This Section covers the rating of stone masonry arch bridges. See Section 2.8 for additional information.

### 3.15.1 Policies and Guidelines

Currently, there is not a recommended analytical software for rating stone masonry arch bridges. Stone masonry arch thickness and material properties are typically unknown; therefore, computing accurate load ratings is typically not feasible. The engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized for the time being. Future versions of Pub. 238 will also provide more specific guidance to bridge types not currently listed in Pub. 238 Appendix IP 03-B.

### 3.15.2 Analysis Method and Software

A new engineering judgement procedure is in development for bridges not currently covered in Pub. 238 Appendix IP 03-B. Until the new Pub. 238 is released, the current procedure should be used for stone masonry arch bridges, as needed.

#### 3.15.2.1 LFR or ASR Method

When appropriate information is available, stone masonry arches should be rated using the ASR method as per AASHTO MBE Article 6A.9.1. LFR method cannot be used for stone masonry arches, as per AASHTO MBE Article C6B.5.3.

#### 3.15.2.2 LRFR Method

Given that the majority of stone masonry arches were built prior to 2011 and designed using the allowable stress method, the LRFR rating method would rarely be utilized.

### 3.15.3 Live Load and Dead Load Distribution

#### 3.15.3.1 LFR or ASR Method

There is not specific guidance on the distribution of loads to stone masonry arches, therefore, the load rater must utilize engineering judgement in determining live load and dead load distribution.

#### 3.15.3.2 LRFR Method

Given that the majority of stone masonry arches were built prior to 2011 and designed using the allowable stress method, the LRFR rating method would rarely be utilized.

### 3.15.4 Resources Available

There are no standard drawings available for stone masonry arches.

### 3.15.5 Modeling Section Properties and Deterioration

See Section 2.8.1 for a discussion of modeling section loss and material properties.

### 3.15.6 Standard Practices

There are no current standard practices for rating stone masonry arches.

### 3.15.7 Common QA Findings

This Section is in development and will be provided in future editions of this manual.

### 3.15.8 Sample Load Rating

Sample load ratings are in development and will be provided in future editions of this manual.

## 3.16 METAL ARCH CULVERTS

This Section covers the rating of metal arch culverts. See Figure 2.6.6-1 for the various metal arch culvert shapes for which the provisions of this Section apply.

### 3.16.1 Policies and Guidelines

Currently, there is not a recommended analytical software for rating metal arch culverts. The engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized for the time being. Future versions of Pub. 238 will also provide more specific guidance to bridge types not currently listed in Pub. 238 Appendix IP 03-B.

### 3.16.2 Analysis Method and Software

A new engineering judgement procedure is in development for bridges not currently covered in Pub. 238 Appendix IP 03-B. Until the new Pub. 238 is released, the current procedure should be used for metal arch culverts.

#### 3.16.2.1 LFR or ASR Method

Engineering judgement may be used. See Section 2.6.6 for further discussion of other analysis methods. The program CANDE may be used to perform a 2D finite element analysis and rating using the ASR method.

#### 3.16.2.2 LRFR Method

Engineering judgement may be used. See Section 2.6.6 for further discussion of other analysis methods. The program CANDE may be used to perform a 2D finite element analysis and rating using the LRFR method.

### 3.16.3 Live Load and Dead Load Distribution

#### 3.16.3.1 LFR or ASR Method

Per AASHTO Std. Spec. 12.1.3, the provisions of AASHTO Std. Spec. 6.4 shall apply except for the phrase "When the depth of fill is 2 feet or more" in AASHTO Std. Spec. 6.4.1 need not be considered. Per AASTHO Std. Spec. 6.4.1, concentrated loads shall be considered as uniformly distributed over a square with sides equal to 1.75 times the depth of fill. Given that the depth of fill will vary along the length of the arch, cases will need to be checked with a varying distributed load to account for the varying depth. Per AASHTO Std. Spec. 6.4.2, in areas in which distributed wheel loads overlap the total load shall be uniformly distributed over the overlapping area.

Also per AASHTO Std. Spec. 6.4.2, the effect of live load may be neglected if:

- Single Spans: Fill is more than 8 ft and exceeds the span length.
- Multiple Spans: Depth of fill exceeds the distance between the faces of the end supports or abutments.

#### 3.16.3.2 LRFR Method

Per AASHTO LRFD Table 3.6.1.2.6a-1, the Live Load Distribution Factor for all culverts (except concrete pipe) and buried structures is to be 1.15.

### 3.16.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| BD-600 Series/BD-600 Series, July 1993 Edition | Design Table for Metal Culverts | 7/1/1993 | BD-635 |
| BD-600_Series, Sept. 1994 Edition | Design Table for Metal Culverts | 9/30/1994 | BD-635 |
| "BD-635 Design Tables for Metal Culverts" was deleted on 6/30/2000 | | | |
| Online document not available | Design Table for Metal Culverts | 6/30/2000 | BD-635M |
| Online document not available | Design Table for Metal Culverts | 1/21/2003 | BD-635M |
| Online document not available | Design Table for Metal Culverts | 9/29/2010 | BD-635M |
| Standards for Bridge Design April 2016 Edition Change 7 | Design Table for Metal Culverts | 4/29/2016 (Latest Revision 10/7/2024) | BD-635M |

### 3.16.5 Modeling Section Properties and Deterioration

See Section 2.6.1.2 for a discussion of modeling section loss and material properties.

### 3.16.6 Standard Practices

While it is not a load rating procedure, the computations for determining remaining life of metal arch culverts in Pub. 100A (see example in Section 3.0, IM Screen, Item IM05, Priority Code 2, Culvert Examples) may be useful in applying engineering judgement for rating these structures. A remaining life calculation spreadsheet is available on the BMS2 Homescreen.

### 3.16.7 Common QA Findings

This Section is in development and will be provided in future editions of this manual.

### 3.16.8 Sample Load Rating

Sample load ratings are in development and will be provided in future editions of this manual.

## 3.17 REINFORCED CONCRETE PIPES

This Section covers the rating of reinforced concrete pipes.

### 3.17.1 Policies and Guidelines

Currently, there is not a recommended analytical software for rating reinforced concrete pipes. The engineering judgement procedure in Pub. 238 Appendix IP 03-B can be utilized for the time being. Future versions of Pub. 238 will also provide more specific guidance to bridge types not currently listed in Pub. 238 Appendix IP 03-B.

### 3.17.2 Analysis Method and Software

A new engineering judgement procedure is in development for bridges not currently covered in Pub. 238 Appendix IP 03-B. Until the new Pub. 238 is released, the current procedure should be used for reinforced concrete pipes. DM-4 Appendix H, which provides guidance on concrete pipes, is currently under development.

#### 3.17.2.1 LFR or ASR Method

Engineering judgement may be used.

#### 3.17.2.2 LRFR Method

Engineering judgement may be used.

### 3.17.3 Live Load and Dead Load Distribution

#### 3.17.3.1 LFR or ASR Method

Per AASHTO Std. Spec. 16.4.4.1, the provisions of AASHTO Std. Spec. 6.4 shall apply except for the requirement of 2ft of minimum fill over the pipe does not apply. Per AASHTO Std. Spec. 6.4.1, concentrated loads shall be considered as uniformly distributed over a square with sides equal to 1.75 times the depth of fill. Given that the depth of fill will vary along the length of the arch, cases will need to be checked with a varying distributed load to account for the varying depth. Per AASHTO Std. Spec. 6.4.2, in areas in which distributed wheel loads overlap the total load shall be uniformly distributed over the overlapping area.

Also per AASHTO Std. Spec. 6.4.2, the effect of live load may be neglected if:

- Single Spans: Fill is more than 8 ft and exceeds the span length.
- Multiple Spans: Depth of fill exceeds the distance between the faces of the end supports or abutments.

#### 3.17.3.2 LRFR Method

Per DM-4 12.10, refer to standard drawing BD-636M for structural design criteria.

### 3.17.4 Resources Available

The following PennDOT standard drawings are available via a link on the BMS2 Homescreen.

| Reference Document | Topic | Approval Date | Relevant Drawings |
|---|---|---|---|
| BD-600 Series, Sept. 1994 Edition | Reinforced Concrete Pipes | 9/30/1994 | BD-636 |
| "BD-636 Reinforced Concrete Pipes" was deleted on 6/30/2000 | | | |
| Online document not available | Reinforced Concrete Pipes | 1/21/2003 | BD-636M |
| Online document not available | Reinforced Concrete Pipes | 9/20/2010 | BD-636M |
| Online document not available | Reinforced Concrete Pipes | 11/26/2013 | BD-636M |
| Standards for Bridge Design April 2016 Edition Change 7 | Reinforced Concrete Pipes | 4/29/2016 (Latest Revision 10/7/2024) | BD-636M |

### 3.17.5 Modeling Section Properties and Deterioration

See Section 2.5.1.2 for a discussion of modeling section loss.

### 3.17.6 Standard Practices

Given the varying depths of fill over pipes, there may be cases where a portion of the pipe requires application of live load while other portions do not. The live load only needs to be applied to the portion of the pipe where required based on the fill depth. See BD-636M for additional guidance on load application.

### 3.17.7 Common QA Findings

This Section is in development and will be provided in future editions of this manual.

### 3.17.8 Sample Load Rating

Sample load ratings are in development and will be provided in future editions of this manual.

## 3.18 DECKS

Concrete decks and metal decks that are carrying normal traffic satisfactorily do not need to be included as part of a routine load rating. Timber decks that exhibit excessive deformation under normal traffic loads can control the bridge rating and should be considered in the load rating (AASHTO MBE 6.1.5.1). The load rating engineer should use their best judgment to determine cases where the deck should be load rated. The following situations may require the deck to be rated:

- Timber deck with deflection under normal traffic load.
- Asphalt filled stay-in-place formwork showing advanced deterioration.
- Open grid decks showing advanced deterioration.
- One or more of the beams in a multi-girder system is considered in-effective for live load and the deck must span over these areas.
- Any deck type with advanced section loss that may control the load rating of the bridge.

See Section 2.4.1 for additional information.

### 3.19 SUBSTRUCTURES

See AASHTO MBE 6.1.5.2 and Section 2.9 of this manual for a discussion of load rating of substructures. Substructures typically do not need to be included as part of a routine load rating. When deemed necessary, a load rating of the substructure should be completed. VBENT may be utilized to determine forces on a pier when needed. Other approved programs may also be utilized to determine forces on a substructure as needed. The load rating engineer should use their best judgment to determine cases where the substructure should be load rated. The following situations may require the substructure to be rated:

- Any substructure which has advanced section loss, serious scour, or settlement/rotation which may limit the load carrying capacity of the bridge. Special consideration shall be given to substructures which support a NSTM member.
- Cross girders must be rated (Pub 238 IP Article 3.4.5). There ia a workaround to load rate a box shaped cross girder, which is presented in PennDOT's BAR7 Manual, Section 3.13.
- Any steel substructure which contains NSTM members.
- Any steel substructure which has bracing that has been compromised and increases the unbraced length of the column.
- For K-frame legs, refer to Section 3.9.

See Section 2.4.3 for additional information.

# APPENDIX IP 04-A

Bridge Load Posting
Recommendation Form

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pennsylvania
DEPARTMENT OF TRANSPORTATION
www.penndot.gov

# BRIDGE LOAD POSTING RECOMMENDATION FORM

| BMS ID | BRKEY | COUNTY | ROUTE SR / T- | SEG | OFFSET | DETOUR LENGTH (Miles) | MUNICIPALITY |
|---|---|---|---|---|---|---|---|
| | | | | | | | |
| Bridge Name: | | | | | | | |
| Feature Carried: | | | | | | School Bus Route: | |
| Feature Intersected: | | | | | | Public Transp Route: | |
| ADT: | ADTT%: | | NHS: | | | PUC Jurisdiction: | |

| PREVIOUS BRIDGE POSTING: | | Year last posted: |
|---|---|---|
| Weight Limit: tons | Except Combinations: tons | ☐ One Truck at a Time |

| RECOMMENDED POSTING UNDER § 4902(A) OF THE PA VEHICLE CODE: | | |
|---|---|---|
| Weight Limit: tons | Except Combinations: tons | ☐ One Truck at a Time: |
| Bridge Closed: | Closed but Open for Pedestrian Access Only: | |
| Posting Based On: | ☐ Structural Analysis AND/OR | ☐ Structural Condition |
| Controlling Member(s): | | |
| Reason (as per Pub 238, Sec IP 4.3.1.1) | | |

| STRUCTURE DATA: | | |
|---|---|---|
| Structure Type: | Main: Approach: | |
| Number of Spans | Structure Length: ft. | Depth of Fill: ft. *(if applicable)* |
| Bridge Roadway Width ft. | No. Traffic Lanes: | Sidewalk Width *(ft)*: Lt. Rt. |
| Year Built: | Year Last Reconstructed/Rehab: | Reconstruction Type: |

| BRIDGE CONDITION RATINGS: | | | |
|---|---|---|---|
| Date of Last Inspection: | | | |
| Deck | Superstructure | Substructure | Culvert |
| Deck Geometry Appraisal: | | Approach Alignment Appraisal: | |
| Comments: | | | |

| **BRIDGE RATING ANALYSIS:** | | | | | |
|---|---|---|---|---|---|
| Controlling Member(s): | | | | Controlling Span(s): | |
| ☐ Non-Redundant | | ☐ Fatigue Sensitive | ☐ Interior/Fascia Girder *(if multi-girder)* | | |
| | | | | | |
| Inventory Ratings: | Full Lanes: H tons | HS tons | ML-80 tons | TK527 tons | |
| | One Truck: H tons | HS tons | ML-80 tons | TK527 tons | |
| Operating Ratings: | Full Lanes: H tons | HS tons | ML-80 tons | TK527 tons | |
| | One Truck: H tons | HS tons | ML-80 tons | TK527 tons | |
| SLC Ratings: | Full Lanes: H tons | HS tons | ML-80 tons | TK527 tons | |
| | One Truck: H tons | HS tons | ML-80 tons | TK527 tons | |
| Analysis Method: | ☐ AASHTO Line Girder with Simplified (S-Over) LL Distribution Factors<br>☐ AASHTO Line Girder with NCHRP LL Distribution Factors<br>☐ 2D/Grillage ( )<br>☐ 3D/FEM ( )<br>☐ PDT Box Culvert Analysis Program<br>☐ Other ( ) | | | | |
| Rating Method: | ☐ Working Stress<br>☐ Load & Resistance Factor<br>☐ Other ( ) | ☐ Load Factor<br>☐ Engineering Judgment | | | |
| Special assumptions used for analysis: | | | | | |
| Controlling member conditions: (i.e., % deterioration, location of deterioration, etc.) | | | | | |
| Are traffic conditions for 'One Truck at a Time' restriction valid according to Pub. 238?<br>*(If 'YES', District Traffic Engineer must approve -- see Page 3 of 3)* | | | | | |

| **PROGRAMMING DATA:** | | | | |
|---|---|---|---|---|
| MPMS #: | Programming Status: ☐ No Work Programmed ☐ Contract ☐ Dept Force<br>☐ On TIP ☐ Twelve Year Program (TYP) - four-year period | | | |
| Scope of Work: | ☐ Replace | ☐ Rehab | ☐ Repair | Estimated Let Date: |
| Costs ($000): | PE - $ | FD - $ | UTL/ROW - $ | Construction - $ |

| **ECONOMIC IMPACT OF RECOMMENDED POSTING:** | |
|---|---|
| Will recommended posting adversely impact Industry/Business? | |
| If yes, provide brief discussion: | |

| **IMPACT OF RECOMMENDED POSTING ON HAULERS ON POSTED AND BONDED ROADWAYS:** | |
|---|---|
| Is the roadway posted, if so what is the weight limit? | Tons |
| Are there any hauling permits already approved that will use this route? | |
| If yes to any question, provide brief description including a potential alternate route or posted bridge permits.: | |

| **IMPACT OF RECOMMENDED POSTING ON EMERGENCY SERVICES, ETC.** | |
|---|---|
| Provide a brief description of the impact on each public service (i.e., permit required, detour required) and explain how each service will be accommodated especially for low load-level postings. Average weights are provided below; however, each owner should verify actual weights of vehicles utilizing the bridge. | |
| Winter services or other maintenance: *(Average loaded weight of 28 Tons for a Plow Truck)* | |
| Ambulance: *(Average weight of 6 Tons)* | |
| Fire Truck: *(Average weight of 19 to 30 Tons)* | |
| School Bus and/or Public Transportation: *(Average loaded weight: School Bus 17 Tons Charter Bus 20 Tons)* | |

| NOTIFICATION TO THE PUBLIC: | |
|---|---|
| If the restriction on the route will cause significant traffic implications, provide description of how the public will be notified (e.g., Press Release). | |

The bridge posting recommendation has been developed and accepted and is accurate to the best of my knowledge.

| APPROVALS | Signature | Date |
|---|---|---|
| District Bridge Engineer<br>*(Only required for PennDOT owned bridges)* | | |
| District Traffic Engineer<br>*(Only if posted "Bridge Limited to One Truck")* | | |
| Local Owner Engineer<br>*(Only for locally owned bridges. Approval must be by a Professional engineer working for the owner or their consultant)* | | |

# APPENDIX IP 04-B

Posting Authorization Request Letter

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pennsylvania
DEPARTMENT OF TRANSPORTATION

**DATE:** **XX-XX-XX**

**SUBJECT:** Posting Authorization Request Letter
BMS No. **XX-XXXX-XXXX-XXXX**
SR **XXXX** over **[FEATURE INTERSECTED]**
**[TOWNSHIP], [COUNTY]**

**TO:** **[NAME]**, District Executive
District **X-**0

**FROM:** **[NAME]**, District Bridge Engineer

Based on **a load rating analysis of current conditions**, the subject bridge requires a weight limit posting of **XX Tons / Except Combinations XX Tons**. The bridge is to be restricted under §4902(a) of the PA Vehicle Code. The bridge is currently **not restricted**. This posting is due **to the main bridge members are deficient and cannot carry legal loads safely.** The most recent inspection date is **[MONTH] XX, XXXX.** The Bridge Load Posting Recommendation Form, Load Rating Summary Form, and **a diagram of standard vehicles and weights (optional)** are attached for your reference.

Based on the weight restriction, Emergency Vehicles will **not be affected.** Also, based on the weight restriction, Haulers will **not need to be re-routed.** Currently, one Permits is approved to use this route. The weight restriction will **have minimal effect on the traveling public; therefore, there is no planned public communication.**

Your signature below will provide approval of the proposed weight limit restriction.

____________________________________
**[NAME]**
District Executive, District **X-0**

Encl: Bridge Load Posting Recommendation Form
Load Rating Summary Form
**Diagram of Standard Vehicles and Weights (optional)**
Site Maps: Portion of State Map and portion of Type 10 County Map

cc: **[NAME]**, Chief Bridge Engineer
**[NAME]**, Assistant Chief Bridge Engineer - Inspection
**[NAME]**, Grade Crossing Unit in the Utilities and Right-of-Way Section **(Only required for Bridges under PUC jurisdiction**)
**[NAME]**, Director of the Center for Program Development and Management

pennsylvania
DEPARTMENT OF TRANSPORTATION

**DATE:** **XX-XX-XX**

**SUBJECT:** Posting Authorization Request Letter
BMS No. **XX-XXXX-XXXX-XXXX**
SR **XXXX** over **[FEATURE INTERSECTED]**
**[TOWNSHIP], [COUNTY]**

**TO:** **[NAME]**, Assistant Chief Bridge Engineer – Inspection

**FROM:** **[NAME]**, District Executive
District **X**-0

Based **on a load rating analysis of current conditions**, the subject bridge requires a weight limit posting of **XX Tons.** The bridge is to be restricted under §4902(a) of the PA Vehicle Code. The bridge is currently posted for **XX Tons.** This posting is due to **the main bridge members are deficient and cannot carry legal loads safely.** The most recent inspection date is **[MONTH] XX, XXXX.** The Bridge Load Posting Recommendation Form, Load Rating Summary Form and **a diagram of standard vehicles and weights (optional)** are attached for your reference.

Based on the weight restriction, Emergency Vehicles will **be affected**. **Large snow removal vehicles and Fire Trucks will require a special permit.** Also, based on the weight restriction, Haulers will **need to be re-routed.** Currently, **two** Permits are approved to use this route. Alternative routes include **utilizing State Route XXXX and XXXX.** The weight restriction will **have minimal effect on the traveling public; therefore, there is no planned public communication.**

Please review this recommendation and forward to the Chief Bridge Engineer for their approval. The signature below will provide approval of the proposed weight limit restriction. Should you require any additional information, please contact, **[NAME]**, District Bridge Engineer at **XXX-XXX-XXXX.**

____________________________________
**[NAME]**
District Executive, District **X-0**

**Approved by:**

____________________________________
**[NAME]**
Chief Bridge Engineer

Encl: Bridge Load Posting Recommendation Form
Load Rating Summary Form
**Diagram of Standard Vehicles and Weights (optional)**
Site Maps: Portion of State Map and portion of Type 10 County Map

cc: **[NAME]**, Deputy Secretary for Highway Administration
**[NAME]**, Chief Bridge Engineer
**[NAME]**, Grade Crossing Unit in the Utilities and Right-of-Way Section **(Only required for Bridges under PUC jurisdiction)**
**[NAME]**, Director of the Center for Program Development and Management

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DEPARTMENT OF TRANSPORTATION

1 TON = 2,000 POUNDS

Average Tractor Trailer (Loaded): **40.0 TONS**

Average Dump Truck (Loaded): **36.0 TONS**


Average Cement Truck (Loaded with 10 Cubic yards of Cement): **33.0 TONS**


Average Plow Truck (Loaded): **28.0 TONS**

Average Garbage Truck (Loaded): **25.0 TONS**


Average Fire Truck: **19.0 TONS - 30 TONS**


Average Charter Bus: **16.0 TONS**


Average School Bus: **12.0 TONS**


Average Delivery Truck (Loaded): **6.0 TONS**


Average Ambulance: **5.0 TONS**

Average Standard Truck: **3.0 TONS**

Average Standard Car: **1.5 TONS**

# APPENDIX IP 06-A

Inspection Report Quality Control
Verification Checklist

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# Inspection Report Quality Control Verification Checklist

## Inspection Report Quality Control Verification Check list

*Structure ID (BRKEY):* ____________ *Inspection Type:* ____________ *Inspection Date:* ____________

*QC Review Completed By:* ____________ *Date:* ____________

| ITEM DESCRIPTION<br>(*Note: Any items marked N should be followed up on and addressed prior to submittal of the report.) | Y | N* | N/A |
|---|---|---|---|
| **Inspection Procedures:** | | | |
| • Were all necessary components inspected and notes updated for changes? Do condition codes match notes and photos? | | | |
| • Was proper access equipment utilized and documented in the report? | | | |
| • Have bridge members been sufficiently cleaned to establish remaining member sections and section loss? | | | |
| • Has the Inspection Planning page in BMS2 been updated? | | | |
| • Was the inspection completed by qualified personnel? | | | |
| • Has an element inspection been completed and elements updated? (Req'd for State >=8' and Local NHS) | | | |
| • Have all necessary compliance inspections been completed? | | | |
| **Maintenance Needs:** | | | |
| • Have maintenance needs been properly identified and in general agreement with Pub 100A guidance? | | | |
| • If there are any Priority 0 or 1 Maintenance needs, has the POA been completed? | | | |
| • Has the proper documentation been entered in BMS2? | | | |
| • Has the owner been notified of any Priority 0 or 1 maintenance needs? | | | |
| • Have Priority 0 maintenance been completed or action been taken to reduce the priority? | | | |
| • If there are Priority 1 maintenance items, has a 6-month Interim been scheduled? (Req'd for all P1's including deferred) | | | |
| **Load Rating:** | | | |
| • Has the District been notified of any recommended need to re-rate the bridge? | | | |
| • Is there proper load rating documentation in the bridge file to support the current rating set? | | | |
| • Is the proper NBI vehicle assigned as per Pub 238 3.6.2? (PHL-93 req'd for bridge built after 2010 designed w/LRFD) | | | |
| • Has the appropriate load rating method of analysis (i.e. simplified vs advanced) been used for the bridge conditions? | | | |
| • If there are portions of any elements in condition state 4 that warrant a structural review, has this been completed? | | | |
| • If Posting is required, are all signs properly installed? | | | |
| **Inspection Scheduling:** | | | |
| • Are the inspection type intervals in accordance with Pub 238 Table IP2.3.4-1.1? | | | |
| • Are the performed inspection type check boxes filled out correctly? | | | |
| • Have the next inspection date fields been updated and are they correct? | | | |
| **Fracture Critical Bridges (as needed):** | | | |
| • Has a hands-on inspection been completed for all NSTMs in accordance with F&F plan? | | | |
| • Is the F&F Plan completed? Has the proper cover sheet been updated and attached to the front? | | | |
| **Report Documentation:** | | | |
| • **Narrative:** (Has a narrative description of general and component/element conditions been provided in BMS3 and supplemental notes as needed? Have changes in condition code been justified?) | | | |
| • **Monitoring/Further Review:** (Has a discussion of factors which require further review, close monitoring, or additional attention during inspections been provided?) | | | |
| • **Posting Evaluation / Load Rating Needs:** (Has the existing posting level or need for a posting been documented? Has the need for an updated load rating been documented? ) | | | |
| • **Location Map** | | | |
| • **Photos:** (Have the following updated photos been provided? Approaches, Elevations, Deck Wrg. Surf., Gen. Super, Gen. Abut, Gen Pier, looking U/S, looking D/S, Regulatory signs, Maint. Items-Priority 0-2 Min.) | | | |
| • **BMS3** | | | |
| • **Scour Assessment/Documentation (as needed):** (Has scour been assessed & the proper scour documentation been provided in accordance with Pub 238 App IP 02-E? Is the SCBI coded properly based on site conditions & existing plans?) | | | |
| • **Sketches (as needed):** (Do sketches provide sufficient detail documenting remaining section to determine capacity?) | | | |
| • **Vertical Clearance Sheets (as needed):** (Does bridge require posting? If yes, are signs properly installed?) | | | |
| • **Load Rating (as needed):** (Provide if updated at a minimum) | | | |
| • **Correspondence (as needed):** (Priority 0/1 Maintenance notification and POA, Load Rating recommendations, etc.) | | | |
| • **Inspection Forms (as needed):** (Non-Comp Adjacent Box, Stone Masonry, or Metal Arch) | | | |
| • **Rocker Bearing Spreadsheet (as needed)** | | | |
| • **F&F Plan with standard cover sheet (as needed)** | | | |
| **BMS2 Inventory and Documents** | | | |
| • Has the assigned load rating approval form been attached in documents when assigned ratings are used? | | | |
| • Have validations been addressed? | | | |
| • Have the following documents been uploaded to BMS2 and tagged to the inspection as needed? (Report pdf, supplemental inspection notes in editable format, Scour POA attachments in pdf form for bridges which need a Scour POA(Document type waterway Cross Sections or Stream Sketch), scour excel document, rocker bearing excel document, Non-Comp Adj Box editable form, Metal arch culvert editable form, Stone masonry editable form) | | | |
| Comments: | | | |

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# APPENDIX IP 06-B

Load Rating Quality Control
Verification Checklist

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## Load Rating Quality Control Verification Checklist

*Structure ID (BRKEY):* ____________ *Rating Program:* ____________ *Load Rating Date:* ________

*QC Review Completed By:* *Date:*

| ITEM DESCRIPTION (*Note: Any items marked N should be followed up on and addressed prior to finalizing the rating) | Y | N* | N/A |
|---|---|---|---|
| **Method of Solution/Section Loss:** | | | |
| • Has the appropriate load rating method of analysis (i.e. simplified vs advanced) been used for the bridge conditions? | | | |
| • Has the appropriate load rating method methodology been utilized (LRFD, LRFR, ASR, or LFR)? | | | |
| • Has the proper analysis/rating program been utilized? (PennDOT programs shall be utilized whenever possible, see DM4 PD 4.4 for acceptable methods of structural analysis and a link to approved programs) | | | |
| • Has any section loss, which would affect the rating, been incorporated? | | | |
| • Has the dead load been distributed properly? Have all necessary dead loads (including wearing surface) been included? | | | |
| • Are the material properties used in agreement with the existing plans or approximated based on the year built (Based on Pub 238 IP 3.7.2.2, MBE 6B5.2.3 and MBE 6B5.2.4)? | | | |
| • If utilized, are temporary measures sufficient to remove live load from a portion of the bridge (Pub 238 IP 3.3.5)? | | | |
| • Are the live load distribution factors applicable based on the load rating methodology, structure type, and field conditions? | | | |
| • Has a skew correction factor been applied to the distribution factor in accordance with Pub 238 IP 3.3.3.1 and DM4 4.6.2.2.3c? | | | |
| • Has the deck been load rated, if necessary, based on conditions? (e.g., timber decks exhibiting excessive deflection under LL, one or more beams considered ineffective and deck must span over this area, or any deck with advanced section loss that may control the rating. See Load Rating Best Practices Manual for additional guidance) | | | |
| • Has the substructure been load rated, if necessary, based on conditions? (e.g., any substructure with advanced section loss or settlement/rotation that may control the ratings, all cross girders, steel substructures with NSTM members, any steel substructure with compromised bracing increasing the unbraced length. See Load Rating Best Practices Manual for additional guidance). | | | |
| • Have PennDOT legal loads and FHWA's EV vehicles been analyzed? (H20, HS20, ML80, TK527, PHL-93 (when req'd), EV2 and EV3) | | | |
| • Have all necessary members been considered in the analysis? (e.g., diaphragms and cross frames in curved superstructures) | | | |
| • Has the SLC factor been applied, when necessary, in accordance with Pub 238 IP 4.3.2? | | | |
| **NSTM Members** | | | |
| • Have all connections on NSTM Members been rated as necessary? (e.g., pin connections, pin and hangers, gusset plate connections, floorbeam hangers, and splice connections) | | | |
| **Steel Beam Bridges** | | | |
| • If considered composite with the deck, is there proper shear transfer between the beams and the deck? | | | |
| • For concrete encased steel I-beams that are considered composite, is composite action appropriate? (Note: Concrete encased I-beams can be considered composite (Bridge type EIB in BAR7) if there is no distress between the beam encasement and deck indicating loss of composite action and the super and deck ratings > 4.) | | | |
| • Are the bracing assumptions used for the rating appropriate? | | | |
| • Are the legs of a K-frame structure rated as part of the super rating? Are section loss and bracing length properly considered? | | | |
| **Timber Bridges** | | | |
| • Has the bridge been rated using ASR? | | | |
| **Non-Composite Adjacent Box Beam Bridges** | | | |
| • Have the live load distribution factors properly accounted for the shear key and post tensioning effectiveness (Pub 238 IE 6B.6.3)? | | | |
| • Has the deterioration been modeled correctly in accordance with Pub 238 IE 6.1.5.3I? | | | |
| • Has the barrier load been distributed in accordance with Pub IE 238 6B.6.1? | | | |

| Load Rating Quality Control Verification Checklist | | | |
|---|---|---|---|
| *Structure ID (BRKEY):* ____________ *Rating Program:* ____________ *Load Rating Date:* ________ | | | |
| *QC Review Completed By:* *Date:* | | | |
| **ITEM DESCRIPTION** (*Note: Any items marked N should be followed up on and addressed prior to finalizing the rating) | **Y** | **N*** | **N/A** |
| **Engineering Judgment Load Ratings** | | | |
| • Is EJ the proper method to analyze the structure (EJ shall only be used when an engineering analysis cannot be completed. Every effort shall be made to complete the rating using engineering analysis including using available design plans, using available applicable PennDOT standard drawings, and obtaining field measurements). | | | |
| • Has the proper EJ procedure been utilized based on the structure type (Pub238 3.6.1.1)? Are the results reasonable based on the structure? | | | |
| **Final Load Rating Package:** | | | |
| • Has a summary table of the controlling ratings been provided and sealed by a PA P.E.? (Similar to the format of Pub 238 Appendix IP 03-C. The form shall include critical information such as listing the controlling member, location, and the section loss modeled.) | | | |
| • Do the input computations and analysis runs contain initials and dates of the preparer and checker? | | | |
| • Has documentation been provided indicating the references and assumptions used for the load rating? | | | |
| • Has documentation been provided of the section loss and how it was captured in the rating analysis? | | | |
| • Has documentation been provided of calculation of dead loads, live load dist. factors, and skew correction factors? | | | |
| • Has the program output or computations been provided to document the computation of the load rating values? (Note the final rating package must include the computations for the controlling members at a minimum. If the output for other members is not provided, a discussion of why they were not provided should be included.) | | | |
| • Has a copy of bridge sketches, design plan sheets, or standard drawings been included for completeness? | | | |
| • Has any correspondence related to the load rating been included for completeness? | | | |
| • Have the ".dat" files been provided for use in APRAS for state owned bridges when PennDOT programs are utilized? If PennDOT programs are not utilized for state owned bridges, have the HS20 one-truck rating factors been provided for use as restricted moment comparison factors in APRAS? | | | |
| **Analysis Results / Posting** | | | |
| • Do the ratings seem reasonable when compared to the previous rating set or the original design? | | | |
| • Has the Posting Rec. form been completed if needed? Have any posting mitigation strategies been considered? | | | |
| **BMS2 Documentation** | | | |
| • Has the new rating set been entered into BMS2 and assigned to the current inspection? | | | |
| • Is the proper NBI vehicle assigned as per Pub 238 IP 3.6.2? (PHL-93 req'd for bridge built after 2010 designed w/LRFD) | | | |
| • Have notes been entered into IR19 to assist future inspections in determining if a re-rating is necessary? (Notes should include critical information such as controlling member and location as well as the section loss in the current rating.) | | | |
| • If needed, has the posting screen been updated? | | | |
| • Has a separate copy of the sealed load rating package been placed in the bridge file? (The preference is to place a copy in BMS2 to allow inspectors to review the rating set to determine if an update is needed.) | | | |
| Comments: | | | |

# APPENDIX IP 06-C

F&F Plan Quality Control
Verification Checklist

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## F&F Plan Quality Control Verification Checklist

*Structure ID (BRKEY):* ______________ *QC Review Completed By:* ______________ *Date:* ______________

*Note: Use this checklist to review the F&F plan prior to the inspection and updated as necesarry. If the inspection team identifies significant issues with the NSTM plan in the field or after the inspection, a return to the field may be necessary to complete a proper NSTM inspection. Attach a copy of the completed checklist to the NSTM plan during each update.*

| **ITEM DESCRIPTION**<br>(*Note: Any items marked N should be followed up on and addressed prior to finalizing the NSTM Plan) | Y | N* | N/A |
|---|---|---|---|
| **F&F Plan Cover Sheet Pub 238 IP 02-H** | | | |
| Is the cover sheet from Pub 238 App IP 02-H included in the front of the plan? | | | |
| Has the F&F plan been reviewed/updated during the Routine Inspection? | | | |
| Does the Bridge Condition/Posting match the current condition of the bridge? | | | |
| Does the inspection scope and interval properly identify the areas of the bridge that require a hands-on NSTM inspection? | | | |
| Has the access equipment and special testing needs been properly identified? | | | |
| Has the proper approval been provided for a limited scope Interim, a less than full hands-on inspection of concrete encased NSTM's, or a proposed interval less than required by Pub 238 Table 2.3.2.4-1? | | | |
| **Location Plans and Detail sketches/photos** | | | |
| Has the location of critical fatigue/fracture prone members and details been identified in a framing plan view with detailed locations labeled by letters or numbers? (includes a legend explaining the letter/number labeling scheme) | | | |
| Has an elevation view of the truss been provided, which identifies the NSTM members? (NSTM members would include all members subjected to tension and stress reversal as indicated in design/rating calculations) | | | |
| Have sketches of fatigue/fracture prone details been provided for each unique detail? (Note, in addition to the sketches, photos are helpful to inspectors to identify the critical details; however, the photos are not required) | | | |
| Have the limits of locations of tension zones for flexural NSTM members been clearly identified if the inspection procedure is to only inspect the tension zones? (Note: Must show limits in the span and show the depth of the neutral axis. Generally the full length and height of a flexural NSTM member should be inspected hands-on unless there are issues with access.) | | | |
| **Table of Fatigue and Fracture Prone Details** | | | |
| Has a table been provided which lists the fatigue/fracture prone details? Are all the locations of the details identified for the bridge? | | | |
| Have all NSTM members and fracture/fatigue prone details been properly identified? AASHTO fatigue category D-E' must be included. If no D or worse details are present on the NSTM member, include the worst category for each unique NSTM member. In addition, include any details with defects. | | | |
| Have member defects been identified (Pub 238 2.4.11.1)? (i.e. plug welds, tack welds, field welds, section loss, existing cracks or arrest holes, and gouges from fabrication or collision damage.) | | | |
| Have complex details been identified? [i.e. pin and hanger(Pub 238 2.4.10.2), details with potential constraint-induced-fracture such as intersecting welds (Pub 238 2.4.11.2), steel connections in tension (i.e. hanger connection of floorbeams, steel truss pins, and truss gusset plates). Some of these details won't have a AASHTO fatigue category, but should be included in the F&F plan.] | | | |
| Have details which are subjected to displacement induced cracking been identified (Pub 238 IP 2.4.9.2)? (i.e. small web gaps which may be present in floorbeam to girder connections, stringer to floorbeam connections, lateral connections to girder or floorbeam webs, or diaphragm connections to girder or stringer webs.) | | | |
| Does the table list the details, provide a description (i.e. weld at end of partial cover plate), and provide a label to identify the detail on the framing plan view? | | | |
| Are the AASHTO fatigue categories labeled for each detail which has a fatigue category?<br>See AASHTO LRFD Bridge Design Specifications, 8th edition for all details except the following: Tack welds (C - AASHTO MBE), Base metal at riveted connections (C - AASHTO MBE), Base metal at riveted connections in poor condition (D - AASHTO MBE). Also, if a member has gouges, lower the fatigue category to E' due to the stress riser. | | | |
| Have inspection procedures been provided for inspection of each detail? (For example, inspect the welds at the ends of the cover plates for cracking). Note, Inspection procedures for truss pins and pins on a pin and hanger detail should include an interval for non-destructive testing of the pins. | | | |
| **BMS2/ Documentation** | | | |
| Has a copy of the excel cover sheet from Pub 238 App IP 02-H been uploaded and tagged to the current inspection? | | | |
| Has a pdf of the F&F plan been uploaded separate from the inspection report and tagged to the current inspection? | | | |
| | | | |

Comments:

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# APPENDIX IP 10-A

Annual and Blanket Permit Vehicles
Authorized in PA

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# Publication 238 (2024 Edition), Appendix IP 10-A
# Annual and Blanket Permit Vehicles Authorized in PA

## Table A - Annual Permit Vehicles

| Load Type | Description | GVW Limit (lbs) | Authorizing Legislation |
|---|---|---|---|
| 34 | Excessive damage (steel coils) | 125,000 | Act 152 of 2002 |
| 35A | Crane (self-propelled) | 100,000 | Act 23 of 1999 |
| 35B | Crane (self-propelled) | 201,000 | Act 23 of 1999 |
| 37A | Float/Flat Glass (5 axle) | 100,000 | Act 23 of 1999<br>Act 37 of 2001 |
| 38A | Waste Coal (overweight) | 95,000 | Act 151 of 1998 |
| 38B | Beneficial Combustion Ash (overweight) | 95,000 | Act 151 of 1998 |
| 38C | Limestone (Overweight) | 95,000 | Act 152 of 2002 |
| 38D | Waste Tires | 95,000 | Act 81 of 2010 |
| 39 | Refined Oil (overweight in bulk) | 107,000 | Act 151 of 1998 |
| 41 | Particleboard/Fiberboard (overweight) | 107,000 | Act 151 of 1998 |
| 42B | Building Structural Component (overweight) | 116,000 | Act 37 of 2001 |
| 50B | Course of Mfg (overweight) - Hot Ingot/Hot Box | 150,000 | Act 151 of 1998 |
| 50C | Course of Mfg (overweight) - Flat Rolled Steel Coils or Slabs | 100,000 | Act 151 of 1998<br>Act 37 of 2001 |
| 50D | Course of Mfg (overweight) - Road Tested Crane | 150,000 | Act 151 of 1998 |
| 50E | Course of Mfg (overweight) - Raw Coal | 95,000 | Act 151 of 1998 |
| 50F | Course of Mfg (overweight) - <= One Mile (milk/coal) | 95,000 | Act 151 of 1998 |
| 50G | Course of Mfg (overweight) - Raw Water (6 axle) | 96,900 | Act 23 of 1999 |
| 50H | Course of Mfg (overweight) - Pulpwood/Chips (5 axle) | 95,000 | Act 23 of 1999 |
| 50I | Course of Mfg (overweight) – Cryogenic Liquid | 102,000 | Act 187 of 2010 |
| 50J | Course of Mfg (overweight) - Pulpwood/Chips (6 axle) | 107,000 | Act 23 of 1999 |
| 50K | Course of Mfg (overweight) – Milk (Except Raw Milk) | 95,000 | Act 34 of 2016 |
| 50L | Course of Mfg (overweight) – Eggs | 95,000 | Act 187 of 2012 |
| 50M | Course of Mfg (overweight) – Sugar | 95,000 | Act 34 of 2016 |
| 50X | Course of Mfg (overweight) – Nonhazardous Liquid Glue | 105,000 | Act 81 of 2010 |
| 56F | Containerized Cargo - Refrigerated Meat Products (6-axle) | 107,500 | Act 50 of 2005 |

**Table B - Blanket Permit Vehicles**

| Load Type | Description | GVW Limit (lbs) | Authorizing Legislation | Designated Networks |
|---|---|---|---|---|
| 44 | Live Domestic Animals | 95,000 | Act 151, 1998 | PA & US Routes, and 4-digit SR’s |
| 45 | Domestic Animal Feed/Grain | 95,000 | Act 8, 1996 | PA & US Routes, and 4-digit SR’s |
| 50A | Course of Manufacture - Raw Milk | 95,000 | Act 151, 1998, Act 89, 2013 | Interstates, PA & US Routes, and 4-digit SR’s |
| 56 A-E | Containerized Cargo | 90,000 | Act 172, 1994 | Interstates, PA & US Routes, and 4-digit SR’s |

# APPENDIX IP 10-B

## ABAS
## Abbreviated Flowchart for Simple Spans

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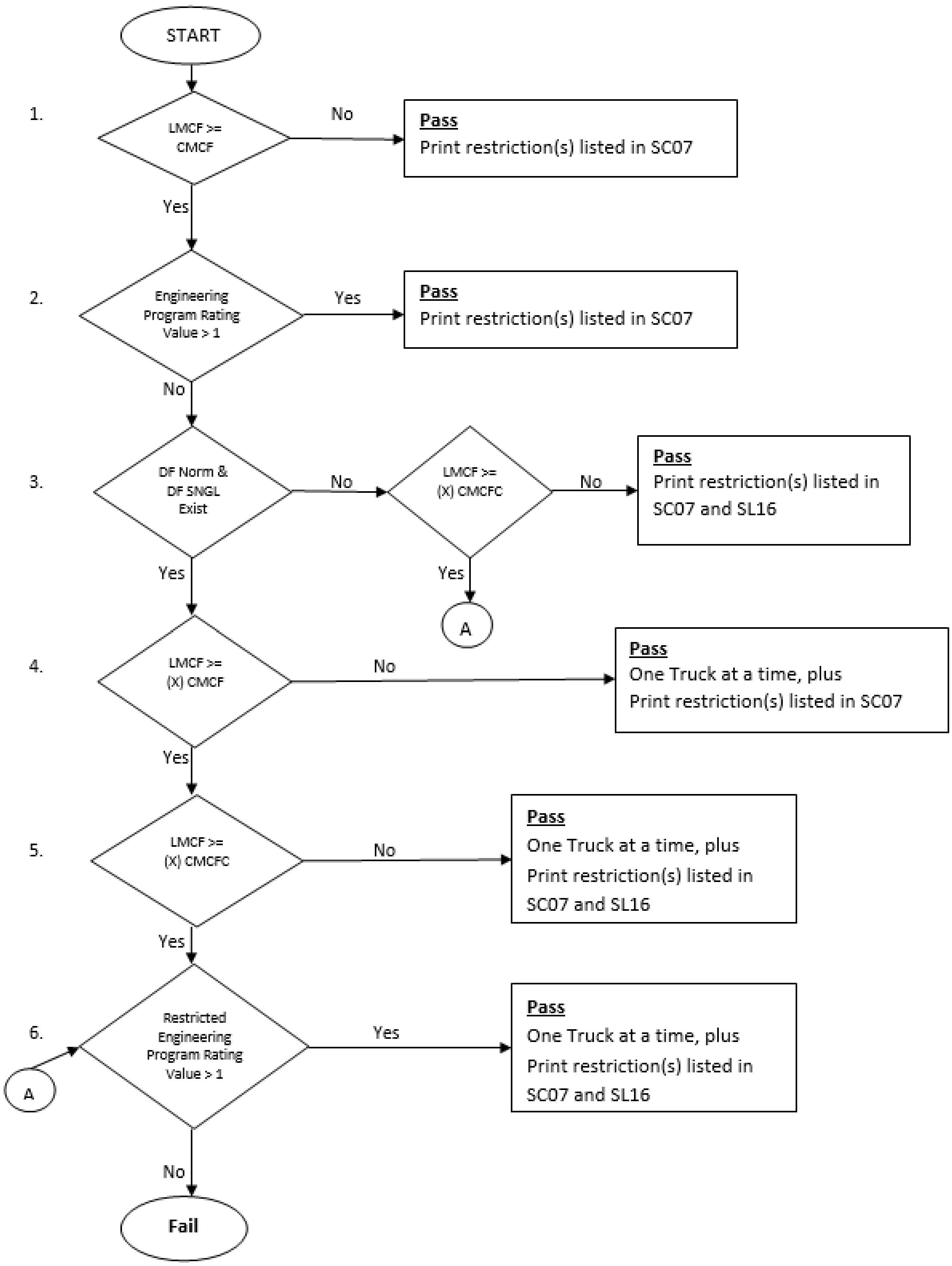
# ABAS Abbreviated Flowchart for Simple Spans



## Description

1. The Load Moment Comparison Factor (LMCF) is generated by our Mainframe computer program P4351050 - Comparison of Live Load Moments and Reactions when executed for the permit vehicle.

   The Capacity Moment Comparison Factor (CMCF) is defined as the ratio of the moment capacity of the span to the maximum moment caused by the HS20 loading at the point of maximum moment. CMCF is stored in BMS2 Items SL12 and SL13.

   The restrictions listed in BMS2 Item SC07 are printed every time the bridge/reference is encountered.

2. An engineering program dataset (BAR7, PS3, BOX5, STLRFD, PSLRFD, BXLRFD) is executed for the permit vehicle. The dataset must be referenced in BMS2 Item SS01 - SPAN ID and stored in the Bridge Rating Input Management System.

3. DF NORM is the Live Load Distribution Factor for Moment for Normal traffic. DF NORM is stored in BMS2 Item SL08. DF SNGL is the Live Load Distribution Factor for Moment for traffic restricted to one truck at a time. DF SNGL is stored in BMS2 Item SL09.

3.1 LMCF same as in 1. The Capacity Moment Comparison Factor Comment (CMCFC) is defined as the ratio of the moment capacity of the span to the maximum moment caused by the HS20 loading at the point of maximum moment modified to allow increased capacity. CMCFC is stored in BMS2 Item SL12. If the CMCFC is used, then the restriction(s) stored in SL16 LOAD CONDITIONS will be printed on the permit.

4. LMCF, CMCF same as in 1. (X) is a calculated value from BMS2 Item SL08. (X) = DF NORM / DF SNGL. If (X) is used, then the permit is approved for one truck at a time. ABAS will not allow (X) to be used with the one truck at a time coding that can be stored in SL16 or SC07.

5. LMCF same as in 1. (X) same as in 4. CMCFC same as in 3.1.

6. A restricted engineering program dataset (BAR7, PS3, BOX5, STLRFD, PSLRFD, BXLRFD) modified for one truck at a time and other restrictions desired, is executed with permit vehicle. The dataset must be referenced in BMS2 Item SL17 - SNGL LANE SPAN ID and stored in the Bridge Rating Input Management System. If the restricted engineering program dataset is used, then the restriction(s) stored in SL16 LOAD CONDITIONS will be printed on the permit.

# APPENDIX IP 10-C

APRAS Vertical Clearance Flowchart

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**Publication 238 (2024 Edition), Appendix IP 10-C**
**APRAS Vertical Clearance Flowchart**

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## PART IE: EVALUATION SPECIFICATIONS



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## PART IE: EVALUATION SPECIFICATIONS

Page

### Chapter 1 – Introduction



### Chapter 2 – Bridge Files and Documentation



### Chapter 3 – Bridge Management Systems



### Chapter 4 – Inspection Procedures

**Chapter 5 – Material Testing**



**Chapter 6 – Load Rating**



**Chapter 7 – Fatigue Evaluation of Steel Bridges**



**Chapter 8 – Nondestructive Load Testing**

**<u>Appendices for Chapter IE 04 Inspection</u>**



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SPECIFICATIONS

### 1.1 PURPOSE

*The following shall supplement M 1.1.*

See IP 1.1 for the Purpose of this Manual. It shall be noted that Part IE of this Manual directly correlates to the MBE. Sections of the MBE which the Department modifies or supplements are listed herein. The absence of a section indicates that the Department accepts that section of the MBE in its entirety.

### 1.3 APPLICABILITY

*The following shall supplement M 1.3.*

The provisions of this Manual apply to all Pennsylvania highway structures that qualify as a bridge in accordance with the definition provided in IP 1.5.

### 1.4 QUALITY

*The following shall replace M 1.4.*

PennDOT uses a combination of quality assurance/quality control and external quality assurance measures to ensure accurate and consistent inspection data collection, analysis and load rating procedures, and file maintenance. Specific policy on the application of quality measures to Pennsylvania highway bridges can be found in this Manual under Part IP, Chapter 6 – Quality Measures for Safety Inspection.

### 1.5 DEFINITIONS AND TERMINOLOGY

*The following shall supplement M 1.5.*

See Glossary and Abbreviations at the beginning of this Manual for definitions and terminology pertinent to this Manual.

### 1.6 REFERENCES

*The following shall supplement M 1.6.*

See IP 1.3.2 and IP 1.3.3 for references pertaining to this Manual.

COMMENTARY

IC1.1 ‘This Manual’ as used within the MBE refers to the MBE. ‘This Manual’ as used within Publication 238 refers to Publication 238.

IC1.5 Some definitions in Publication 238 may differ from those in MBE. The reader shall reference the appropriate definitions and terminology when using each manual.

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SPECIFICATIONS

COMMENTARY

*The following shall replace M 2.*

## 2.1I GENERAL

Chapter IP 8 contains PennDOT's requirements for bridge inspection records and files.

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SPECIFICATIONS

COMMENTARY

*The following shall replace M 3.*

## 3.1I GENERAL

A detailed description of Pennsylvania's Bridge Management System 2 (BMS2) is contained in Chapter IP 5.

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SPECIFICATIONS COMMENTARY

## 4.1 INTRODUCTION

*The following shall supplement M 4.1.*

The Bridge Safety Inspection Program is managed and administrated by the Bridge Unit in each of the Department's Engineering Districts, in conjunction with the bridge owners.

Bridge safety inspection provides information on each bridge that is needed to complete and update each bridge's inventory/inspection database. This data resides in the Bridge Management System 2 (BMS2). This system accepts, stores, updates, and reports physical and operating characteristics for all public bridges in Pennsylvania.

Additional requirements for the safety inspection of PA bridges are to be found in the various sections of this Manual, in the Policies and Procedures Chapters, and as noted below.

## 4.2 PROVISIONS TO SUPPORT THE NBIS REQUIREMENTS

### 4.2.1 Bridge Inspection Organization

*The following shall supplement M 4.2.1.*

For information on the Department's organization for safety inspection of PA bridges see IP 2.1.1.

### 4.2.2 Qualifications of Personnel

*The following shall supplement M 4.2.2.*

See IP 2.1.3 for PA requirements for qualifications of personnel.

### 4.2.3 Inspection Types

#### 4.2.3.1 INITIAL INSPECTION

*The following shall supplement M 4.2.3.1.*

Additional requirements for PA bridges are contained in IP 2.3.1.

#### 4.2.3.2 ROUTINE INSPECTION

*The following shall supplement M 4.2.3.2.*

The Routine Inspection should pay particular attention to critical areas of the structure such as at or under deck joints and drains, at bearings, at splices, connections, etc.

Additional requirements for PA bridges are contained in IP 2.3.2.

#### 4.2.3.3 IN-DEPTH INSPECTION

*The following shall supplement M 4.2.3.3.*

SPECIFICATIONS COMMENTARY

Additional requirements for PA bridges are contained in IP 2.3.4.

**4.2.3.4** FRACTURE-CRITICAL MEMBER INSPECTION

*The following shall supplement M 4.2.3.4.*

Additional requirements for PA bridges are contained in IP 2.4.

**4.2.3.5** UNDERWATER INSPECTION

*The following shall supplement M 4.2.3.5.*

Additional requirements for PA bridges are contained in IP 2.6.2.

**4.2.3.6** SPECIAL INSPECTION

*The following shall supplement M 4.2.3.6.*

Throughout this manual, these inspections are referred to as "Other Special (Interim) Inspections" instead of "Special Inspections." Additional requirements for PA bridges are contained in IP 2.3.5.

**4.2.3.7** DAMAGE INSPECTION

*The following shall supplement M 4.2.3.7.*

Additional requirements for PA bridges are contained in IP 2.3.3.

**4.2.4 Inspection Interval**

**4.2.4.1** INITIAL INSPECTION INTERVAL

*The following shall supplement M 4.2.4.1.*

Additional requirements for PA bridges are specified in IP 2.3.1.4.

**4.2.4.2** ROUTINE INSPECTION INTERVAL

*The following shall supplement M 4.2.4.2.*

Additional requirements for PA bridges are specified in IP 2.3.2.4.

**4.2.4.3** IN-DEPTH INSPECTION INTERVAL

*The following shall supplement M 4.2.4.3.*

Additional requirements for PA bridges are specified in IP 2.3.4.4.

**4.2.4.4** FRACTURE-CRITICAL MEMBER INSPECTION INTERVAL

*The following shall supplement M 4.2.4.4.*

Additional requirements for PA bridges are specified IP 2.4.7.

SPECIFICATIONS COMMENTARY

**4.2.4.5** UNDERWATER INSPECTION INTERVAL

*The following shall supplement M 4.2.4.5.*

Additional requirements for PA bridges are specified in IP 2.6.2.4.

**4.2.4.6** SPECIAL INSPECTION INTERVAL

*The following shall supplement M 4.2.4.6.*

Additional requirements for PA bridges are specified in IP 2.3.5.4.

**4.2.4.7** DAMAGE INSPECTION INTERVAL

*The following shall supplement M 4.2.4.7.*

Additional requirements for PA bridges are specified in IP 2.3.3.4.

## 4.3 NONREGULATORY INSPECTION PRACTICES

### 4.3.3 Planning, Scheduling, and Equipment

**4.3.3.3** EQUIPMENT

*The following shall supplement M 4.3.3.3.*

Additional guidelines for standard inspection tools and equipment are given in IP 9.

**4.3.3.3.1** Access Methods and Equipment

*The following shall supplement M 4.3.3.3.1.*

The Department owns a fleet of under-bridge inspection cranes to be used for the safety inspection and maintenance of its structures. See IP 1.12 for a description of the crane program.

### 4.3.4 Inspection Forms and Reports

**4.3.4.2** REPORTS

*The following shall supplement M 4.3.4.2.*

For report requirements for the inspection of PA bridges, see the General Scope of Work documents; Appendix IP 01-F through IP 01-H and Appendix IP 02-D for sign structures.

### 4.3.5 Inspection Techniques

**4.3.5.2** CLEANING

*Insert the following after the first paragraph in M 4.3.5.2.*

Many bridge problems caused by corrosion and concrete deterioration have become emergencies because the structural deterioration was accelerated

SPECIFICATIONS COMMENTARY

and/or not discovered during inspection due to debris build-up on bridge members. The high cost of emergency repairs and retrofitting to correct these deficiencies emphasizes the importance of cleaning bridges sufficiently to ensure that the inspection identifies problems in a timely manner. If portions of the bridge inspection cannot be completed to a satisfactory level of intensity because extensive cleaning is required, that cleaning should be scheduled promptly to ensure the inspection can be completed. This shall include cleaning and flushing the bridge deck, horizontal steel surfaces of the superstructure, and any other details that are likely to trap debris, moisture, and bird droppings.

Identify these bridge cleaning needs on the IM inspection form.

**4.3.5.4** CRITICAL INSPECTION FINDINGS

*The following shall replace M 4.3.5.4.*

Critical structural and safety-related deficiencies found during the field inspection and/or evaluation of a bridge should be brought to the attention of the bridge owner immediately. If the deficiency threatens the structural integrity of the bridge to the point that public safety cannot be assured, close the bridge immediately. The bridge should not remain open to pedestrians only unless an evaluation has determined it to be safe for that loading.

Once closed, the bridge may not be re-opened until further evaluation and/or repairs are made to ensure the bridge is safe for its posted weight limit. This decision to re-open the bridge must be made by the Professional Engineer in charge of the inspection because of the public safety issues.

For additional information see IP 2.13.2 through IP 2.14.

**4.3.5.5** DECKS

**4.3.5.5.1** Concrete Decks

*The following shall supplement M 4.3.5.5.1.*

Adjacent box beam structures that do not have a separate concrete deck shall have the top flange of the adjacent box beams treated as a deck for the purpose of establishing a deck condition rating (BMS2 Item 1A01). If the box beams have been covered by an asphalt wearing surface, the deck rating may be based on:

- The condition of the top of the beams before the wearing surface was placed, if known.
- The condition of the underside of the superstructure.
- Because the condition of the wearing surface gives an indication of the deck condition, the deck condition rating typically should not be higher than the wearing surface condition rating unless there is strong evidence to support otherwise.

**4.3.5.5.5** Expansion Joints

*The following shall supplement M 4.3.5.5.5.*

SPECIFICATIONS

Debris in joints causes damage to the joint. A maintenance item for cleaning and flushing the deck should be recorded to clean the joint. See BMS2 Coding Manual (Pub. 100A) BMS2 Item IM03.

**4.3.5.5.7** Drainage

*The following shall supplement M 4.3.5.5.7.*

Drainage deficiencies on non-redundant structures, especially those with FCMs shall be given a high priority for maintenance. See BMS2 Coding Manual, Publication 100A BMS2 Item IM05.

**4.3.5.6** SUPERSTRUCTURE

*The following shall replace the first sentence of M 4.3.5.6.*

This section includes discussions covering inspection of superstructure components composed of prestressed concrete, reinforced concrete, structural steel, or timber, including bearings, connection devices, and protective coatings.

**4.3.5.6.1** Steel Beams, Girders, and Box Sections

*The following shall supplement M 4.3.5.6.1.*

Guidance and requirements for the inspection of steel bridges considering fatigue and fracture is presented in IP 2.4.

Guidance and requirements for the inspection of uncoated weathering steel bridges is presented in IP 2.2.10.

**4.3.5.6.2** Reinforced Concrete Beams, Girders, and Box Sections

*The following shall supplement the first paragraph of M 4.3.5.6.2.*

To aid in locating hairline cracks, wet the concrete surface with small amounts of water and allow to dry. Cracks will be visible due to capillary action of the water in the cracks.

**4.3.5.6.3** Prestressed Concrete Beams, Girders, and Box Sections

*The following shall supplement the first paragraph of M 4.3.5.6.3.*

For Prestressed beams made continuous for live load, examine the beams carefully for cracks in the region within two to three beam depths from interior supports. Diagonal web cracks may be evidence of shear-related problems. Transverse cracks across the bottom flange may be caused by poor bonding or development of the positive moment hook bars and/or the pre-stressing strands. Longitudinal cracking of the bottom flange, especially in box beams, may be an indication of corrosion of prestress strands. The level of inspection intensity and the presence or lack of cracking should be noted in the field reports so that long-term performance of beams can be tracked. Because the details and methods of construction for prestressed beam bridges made continuous for live load are varied, the design, shop drawings, and construction records should be carefully reviewed for the inspection.

COMMENTARY

IC4.3.5.6.3 Prestressed concrete beams made continuous for live load may be subject to positive moment stresses at interior supports due to forces created by restraint of creep and shrinkage of the beam concrete.

SPECIFICATIONS

To aid in locating hairline cracks, wet the concrete surface with small amounts of water and allow to dry. Cracks will be visible due to capillary action of the water in the cracks.

**4.3.5.6.3.1I** *Adjacent Non-Composite Prestressed Concrete Box Beams*

The inspection of adjacent non-composite prestressed concrete box beams is to include a review of the items listed below with the findings documented in the inspection report:

Beam Spalls and/or Delaminations:

- Location on beam
- Location within span
- Dimensions of spall (length, width, depth)
- Type and size of steel exposed, if any, (mild or prestressing steel)
- Probable cause of spall
- Date spalls were first discovered

Note: Loose concrete should be removed during inspection to determine extent of spall and to prevent debris from falling on any underpassing route.

Exposed and/or Damaged Strands:

- Location within span.
- Number and size of strands exposed/damaged
- Date strand exposure/damage first noted
- Probable Cause, if different from spall

Web Cracking:

- Number
- Width
- Orientation
- Location within span
- Rust staining in area
- **Note: Cracks directly under or beginning at an open deflection joint parapet in the middle ½ of the span should be suspected as a potential indicator of sudden beam failure and shall be monitored.**

Flange Cracking:

- Number
- Width
- Location – within span and transversely on beam
- Orientation
- Rust staining in area

Other General Information

- Beam camber or sag - Flat or negative beam camber seen in the field may be indicative of internal distress. Measurements can be made to compare to as-built conditions or shop drawings.
- Shear key condition, if visible. Leakage through the shear keys or longitudinal cracks in the pavement shall be noted.
- See Appendix IE 04-C for inspection forms to aid in documentation.

Plan and Cross-Section Sketches of Beams

COMMENTARY

IC4.3.5.6.3.1I Without an effective Non-Destructive Evaluation (NDE) tool to detect the extent of strand corrosion and the remaining effective prestressing force, the best information of current beam conditions must be made available to the rating engineer to predict the safe load capacity. Some items, above and beyond the strand loss and concrete deterioration/damage, that may be contributing factors to failures include:

- No concrete deck – when only an asphalt wearing surface and no waterproofing membrane is provided, roadway drainage can be held in the overlay, creating a continually wet environment for corrosion.
- Without a composite concrete deck, redundancy of beams is reduced.
- Shear keys – poor quality grout does not provide an effective load transfer mechanism between beams. The effectiveness of the shear key can deteriorate with age.
- Transverse tie rods – without significant posttensioning and/or effective shear keys, tie-rods cannot be fully depended upon for load sharing, especially for fascia beams.
- Severe skew (< 60º)
- Asymmetrical loss of prestressing force and/or concrete quality due to damage or corrosion.
- Open joints between parapet sections can direct roadway drainage onto the outside face of the fascia beam and provide a point

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The bridge inspection and rating file shall contain a plan and cross-section of any beam rated. All beams with exposed strands shall have a cross-section showing the size and locations of exposed and/or damaged strands. For consistency, refer to Appendix IE 04-D for examples of beam sketches with exposed or damaged strands for documentation during inspection and analysis.

Adjacent non-composite prestressed concrete box beam bridges with damaged strands or concrete shall be considered high priority for inspection and ratings.

**4.3.5.6.4** Timber Systems

*The following shall supplement M 4.3.5.6.4.*

Stressed timber superstructures should receive special attention during inspections. Stressed timber superstructures consist of longitudinal timber planks (set on edge) that are squeezed together by transverse post-tensioning high strength steel bars. This post-tensioning makes the timber planks act together as if the bridge were a solid slab. Because of the potential for creep of the planks or the crushing of the wood under the anchor plates for the transverse post-tensioning, over time the applied force may relax and the "slab" action may be reduced or lost, resulting in a loss of live load capacity.

Two items that may be indicative of the ongoing structural performance of a stressed timber superstructure bridge are:

- Live Load Deflection –Should be limited to L/500 as recommended in the AASHTO Specifications.
- Bar Force in the Post-tentioned Tendons – For bridges of sawn lumber, the bar force should be checked annually for the first 2 years and subsequently every 2 years. After the bar force stabilizes, this period may be extended to 2 to 5 year intervals.

**4.3.5.6.6** Trusses

*The following shall supplement M 4.3.5.6.6.*

Check for global buckling of truss compression members along their length and also check for localized buckling of truss member elements. Missing/deficient lacing bars and/or batten plates on built-up truss compression members can severely limit their capacity against buckling.

Refer to Appendix IE 04-A for blank forms to aid in the inspection of truss bridges.

GUSSET PLATES

Truss gusset plates shall be inspected to obtain the necessary information to perform a load rating analysis, and examined for the following deficiencies:

**Out-of-plane distortion (bowing):** Gusset plate distortion can be caused by overstressing of the plate due to overloaded vehicles or inadequate bracing during initial erection. Pack rust may be another cause for distortion (bowing). Bowing due to pack rust is generally directly proportional to the amount of pack rust between the plate and the member. Distortion may occur at the edges or internal regions of the plate.

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of reduced beam stiffness or stress concentration.

IC4.3.5.6.4 The Transportation Research Record 1740 Paper No. 00-1191 entitled "Field Performance of Stress-Laminated Timber Bridges" provides a good overview of this bridge type and was the source for the recommendations in the second paragraph.

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Use a straightedge to evaluate and quantify any distortion. The plate distortion shall be measured as the distance between the straightedge and the plate.

**Corrosion and section loss:** Corrosion is formed on steel surfaces due to moisture penetrating the protective coating. Areas that trap debris or hold water are most susceptible to corrosion and section loss. Proper visual inspections can be impeded due to debris and heavy rust. Areas with corrosion should be cleaned and evaluated.

The detection of corrosion in gusset connections is often hampered by its configuration. The insides of gusset plates, which are perhaps the most susceptible to corrosion, are often difficult to visually inspect. Therefore, nondestructive evaluation (NDE) technologies such as D-meters and ultrasonic equipment shall be used at locations where visual inspections may be inadequate to assess and quantify conditions such as section loss due to corrosion. Inspectors are to identify locations requiring NDE and recommend the appropriate type of NDE to be used.

**Cracked welds:** Welds on tension members are considered fatigue prone details because when/if the weld cracks, there is a potential for the weld to propagate into the base metal.

Thoroughly document partial and full length cracked tack welds. Removal of partial length cracked tack welds is recommended.

**Slippage or cracks at mechanical connections:** Depending on the detail, pack rust causing plate separation can lead to overstressed mechanical fasteners. Rivet or bolt heads can “pop” off (tension failure) under the extreme force generated by pack rust. Also, rivets or bolts may be missing from the connection.

Inspect fasteners by hammer sounding, and observe connection for slipped surfaces around individual fasteners. Inspect around gusset plate fastener heads for evidence of cracks emanating from the fastener holes. Any crack found in a gusset plate should be considered critical.

**Repairs/retrofits:** Structural steel repairs and retrofits are used to strengthen deteriorated and bowed gusset plates.

All repairs/retrofits should be inspected for alignment, deterioration, pack rust, etc. as a means to ensure the repairs/retrofits are functioning as intended.

**4.3.5.6.7** Cables

*The following shall supplement M 4.3.5.6.7.*

Note any abrasions on the cable due to contact with steel pieces. Cables consisting of helically wrapped strands will rotate clockwise and counterclockwise under live load deflection. If these cables are in contact with steel pieces that do not move in unison with the cable, this rotation will effectively saw through the outer strands of the cable.

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**4.3.5.6.8** Diaphragms and Cross-Frames

*The following shall supplement M 4.3.5.6.8.*

Diaphragms and cross frames in curved steel multi-girder bridges and in straight steel bridges with skew angles less than 70° can carry significant loads and are considered to be main structural members. Because the diaphragms and cross-frames are essential to the structural integrity of curved girder bridges, especially note deficiencies such as buckling, and deteriorated or cracked members and connections and assign an appropriate priority for their repair.

**4.3.5.6.9** Lateral Bracing, Portals, and Sway Frames

*The following shall supplement M 4.3.5.6.9.*

Note any missing or deteriorated connection bolts or rivets.

Any bracing or cross frame details with welds intersecting with or ending near welds on the main girder may be subject to fracture without notice. See IP 2.4.11.2 for additional guidance on such details.

**4.3.5.6.11** Pins and Hangers

*The following shall supplement M 4.3.5.6.11.*

On many of PA's bridges with pin hangers, secondary or "catcher" systems have been installed to provide redundancy in the event of a pin hanger failure. Typically, these systems were designed to be effective only if the pin/hanger failed and must be monitored to ensure they allow adequate thermal movement of the bridge. All members, connections, and other appurtenances associated with these systems should be inspected as part of the fracture critical inspection. Auxiliary neoprene bearings were used on the catcher beam to limit the free fall of the suspended girder and reduce its impact loading on the catcher system. This auxiliary bearing must be monitored to ensure it is in the proper position as noted on the design/shop drawings.

**4.3.5.6.12** Bearings

*The following shall supplement M 4.3.5.6.12.*

Abnormal or unusual gap measurements at deck expansion joints may be an indication of frozen or improperly functioning bearings as described in IE 4.3.5.6.12.1I. This may also be an indication of substructure deflection or movement. For bridge joints with movements greater than 3", it is good practice to record the gap with each inspection to establish long-term expansion movements. Additional readings during different seasons at extreme temperatures may be needed for a more complete assessment.

**4.3.5.6.12.1I** *Rocker Bearings*

Rocker bearings are generally designed to be set at 68° F, which means that the rocker bearings should be vertical (no tilt) at 68° F by design. However, due to fabrication and construction tolerances, rocker bearings in the vertical position at ambient temperatures up to 15° F higher and lower than 68° F would still be acceptable. The normal behavior of rocker bearings is to tilt away from

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IC4.3.5.6.12.1I There have been two known incidents involving bridges with steel rocker bearing that have exceeded the available movement limit. The first incident occurred in August 2005 carrying I-787 Ramp

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the fixed bearing for that span unit when the temperature rises and to tilt toward the fixed bearing for that span unit when the temperature falls.

Abnormal behavior refers to bearings that are in the contracted position (tilted toward the fixed bearing) in warm weather (above 68° F) or in the expanded position (tilted away from the fixed bearing) in cold weather (below 68° F). In cases where there are two lines of expansion bearings from separate, adjacent span units at a common support, an indication of abnormal behavior is identified by bearings being tilted in the same direction instead of converging or diverging. A rocker bearing that exceeds the acceptable limit of tilt or is bearing on the outer one-quarter width of the rocker for a pier (outer one-tenth width for an abutment) is also an abnormal condition. Abnormal behavior of the bearings may indicate movement of the substructure on which the rocker bearing is founded, movement of the substructure where the fixed bearings are located, or loss of bearing freedom of movement. Note which of these cases may have caused the abnormal behavior.

<u>Any rocker bearing that exceeds the acceptable limit of tilt, (i.e., the rocker is bearing on the outer one-quarter of its width at a pier or the outer one-tenth of its width at an abutment) is considered a critical deficiency</u>.

Critical and High Priority deficiencies found during the inspection should be documented appropriately with photographs and the required information obtained in Appendix IE 04-B. Critical Deficiencies should be brought to the attention of the bridge owner immediately in accordance with Article IE 4.3.5.4. Additionally, for every inspection performed on bridges having rocker bearings, the information in Appendix IE 04-B shall be included in all inspection reports for each location where rocker bearings are present and become a permanent part of the bridge file.

Contact the Assistant District Bridge Engineer-Inspection immediately if a pier with two lines of expansion rockers has any rockers bearing on the outer one-quarter width.

The amount of allowable tilt varies with respect to bearing geometry, expansion length, bridge type, and ambient temperature. To compare the actual tilt to the allowable tilt, the inspector should determine the allowable tilt by completing the tables included in Appendix IE 04-B for initial, routine, in-depth and special inspections. A spreadsheet is available on the BMS2 Home screen to perform the rocker bearing calculations.

Initial readings should be taken after any bearings are reset or if replacement of the deck joints occurs. This will provide a baseline reading for the bearing measurements.

The flexibility of the pier also makes it susceptible to movement from forces generated by temperature change in the superstructure when the bearings lose functionality. Intended functionality or freedom of movement may be restrained or lost by pack rust at bearing surfaces, deck expansion joints that do not allow full range of movement, etc. A high degree of flexibility allows for large deflections at the top of the pier due to the unintended transfer of force to the substructure through improperly functioning rocker bearings. Therefore, pier stems / columns should be inspected for abnormal movement/deflection and flexural cracking; if deemed necessary, pier movement should be monitored with

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Northbound in Albany, New York. The other incident occurred in February 2008, carrying SR 2085 in Pittsburgh, PA. Some of the common characteristics of both bridges at the pier line involving the bearings that exceeded available movement limit are:

- Pier fixity consisted of expansion – expansion
- Piers were relatively tall (greater than 61 feet) and thus relatively flexible compared to adjacent piers
- Inspection documentation over several cycles recorded the bearings being oriented in a parallel displacement configuration instead of diverging or converging.

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surveys. Compare center-to-center of bearing span lengths with the as-built geometry for indications of pier movement.

Excessive abutment rotation/movement may also cause rocker bearings to exhibit abnormal behavior with respect to tilt. Plumbness of the abutment should be checked with a plumb bob and/or survey if necessary.

The physical condition of the bearing (state of corrosion, cracked welds, paint condition, etc.) is assessed independently of the required maintenance to restore the bearing to a fully functioning, as-designed, service state.

Rocker bearing deficiencies are divided into two categories when assigning Maintenance Actions (BMS2 Item Number IM03) and Maintenance Priority Codes (IM05) to restore a rocker bearing to its functioning service state:

1. Normal (due to wear and tear)
2. Critical and High Priority (due to abnormal behavior and extreme functional deficiencies)

Maintenance actions such as cleaning, lubricating, resetting, replacement, etc., and their priorities, as required and due to normal deficiencies should be assigned considering the vulnerability of the structure with respect to structural redundancy, bearing seat width, and minor abnormal behavior. See PUB100A, "IM Inspection – Maintenance" for general guidelines in assigning these actions and priorities. The inspector must use good judgment when assigning high priorities and justify such priorities with adequate documentation. Also, consider adjusting maintenance priority based on ADT of the bridge, feature under the bridge, flexibility of the substructure, number of bearings in a line with excessive tilt, type of superstructure, ability of the bearing to freely move, and evidence of pintel failure. In addition, flexible piers with two lines of expansion bearings shall be given extra scrutiny and maintenance priority shall be assigned accordingly.

Critical and high priority deficiencies of rocker bearings should also be addressed considering structural vulnerability and by assigning Maintenance Actions and Maintenance Priority Codes to correct any noted problems; however, the cause of the functional deficiency should also be addressed. The cause may be due to a more serious structural problem (substructure movement/settlement, for instance) which may require repairs in addition to rocker bearing repairs. The structural problem, if not addressed, may increase structural vulnerability which could lead to more serious consequences such as partial or complete failure of the bridge.

Address the following deficiencies and take action as indicated:

<u>Priority 0 - Critical</u>

- Critical Rocker Bearing Tilt: Applicable where there are one or two lines of expansion rocker bearings on a single pier or abutment and one or more bearings in a line have reached the maximum movement capacity, bearing on the outer one-quarter of the rocker plate on piers (outer one-tenth of the rocker plate for rockers on abutments).

  Remediation options include (but are not limited to):

  - Install temporary supports and assign appropriate monitoring frequency as a temporary measure (note: this option does not alleviate unintended transfer of horizontal force to the

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substructure if bearing freedom of movement is lost). Adequately designed steel or timber cribbing can be used as a temporary measure to mitigate a Priority 0 to a Priority 1; however, a 6 month Other Special (Interim) Inspection will still be required. The monitoring should be completed during extreme temperatures. The purpose of the re-inspection during extreme temperature is to ensure the condition has not gotten worse.

- Reset the bearings by one of the following means: reposition the sole plate by removing the existing welds and re-welding the sole plate to the girder, enlarge and/or slot the anchor bolt holes in the masonry plate to adjust its location, or re-fabricate a masonry plate with the adjusted pintel hole locations.

- Bearing Replacement.

<u>Priority 1 – High Priority</u>

- Potential to Reach Critical Rocker Bearing Tilt: The bearing rotation exceeds the rotation limit for the ambient temperature; however, the bearing has not exceeded the maximum rotation capacity. Applicable when movement analysis indicates a potential for one or more bearings to reach or exceed its maximum movement capacity, potentially bearing on the outer one-quarter of the rocker plate on piers (outer one-tenth of the rocker plate for rockers on abutments).

  Remediation options include (but are not limited to):
  - A follow-up Other Special (Interim) Inspection should be completed at the temperature extreme that is anticipated may cause the bearing to exceed the maximum rotation capacity.
    - Revise the maintenance priority from a Priority 1 to a Priority 0 if the rotation of bearings at the extreme temperature exceeds the maximum rotation capacity.
    - Consider revising the maintenance priority from a Priority 1 to a Priority 2 if the bearings are still within the maximum rotation capacity at the temperature extreme. The bearings can be monitored during scheduled Routine and Interim Inspections if assigned a Priority 2.

  - Install temporary supports and assign appropriate monitoring frequency (note: this option does not alleviate unintended transfer of horizontal force to the substructure if bearing freedom of movement is lost). Adequately designed steel or timber cribbing can be used as a temporary measure to mitigate the Priority 1 to a Priority 2. The bearings can be monitored during scheduled Routine and Interim Inspections.

  - Reset the bearings by one of the following means: reposition the sole plate by removing the existing welds and re-welding the sole plate to the girder, enlarge and/or slot the anchor bolt holes in the masonry plate to adjust its location, or re-fabricate a masonry plate with the adjusted pintel hole locations.

  - Bearing Replacement.

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Extreme temperature is defined as ambient temperature greater than 80° F, less than 40° F, or a temperature difference of 40° F or more from the original inspection.

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- Rocker Bearing Debris Restriction: Rocker bearings located on piers or abutments with heavy accumulations of pack rust, corrosion, and/or debris under one or more rocker could potentially limit or prevent the bearing from operating as it was intended during structure expansion and contraction.

  Remediation options include (but are not limited to):
  - Monitor the bearings and substructure at a 6-month frequency with an Other Special (Interim) Inspection until the repairs are made. The monitoring should be completed during extreme temperatures when possible.
  - Clean and lubricate the bearing to restore proper function. The bearing may need to be completely removed and pack rust removed by mechanical means to properly restore the function of the bearing.
  - Bearing Replacement.

<u>Priority 3 – Schedule</u>

- Abnormal Rocker Bearing Tilt: The bearings are within the acceptable limits at the ambient temperature; however, there is at least one line of expansion rocker bearings and one or more bearings in a line exhibit tilt in the opposite direction indicated by ambient air temperature. Rocker bearings in the contracted position (tilted toward the fixed bearing) in warm weather (ambient temperature above 68° F) or in the expanded position (tilted away from the fixed bearing) in cold weather (ambient temperature below 68° F.)

  Remediation options include (but are not limited to):
  - Monitoring the bearing rotation during regularly scheduled Routine inspections as long as the bearings stay within acceptable limits at the ambient temperature.
  - Reset the bearings by one of the following means: reposition the sole plate by removing the existing welds and re-welding the sole plate to the girder, enlarge and/or slot the anchor bolt holes in the masonry plate to adjust its location, or re-fabricate a masonry plate with the adjusted pintel hole locations.
  - Bearing Replacement.

- Rocker Bearing has exceeded 50% of Movement Capacity: The bearings have exceeded 50% of the movement capacity at the ambient temperature.

  Remediation options include (but are not limited to):
  - Monitoring the bearing rotation during regularly scheduled Routine inspections as long as the bearings stay within acceptable limits at the ambient temperature.
  - Reset the bearings by one of the following means: reposition the sole plate by removing the existing welds and re-welding the sole plate to the girder, enlarge and/or slot the anchor bolt holes

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in the masonry plate to adjust its location, or re-fabricate a masonry plate with the adjusted pintel hole locations.

- Bearing Replacement.

All repairs and superstructure jacking procedures must be prepared, signed, and sealed by a Professional Engineer licensed in the Commonwealth of Pennsylvania.

4.3.5.6.15 Arches

*The following shall supplement M 4.3.5.6.15.*

Check arch spandrel walls for separation from the arch ring and leakage of fill material. Check vertical and longitudinal alignment of the spandrel wall and note any bulging or lateral displacement. Broken or clogged drainage through the arch fill can lead to a long term loss of fine materials in the fill.

The occurrence of winter weather freeze-thaw cycles increases the likelihood of accumulated moisture in the poorly drained fill material of stone masonry structures. As a result, the stability of the structures can be adversely affected to the point of failure. Routine inspection of these structures is warranted just after freeze-thaw cycles so that the Department can identify, prioritize and complete maintenance work. Routine inspection schedules for State-owned stone masonry arches should be adjusted so that they are inspected just after the freeze-thaw cycle. The biennial NBI inspection shall be performed between March 1 and April 30 in order to have these structures evaluated just after the seasonal freeze-thaw cycle.

In addition, all stone masonry arches in poor condition must have an annual Other Special (Interim) inspection performed each year to ensure the safety of these structure types. The Other Special (Interim) inspection shall include all stone masonry portions of the structure.

For both the routine and interim inspections, the inspection forms included in the Stone Masonry Arch Condition Rating Guidelines (Appendix G of BMS2 Coding Manual, Publication 100A) must be used.

4.3.5.7 SUBSTRUCTURE

4.3.5.7.2 Retaining Walls

4.3.5.7.2.1I *Mechanically Stabilized Earth Retaining Walls*

Mechanically Stabilized Earth (MSE) retaining walls should be inspected for evidence of wall movement including rotation and settlement.

- Examine barrier and moment slab for evidence of movement as well as the MSE wall for evidence of bulging, bowing or panel offset.
- Perform a survey if movement is suspected to compare to initial inspection data to gauge amount of movement.
- Examine the roadway above MSE walls for indications of failing pavement or tension cracking. These may indicate a loss of fill.

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- For MSE walls in front of sloping backfill, the crest of the embankment should be investigated for soil stress or failure, both of which may indicate settlement or wall movement.

The joints between panels of MSE walls are to be inspected and examined for loss of backfill, change in spacing, and indications of settlement. The specification requirement for joint spacing is a maximum ¾”.

- Inspect walls for evidence of backfill loss (piles of aggregate at the base of the wall).
- Indicate visibility of backfill or fabric behind the panel through joints.
- Examine for evidence of damage to the geotextile fabric, if visible.
- Look for variation in joint spacing. Note vegetation growing in joints.
- Vertical slip (expansion joints) used on long lengths of walls should be investigated similar to panel joints. The initial spacing at the slip joint should be determined from design, shop or as-built drawings.

Wall panels shall be checked for cracking, spalling, other forms of deterioration, and collision damage.

Drainage systems through or along MSE walls should be inspected to verify water is free flowing into and out of the appropriate facility.

- Ensure that weep holes are free draining.
- Inspect all inlets to verify water is draining into the inlet, and flowing freely to the inlet and out of the outlet. Examine inlets for cracks.
- Inspect visually or use down hole cameras (as appropriate) for all culverts and pipes contained or having portions in, behind, or above the MSE wall mass and for pipes or culverts which run above, adjacent to, or outlet through the MSE walls to verify pipes are free draining and water is flowing through (and not under or around) the pipe. Examine drainage pipes for cracking or damage with emphasis on areas where water may flow, or is flowing, into the MSE wall soil mass. Inspect outlet ends to verify free drainage or for evidence of migration of fill or other material.
- Inspect swales above the MSE wall. Verify rock fall or other materials (trees, etc.) are not blocking, redirecting, or restricting the flow of water through the drainage ditch above the MSE wall to the appropriate receptacle.
- Inspect collection and outlet basins to verify water is draining freely. Look for any signs of infiltration or migration of material which may prevent water from draining from the wall.
- Identify inappropriate appearance of water along the base of the wall (i.e., if water is appearing when weather conditions have been particularly dry). Note areas where there is inappropriate collection and/or lack of drainage for water along the length of the MSE Wall.
- Note erosion of soil along the base of the wall exposing or undermining the leveling pad.

4.3.5.7.2.2I *Geosynthetic Reinforced Soil Retaining Walls*

Geosynthetic Reinforced Soil (GRS) retaining walls should be inspected for evidence of wall movement, loss of backfill and other visible forms of

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deterioration. These walls are similar to MSE walls so the guidance outlined in IE 4.3.5.7.2.1I shall be followed as applicable.

4.3.5.7.3 Piers and Bents

*The following shall supplement M 4.3.5.7.3.*

Where piles are exposed by scour or by design, check piles for lateral stability. Inspect and evaluate piles for both local buckling of the web/flange elements and global buckling of the exposed pile length.

4.3.5.7.4 Bridge Stability and Movements

*Add the following at the end of the third paragraph of M 4.3.5.7.4.*

For large embankments with steep slopes, movements may be caused by deep failure of the embankment and/or underlying soil. A thorough review of foundation information and additional testing may be needed to ascertain the problem. One method for measuring slope movement is to install slope inclinometers and provide long-term monitoring.

4.3.5.8 SCOUR AND WATERWAY INSPECTIONS

4.3.5.8.2 Underwater Inspection

*The following shall supplement M 4.3.5.8.2.*

For additional information on underwater bridge inspections, see IP 2.6.2 and the General Scope of Work for Underwater Inspection of Bridges in Appendix IP 01-H.

4.3.5.8.4 Channel Protection Features-Dolphins and Fenders

*The following shall supplement M 4.3.5.8.4.*

The navigational controls are to be inventoried and noted in BMS2 Items 4A21-4A24 and 4A07. The conditions of these controls should be noted in the inspection report with the substructure unit(s) they protect.

4.3.5.10 CORRUGATED METAL PLATE STRUCTURES

*The following shall supplement M 4.3.5.10.*

The following items shall be inspected and recorded during inspection of Corrugated Metal Plate (CMP) structures:

- Structure length, maximum span length, barrel length, height of cover, and number of barrels
- Evidence of roadway settlement or repair, such as pavement distress or cracks, sign of recent repairing and repaving, and guardrail sagging over the culvert.
- Type and condition of end segment treatment
  a. Type of Headwall with or without traffic barrier
  b. Type of Wingwall or natural slope
- Installation and previous inspection records for comparison.
- Generate a stream sketch to capture current conditions:

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a. Upstream and downstream stream alignment
b. Inlet and Outlet scour condition, (rip-rap, failed headwalls)
c. Culvert blockage/debris
d. Erosion indicated

- Shape or geometry of culvert. Document the location and extent of the observed deterioration (detailed measurements required). It is recommended measurements be taken using a survey rod with a self-leveling cross-line laser level set on a tripod.
  a. Crown flattening
  b. Bulging
  c. Side bow
- Metal plate thickness and condition
  a. Extent of corrosion
  b. Cracking
  c. Missing bolts
- Infiltration and loss of structural backfill through joints or hole through the culvert.

Corrugated metal arch culverts shall be inspected to obtain the necessary information to estimate the current condition rating and examined for the following deficiencies:

**Out-of-plane distortion (bulging):** Metal arch barrel distortion can be caused by overstressing of the plate due to overloaded vehicles or section loss. Pack rust may be another cause for distortion (bulging).

An important feature to observe and measure when inspecting corrugated metal pipe culverts is the cross-sectional shape of the culvert barrel. Any distortions need to be documented. When distortion or curve flattening is apparent, the extent of the distorted area, in terms of length of culvert affected, width of defect (measured along arc length), and the location of the flattened area should be documented. Baseline measurements shall be taken at the first observance of a distortion if no baseline was previously established (e.g., at the time of installation). New measurements shall be taken at all subsequent inspections and compared to the baseline to monitor any changes.

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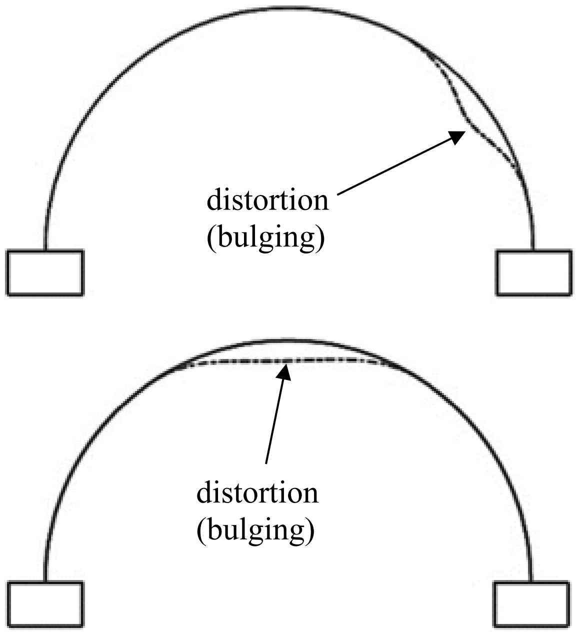


**Figure IE 4.3.5.10-1 – Examples of Out-of-Plane Distortions in Corrugated Metal Arch Culverts**

When distortions are apparent, assign Maintenance Priority Codes as follows:

> 3″ distortion caused by corrosion – Code 0 (critical)
> 2″ ≤ 3″ distortion caused by corrosion– Code 1 (high priority)
≤ 2″ distortion caused by corrosion– Code 2 (priority)

**Corrosion and Section Loss:** Corrosion is formed on steel surfaces due to moisture penetrating the protective coating. Segment seam or bolted joints are typically the places most susceptible to corrosion.

Excessive section loss along longitudinal seams due to corrosion and missing or loose bolts/nuts will result in reduced capacity (lower load ratings) or even culvert collapse. Corrosion loss can be estimated using NDE technology such as D-meters or using engineering judgment.

Document the corrosion and identify the worst 4 ft. length (see Figure IE 4.3.5.10-3). Count the corrugations with holes and measure/estimate the section loss in the other corrugations.

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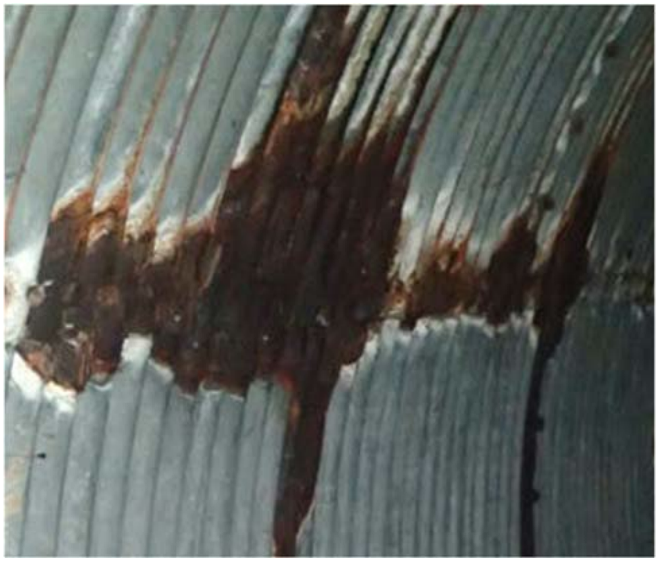

**Figure IE 4.3.5.10-2 – Example of Corrosion and Section Loss Along Longitudinal Bolt Line**

Assign Maintenance Priority Codes for corrosion as follows:

- Severe corrosion along a 4 ft. length with visible holes in approximately 50% or more of the corrugations – Code 0 (critical)
- Serious corrosion along a 4 ft. length with some minor holes – Code 1 (high priority)
- Advanced corrosion along a 4 ft. length – Code 2 (priority)

If a Priority Code 2 is assigned, estimate the remaining useful life of the culvert. To simplify the estimation, section loss in each corrugation may be categorized (e.g., 100%, 75%, 50%, 25% and 0%). The BMS2 Coding Manual, Publication 100A provides an example remaining life calculation (see Section 3.0, IM Screen, Item IM05, Priority Code 2, Culvert Examples). There is also a spreadsheet available for use on the BMS2 homepage.

Assign Maintenance Priority Codes for bolt deficiencies as follows:

- Approximately 50% or more bolts/nuts missing or ineffective over a 4 ft. length – Code 0 (critical)
- Consecutive bolts/nuts missing for a length of 1 ft. to 2 ft. – Code 1 (high priority)

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Example Corrosion Assessment:

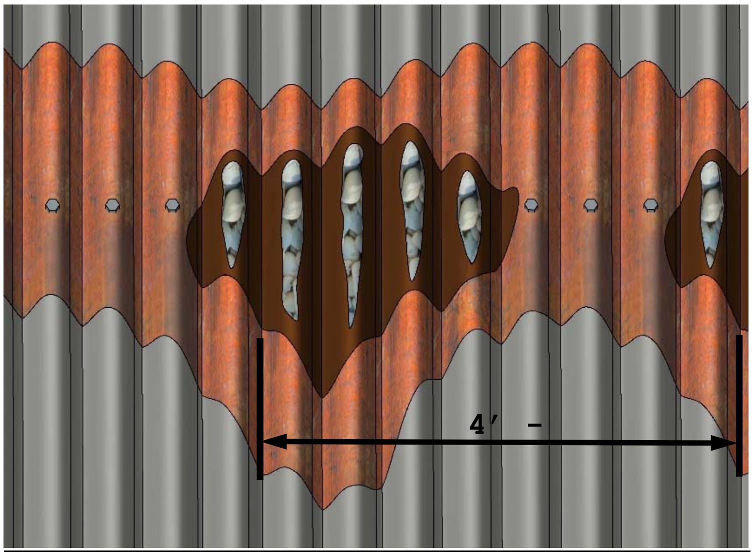


**Figure IE 4.3.5.10-3 – Documentation of Corrosion and Section Loss Along Longitudinal Bolt Line**

Observation: severe corrosion along longitudinal bolt line with visible holes in 5 of 8 corrugations in the worst 4 ft. length

5 / 8 = 63% of corrugations

Conclusion: with holes in more than 50% of the corrugations in a 4 ft. length, Priority Code 0 is recommended.

Additional requirements for PA bridges are contained in IP 2.5.2. See Appendix IE 04-E for inspection forms to aid in the inspection of corrugated metal plate culverts. For suggested repairs, see Publication 55, Bridge Maintenance Manual.

### 4.3.6 Complex Bridge Inspections

4.3.6.5 PRESTRESSED CONCRETE SEGMENTAL BRIDGES

*The following shall supplement M 4.3.6.5.*

Check for corrosion staining especially at segment joints. Note any clogged drain holes or any standing water in box sections.

### 4.3.7 Fatigue-Prone Details and Fracture-Critical Members

*The following shall supplement M 4.3.7.*

Additional requirements for PA bridges are contained in IP 2.4.

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### 4.3.8 Data Collection for Load Rating

*The following shall supplement M 4.3.8.*

Additional requirements for load rating analysis for PA bridges are contained in IP 3 and IE 6.

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SPECIFICATIONS COMMENTARY

## 5.1 GENERAL

*The following shall supplement M 5.*

Additional requirements for PA bridges are contained in IP 3.7.

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## 6.1 SCOPE

*The following shall supplement M 6.1.*

Additional requirements for PA bridges are contained in IP 3.

### 6.1.2 Condition of Bridge Members

*The following shall supplement M 6.1.2.*

Built-up compression members consist of compression elements (channels, angles, plates, etc.) and connecting elements (lacing bars, batten plates, etc.). In addition to recording the condition of the compression elements, record the condition of the connecting elements. Built-up compression members shall have all their connecting elements intact and properly connected to ensure that the entire member is acting to resist the load. If connecting elements have severe section loss, are not properly connected, or are missing, record the location and length of this deficiency. This information will be used to check local buckling of compression elements, which may control the capacity of the built-up compression member.

### 6.1.4 Bridges with Unknown Structural Components

*The following shall replace the first sentence of M 6.1.4.*

For redundant bridges where necessary details, such as reinforcement in a concrete bridge, are not available from plans or field measurements, a physical inspection of the bridge by a qualified inspector and evaluation by a Professional Engineer is sufficient to determine the Inventory and Operating ratings. These ratings shall be recorded in the BMS2 and the bridge inspection as Engineering Judgment. Requirements for Engineering Judgement load ratings for PA Bridges are contained in IP 3.6.1.1.

### 6.1.5 Component-Specific Evaluation

#### **6.1.5.2** SUBSTRUCTURES

*The following shall supplement M 6.1.5.2.*

Reinforced concrete piers shall be load rated if they show signs of distress such as excessive concrete spalling, excessive reinforcing steel corrosion, reinforcing bars not engaged by concrete, or show signs of movement, or exhibit other distress, as directed by the Department. Record the extent and depth of spalling, loss of reinforcing steel cross-sectional area, loss of concrete cross-section, and distressed locations. Calculate the remaining cross-sectional area of the concrete component and the reinforcing steel.

Perform pier load ratings based on operating levels for reinforced concrete members as stated in the current AASHTO Standard Specifications for Highway Bridges, Section 8, or AASHTO LRFD Bridge Design Specifications, Section 5, as supplemented by PennDOT Design Manual 4, or by the strut and tie approach specified in IE 6B.5.3.4I.2 to determine ratings. For shear analysis using working stress methods, do not exceed the limitations contained in the 1973 AASHTO Specifications. The strut and tie method uses lower effective concrete strengths and lower resistance factors than traditional analysis methods

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and therefore, will not predict higher capacities in all cases. However, the strut and tie model may be beneficial where deficient shear reinforcement is compensated by the reserve capacity in the flexural reinforcement or vice versa and in cases with concentrated loads close to supports.

**6.1.5.2.1I** Analysis of Reinforced Concrete Pier Caps

Determine applicable loads, load factors and load cases for the existing pier caps. Actual wearing surface thicknesses shall be used in evaluating the pier caps. Rate the pier caps without the effects of longitudinal loads from live load, i.e. braking forces. Do not include the effects of wind when rating the pier caps.

**6.1.5.2.1I.1** *Pier Cap Rating General Guidance*

The provisions of IP 3 shall apply when load rating reinforced concrete pier caps. Analysis programs which are typically used to analyze new piers may be used to analyze existing piers which show signs of distress. Modifications of the input parameters will be necessary to account for the deterioration of existing concrete and reinforcing steel. Field measurements of all bearings shall be taken to determine if the bearings were placed according to the plans. The bearing locations shall be modeled in the as-built condition to account for any eccentricity which may exist.

Rating calculations will typically need to be computed outside of the program. The rating engineer shall obtain unfactored/factored force effects from the program to compute the ratings. Obtain the concurrent shear and torsion forces for the rating analysis and remove the effects of torsion if torsion was not considered in the original design. Consider the beneficial effects of axial compression on the shear capacity of members per AASHTO SD Equation 8-50, if applicable.

Account for the vertical component of the inclined flexural compression and of the inclined flexural tension force in the tapered members using the equation below. For tapered members, the design shear force shall be adjusted as follows:

$$V_{u,adj} = V_u\left(1 - \frac{M_u}{V_u \cdot d} \cdot \tan\psi\right)$$

where:
- $M_u$ = absolute value of the factored moment
- $V_u$ = absolute value of the factored shear
- d = effective depth
- $\psi$ = sum of angles of compression face and of centroid of flexural reinforcement in tension relative to member axis. Angle $\psi$ is taken as positive if magnitude of moment and depth of member increase or decrease in same sense, negative otherwise.

If shear controls rating, the concrete load near the column may be neglected per AASHTO LRFD AD 5.7.3.2, as this force is transferred by a compression diagonal to the column.

**6.1.5.2.1I.2** *Strut-and-Tie Models of Reinforced Pier Caps*

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IC6.1.5.2.1I.2 Strut-and-tie models can be made externally determinate by proper selection of support reactions. This may require replacing some support

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The flow of forces in a reinforced concrete structure or a portion of a reinforced concrete structure may be approximated by a truss composed of compression members (struts) and tension members (ties) which are connected at nodes. The load rating of reinforced concrete piers shall be based only on the ratings computed in the general zone; do not compute ratings for local zones.

The selected truss model shall establish equilibrium between internal and external forces applied to the structure or the portion of the structure considered.

Tie forces are resisted by reinforcement. Orientation and the centroid of such reinforcement shall coincide with the corresponding ties in the strut-and-tie model.

Strut forces shall be resisted by compressive stresses in the concrete. Select strut dimensions such that the strut forces do not exceed the strut capacities and all struts fit within the boundaries of the portion of the structure considered. The strut area shall be determined by considering both the available concrete area and the anchorage conditions at the ends of the strut, as shown in Figure IE 6.1.5.2.1I.2-1. When a strut is anchored by reinforcement, the effective concrete area may be considered to extend a distance of up to six bar diameters from the anchored bar, as shown in Figure IE 6.1.5.2.1I.2-1(a).

Transverse tensile stresses due to lateral dispersal of compressive stresses in struts of changing width shall be considered.

The angles between struts and ties in the selected truss model shall preferably be greater than 30 degrees and not less than 25 degrees.

Proportion nodal regions to satisfy the geometrical requirements of the nodal boundaries defined by adjacent struts, ties, reinforcing steel and/or bearings. See Figure IE 6.1.5.2.1I.2-1.

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restraint conditions by known reaction forces or vice versa. Strut-and-tie models can be made internally determinate by adding or removing struts.

Several reinforcement bars in the structure may be represented by a single tie in the strut-and-tie model.

Struts represent an idealization of the actual compressive stresses in the structure. At the location of concentrated forces compressive stresses will initially disperse laterally and at some distance from the node become parallel to the strut axis again, forming a bottle-shaped compressive stress field. This redirection of compressive stresses induces transverse tensile stresses. Bottle-shaped stress fields can be modeled using a local strut-and-tie model by splitting the single strut representing the bottle shaped strut into two sets of smaller struts which follow the deviation of the compression stress field.

More guidance for the proportioning of struts, and nodal region can be found in ACI Appendix A, and various industry publications.

References:

“Verification and Implementation of Strut-and Tie Model in LRFD Bridge Design Specifications,” NCHRP report 20-07 (Task 217), Modjeki and Masters Inc., AASHTO 2007

"Toward a Consistent Design of Structural Concrete", Jorg Schlaich,

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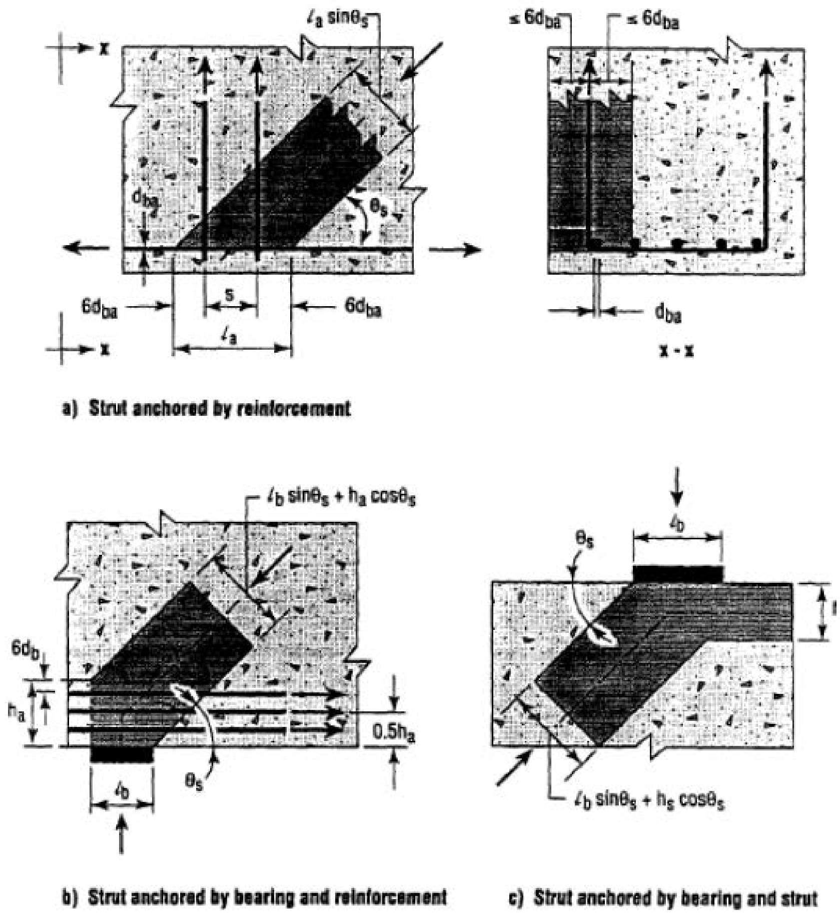


**Figure IE 6.1.5.2.1I.2-1 Influence of Anchorage Conditions on Effective Cross-Sectional Area of Strut**

STRENGTH REDUCTION FACTORS

The strength reduction factors (from AD 5.5.4.2) for strut-and-tie models shall be taken as follows:

Struts, $\phi = 0.70$

Ties, $\phi = 0.90$

Tie reinforcement must be fully developed at the inside corner of the node that the tie is connected to be completely effective.

STRENGTH OF TIES

The nominal strength of ties is given by:

$$P_n = f_y \cdot A_{st}$$

where: $f_y =$ yield strength of the tie reinforcement

$A_{st} =$ total area of the tie reinforcement considering the effects of deterioration

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PCI Journal, May/June 1987, Vol 32, No 3,

Mitchell, Collins, Bhide, Rabbat, (2004) AASHTO LRFD Strut-and-Tie Model Design Examples, PCA

Reineck et al., (2002) Examples for the Design of Structural Concrete with Strut-and-Tie Models, ACI SP-208.

Hsu, Thomas T. C. (1993), Unified Theory of Reinforced Concrete, CRC Press: Boca Raton, Florida.

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STRENGTH OF STRUTS

Strut capacities are determined by the type of node a strut is connected to, the extent, width, and orientation of cracks crossing the strut, and a minimum amount of confinement reinforcement must be present to control such cracking.

The nominal strength of struts is given by:

$$P_n = f_{ce} \cdot A_{cs} + f_y \cdot A_{ss}$$

where: $f_{ce}$ = effective concrete compressive strength

$A_{cs}$ = cross-sectional area of strut considering the effects of Deterioration

$f_y$ = yield strength of tie reinforcement

$A_{ss}$ = area of reinforcement in the strut

The effective concrete compressive strength shall be taken as:

<u>Struts</u>
- prismatic struts without cracks = 0.85 f'c
- struts across cracks of normal width, if confining reinforcement is provided = 0.70 f'c
- struts across cracks of normal width, without confining reinforcement = 0.55 f'c
- struts in tension members and across wide cracks = 0.35 f'c

<u>Nodes</u>
- joining three struts (CCC node) = 0.85 f'c
- anchoring one tie (CCT node) = 0.75 f'c
- anchoring more than one tie (CTT node) = 0.65 f'c

LIMITING COMPRESSIVE STRENGTH IN STRUT

The limiting compressive stress, $f_{cu}$ shall be taken as:

$$f_{cu} = \frac{f'_c}{0.8 + 170\,\varepsilon_1} \le 0.85 f'_c$$

In which:

$$\varepsilon_1 = \varepsilon_s + (\varepsilon_s + 0.002)\cot^2 \alpha_s$$

where:

$\alpha_s$ = the smallest angle between the compressive strut and adjoining tension ties (°)

$\varepsilon_s$ = the tensile strain in the concrete in the direction of the tension tie (in./in.)

$f'_c$ = specified compressive strength (ksi)

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The effective concrete compressive strengths by strut type are based on ACI 318 19, Chapter 23, adjusted for the difference in the strength reduction factors for struts compared to AD. Alternatively, AD 5.6.3.3 may be used for determination of the compressive strengths of struts. The effective strengths of struts by node type are in accordance with AD 5.6.3.5.

If the concrete is not subjected to principal tensile strains greater than about 0.002, it can resist a compressive stress of 0.85 f'c. This will be the limit for regions of the struts not crossed by or joined to tension ties. The reinforcing bars of a tension tie are bonded to the surrounding concrete. If the reinforcing bars are to yield in tension, there should be significant tensile strains imposed on the concrete. As these tensile strains increase, $f_{cu}$ decreases.

The expression for $\varepsilon_1$ is based on the assumption that the principal

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compressive strain $\varepsilon_2$ in the direction of the strut equals 0.002 and that the tensile strain in the direction of the tension tie equals $\varepsilon_s$. As the angle between the strut-and-tie decreases, $\varepsilon_1$ increases and hence $f_{cu}$ decreases. In the limit, no compressive stresses would be permitted in a strut that is superimposed on a tension tie, i.e., $\alpha_s$=0, a situation that violates compatibility.

For a tension tie consisting of reinforcing bars, $\varepsilon_s$ can be taken as 0.0 until the precompression of the concrete is overcome. For higher stresses, $\varepsilon_s$ would equal ($f_{ps}$-$f_{pe}$)/Ep.

If the strain $\varepsilon_s$ varies over the width of the strut, it is appropriate to use the value at the centerline of the strut.

MINIMUM CONFINEMENT REINFORCEMENT

The minimum amount of confinement reinforcement required to prevent cracking shall meet the following:

$$\sum_i \rho_i \cdot \sin \alpha_i \geq 0.003$$

where: $\rho_i = \dfrac{A_{si}}{b_s \cdot s_i}$ = reinforcement ratio in direction i

$A_{si}$ = reinforcement area in direction i
$s_i$ = reinforcement spacing perpendicular to direction i
$b_s$ = thickness of strut
$\alpha_i$ = angle between direction i and axis of strut

Minimum confinement reinforcement may be omitted only in massive concrete elements where reinforcing steel detailing has followed accepted practice or where uncontrolled cracking at a skew angle to the strut axis is unlikely. Refer to AD 5.8.2.6.

SERVICEABILITY

The strut-and-tie model developed for strength checks can be used to perform serviceability checks. Allowable stresses in reinforcing steel under service loads are listed in Table M 6B.5.2.3-1. For structures satisfying the strength requirements and minimum confinement reinforcement requirements stated above, allowable stresses under service loads may be increased to 60% and 75% of the yield strength at inventory and operating level, respectively, but not to exceed the values corresponding to Grade 60 reinforcing steel.

These minimum confinement reinforcement requirements follow those stipulated in ACI 318-05, which are more flexible than the corresponding specifications in AD. The strut-and-tie modeling approach relies on the capability of a reinforced concrete structure to redistribute internal forces with concrete cracking, yielding of reinforcement, and development of inelastic compressive strains in the concrete. This capability is greatly enhanced with the provision of minimum confinement reinforcement. Minimum confinement reinforcement is also essential to ensure ductile failure modes and to

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control cracking under service loads.

**6.1.5.3I** ADJACENT NON-COMPOSITE PRESTRESSED CONCRETE BOX BEAMS

Load ratings of adjacent non-composite prestressed concrete box beams with deterioration are to be based on one of the following procedures. The engineer shall determine the most suitable method for the bridge being analyzed. Method A is a conservative simplified method to determine the number of strands to remove for analysis. Method B is less conservative and more detailed than Method A and is based on Lehigh University's ATLSS 09-10 Study completed in 2010. Method C is special analysis of fascia beams required when the Capacity/Dead Load is < 1.5. The following procedures shall also be used to evaluate non-composite prestressed slab beam superstructures (plank beams). For an analysis example using Method A and Method B, see Appendix IE 04-D and the ATLSS 09-10 Study.

METHOD A – Simplified Method:

- Visually observed strands + 25% - Deduct 100% of all exposed strands plus an additional 25% (125% of the total area of the exposed strands) from capacity calculations. These strands shall be considered ineffective from the beam end to the location of the end of the spall. The strands shall be considered ineffective for the full beam length if the exposed strands are in the middle 1/3 of the span. Engineering judgement shall be used regarding the redevelopment of the strands.
- Strands adjacent to or intersecting a crack shall be considered ineffective in the region immediately adjacent to the crack.
- If significant strand loss is noted (>20%), especially for fascia beams, contact BDTD for further instructions.
- For beams with no exposed strands but which appear to have internal damage (as evidenced by bottom flange cracking with rust and/or delamination), contact BDTD for further instructions.
- For fascia beams with Capacity/Dead Load < 1.5, use Method C below.

IC6.1.5.3I Based on limited research of beams with longitudinal cracks in the bottom flange, the strand above the crack as well as the two adjacent lower layer strands may be deteriorating. For this condition, a parametric study of strand loss should be performed to determine the sensitivity of beam capacity to strand loss.

METHOD B – Refined analysis based on ATLSS 09-10 Study:

Longitudinal Cracking:

1. The following strand areas shall be reduced to 75% of the original cross-sectional area within the analysis window for capacity calculations:
   a) Strands on each level directly in line with the crack
   b) Strands closest to the exterior surface adjacent to the longitudinal crack. If the adjacent strand is greater than 3" from the crack, see the following item for area reduction.
2. For beams with longitudinal cracking or corrosion induced spalling, all other strands in the section shall be reduced to 95% of the original cross-sectional area within the analysis window for capacity calculations.

Deteriorated Concrete:

1. For exposed strands observed with sound concrete adjacent to and above the exposed strands, disregard the full strength of the exposed strands within the analysis window for capacity calculations.

According to ATLSS Report No. 09-10 performed by Lehigh University in 2010, the factors that affect the beam capacity are longitudinal cracking and deteriorated concrete (including spalling and exposed reinforcement). These defects shall be accounted for in evaluation of the beam member.

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2. For exposed strands observed with adjacent unsound concrete, disregard the full strength of the exposed strands and all strands in regions of unsound concrete within the analysis window for capacity calculations.
3. For exposed shear reinforcement bars, disregard the full strength of strands located in the lower row directly above the exposed section of stirrups for capacity calculations. If the concrete is found to be unsound adjacent to the exposed area, disregard the strength of all strands in all rows above the area of unsound concrete within the analysis window for capacity calculations.
4. For area of concrete where delaminations have been observed, remove all delaminated concrete to determine the depth of the concrete deterioration:
    a) If shear reinforcement bars or strands are exposed, treat as in cases "1" through "3" as shown above.
    b) If no shear reinforcement bars or strands are exposed but there are indications that the exposed concrete is unsound within the effected area, disregard the strength of all strands located in the rows of strands above the area within the analysis window for capacity calculations.
    c) If no steel reinforcement is exposed in the affected area and the concrete is deemed as sound, do not disregard the strength of strands within the analysis window in capacity calculations.
5. For wet or stained areas of concrete observed on the bottom or side of beams, closely inspect those areas to determine the soundness of concrete:
    a) If close inspection indicates that the concrete is unsound or delaminated, treat as in case "4" above.
    b) If close inspection confirms that the concrete is sound, do not disregard the strength of strands within the analysis window in the capacity calculations.
6. For fascia beams with Capacity/Dead Load < 1.5, use Method C below.

For the purpose of load rating, all damage within a region of two development lengths shall be considered to occur at the same analysis window. The computed development length can be used; however, if design information is unavailable the lengths presented in Table IE 6.1.5.3I-1 can be used for typical seven wire strands:

**Table IE 6.1.5.3I-1: Analysis Window Length Based on Strand Diameter**

| Strand Nominal Diameter (in) | Analysis Window Length (in) |
|---|---|
| 3/8 | 128 |
| 7/16 | 150 |
| 1/2 | 170 |
| ½ Special | 180 |

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METHOD C – Special Analysis of Fascia Beams

- Because the live load portion of the total load carried by fascia beams is small, the load rating may be > 1.0 and not reflect the marginal capacity above dead load. Thus, when Capacity/Dead Load is < 1.5, a more detailed analysis is required. For fascia beams with Capacity/Dead Load < 1.5 or an Operating Rating < 1.5 based on a conventional analysis, an analysis that considers biaxial stresses will be performed by BDTD.

### 6.1.6 Evaluation of Complex Structures

*The following shall supplement M 6.1.6.*

Additional requirements for PA bridges are contained in IP 3.3.3. and IP 3.3.4.

### 6.1.9 Documentation of Rating

*The following shall supplement M 6.1.9.*

Additional Requirements for PA bridges are contained in IP 8.3.2.

## PART A – LOAD AND RESISTANCE FACTOR RATING

### 6A.1 INTRODUCTION

*The following shall supplement M 6A.1.*

LRFR method as outlined in the MBE Section 6A may be used on PA bridges when it is warranted which is a determination made in conversation with the Bridge Inspection Section. See IP 3.6 for guidance on the use of LRFR ratings.

## PART B – ALLOWABLE STRESS RATING AND LOAD FACTOR RATING

### 6B.1 GENERAL

*The following shall supplement M 6B.1.*

Additional requirements for PA bridges are contained in IP 3.

### 6B.3 RATING METHODS

*The following shall supplement M 6B.3.*

For live load capacity rating methods for PA bridges see IP 3.6.

### 6B.5 NOMINAL CAPACITY: C

### 6B.5.2 Allowable Stress Method

#### 6B.5.2.1 STRUCTURAL STEEL

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*The following shall replace the first sentence of M 6B.5.2.1.*

The allowable unit stresses used for determining the Inventory Ratings and Operating Ratings depend on the type of steel used in the structural members.

*The following shall replace the formula for compression in concentrically loaded columns in Table M 6B.5.2.1.*

With $C_c = \sqrt{2\pi^2 E/Fy}$

$F_a = F_y/F.S.\left[1 - \left[\left[ (KL/r)^2\ F_y\right] / 4\pi^2\ E\right]\right]$ when $KL/r \leq C_c$

$F_a = \pi^2 E / \left[F.S.(KL/r)^2\right] = 168{,}363{,}840 / (KL/r)^2$ when $KL/r > C_c$

With F.S. = 1.70

**6B.5.2.1.1** Combined Stresses

*The following shall supplement M 6B.5.2.1.1.*

When performing compression member analyses, and in particular of lightly constructed, built-up compression truss member end posts that are unsymmetrical, the pin eccentricity (i.e. the distance between the centerline of pin and the neutral axis of member) must be evaluated.

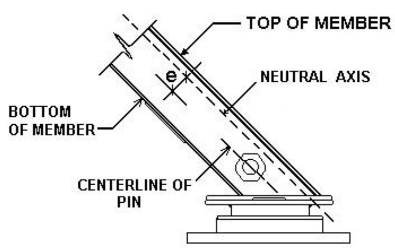


**Figure IE 6B.5.2.1.1-1**

**6B.5.2.1.3I** Gusset Plates in Truss Bridges

See MBE Appendix L6A for analysis of gusset plates in truss bridges.

**6B.6 LOADINGS**

*The following shall supplement M 6B.6.*

Additional requirements for PA bridges are contained in IP 3.2.

**6B.6.1 Dead Load: D**

*The following shall supplement the second paragraph of M 6B.6.1.*

COMMENTARY

IC6B5.2.1 Safe Load Capacity is discussed under bridge postings, IP 4.3.2.

IC6B5.2.1.1 For the case shown in Figure IE 6B.5.2.1.1-1 the bottom flange is more critical, therefore $f_{bx} = Pe/S_{bottom}$ when determining combined stresses as per M 6B.5.2.1.1.

In the event of significant damage (buckled lacing and/or sheared rivets/bolts caused by impact) to the end posts of the truss, considerations shall be made to reduce the members capacity. The amount of reduction should be evaluated on a case by case basis, utilizing engineering judgment based on the type and amount of damage to the member element.

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For encased I-beam (EIB) bridge analyses, the following criteria will determine whether the composite or non-composite section carries the superimposed dead load and live load if plans and/or details are unavailable to verify the design:

- The concrete encased I-girder may be considered composite for live load (bridge type EIB in BAR7) if the conditions below are met:
  - There is no distress noted between the concrete encasement around the beams and the deck indicating a loss of composite action.
  - The superstructure and deck ratings are both greater than 4.
- If the conditions above are not met, the bridge must be modeled as non-composite for live load (bridge type GGG in BAR7).
- Concrete encased I-girder bridges may have been constructed with shored or unshored formwork. For shored construction the DL1 loads are applied to the composite section properties, as the shoring would remain until the concrete sets.

*The following shall supplement M 6B.6.1.*

For adjacent non-composite prestressed concrete box beams, the following criteria shall be used to determine the distribution of barrier dead loads:

- Assume fascia beams support 100% of the barrier dead load.
- Assume the first interior beams support 50% of the barrier dead load.

**6B.6.2 Rating Live Load**

*The following shall supplement M 6B.6.2.*

Additional requirements for PA bridges are contained in IP 3.2.2.

**6B.6.3 Distribution of Loads**

*The following shall supplement M 6B.6.3.*

For adjacent non-composite prestressed concrete box beams, the following criteria shall be used to determine the distribution of live loads for moment and shear:

- Fascia girder shall use the larger of the LFD Distribution Factor (IP 3.3.2.2) or Lever Rule (AD 4.6.2.2).
- Interior girder shall use a wheel load distribution factor = 1.0 where there is a loss of grout in the shear key and/or tie rod.

**6B.6.7 Environmental Loads**

**6B.6.7.4** STREAM FLOW

*The following shall replace the second sentence of M 6B.6.7.4.*

However, remedial action should be considered if these forces are especially critical to the structure’s stability.

**6B.6.7.5** ICE PRESSURE

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*The following shall replace the second sentence of M 6B.6.7.5.*

If these forces are especially important, then corrective action should be recommended.

### 6B.7 POSTING OF BRIDGES

*The following shall replace M 6B.7.*

Requirements for posting of PA Bridges are contained in IP 4.

#### 6B.7.1 General

*Delete the third paragraph of M 6B.7.1.*

### 6B.8 PERMITS

*The following shall replace M 6B.8.*

Requirements for PA permitting are contained in IP 10.

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*The following shall replace M 7.*

## 7.1I GENERAL

For PA bridges see DM4 PP 5.1 and PD 6.6.1.

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*The following shall replace M 8.*

## 8.1I BRIDGE INSTRUMENTATION INTRODUCTION

The purpose of instrumentation and monitoring of a bridge is to diagnose a known deficiency and to develop retrofit details to correct the deficiency. The diagnostic testing shall include both the static and dynamic response of the bridge. The diagnostic testing must be implemented in accordance with a pre-defined test program including objectives and a detailed instrumentation plan. Typically, instrumentation and monitoring has been used to perform fatigue evaluation of steel bridges for the purpose of estimating remaining fatigue life and developing retrofit concepts.

The instrumentation and monitoring of a bridge shall not be used to determine the load carrying capacity or load posting unless otherwise approved by the Chief Bridge Engineer. The bridge load carrying capacity predicted from field load testing is affected by several factors such as:

- unintended composite action
- unintended continuity or bearing fixity
- participation of secondary members
- participation of nonstructural members
- portion of load carried by deck

These factors tend to influence test results by indicating lower live load stresses than calculated stresses, and thus predict a higher load carrying capacity at service load levels. The enhanced behavior due to unintended participation may not be present at load levels higher than the test load. At the higher load levels, the loss of unintended participation will result in increased stresses in main members. Therefore, the extrapolation of test results to determine the load carrying capacity at load levels in excess of the tested live loads is not permitted.

## 8.2I CONTRACTING PROCEDURES

The instrumentation and monitoring of bridges is a professional service. Companies that intend to provide these services must be a registered business partner with a relationship type classification of consultant in the Department's Engineering and Construction Management System (ECMS). In addition to registering as a business partner, the company must have an active consultant qualification package, an approved overhead rate, and an employee roster in ECMS.

Companies intending to provide testing services must have a demonstrated record of performing such work, including both the physical installation of monitoring systems as well as the interpretation of the data.

Typically, this service is one of many tasks within a contract, and therefore bridge testing service is performed by a sub-consultant on behalf of the prime consultant.

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## 8.3I INSTRUMENTATION

The instrumentation used for diagnostic bridge testing may include:

- Strain gages or other types of strain transducers
- Displacement transducers
- Rotation gages
- Accelerometers

The instrumentation must be connected to a data acquisition system that is capable of capturing and storing response data for processing of the results to fulfill the objectives of the testing program. Specifically, data must be sampled at a rate high enough to ensure that the peak response of a given bridge element is captured. Moreover, the data acquisition system must have sufficiently high resolution to ensure that the complete response of a given bridge element is measured.

For fatigue evaluations, the data acquisition must be capable of performing cycle counting using the rainflow cycle-counting algorithm. This algorithm is used to develop the measured stress-range histograms at strain gauge locations.

## 8.4I DIAGNOSTIC TEST PROGRAM

A diagnostic testing program shall be developed prior to the field instrumentation and monitoring of a bridge. The program shall include the test objectives, level of structural analysis to be performed prior to and after field testing, types and layout of instrumentation, the test load to be used, field testing procedures, expected duration and requirements for maintenance and protection of traffic, as well as evaluation methodology of test data and reporting.

The test load may consist of controlled vehicles of known weight and/or the regular traffic on the bridge. Using controlled vehicles of known weight traveling at known locations provides data to correlate the responses of all sensors with a known load. The controlled test, when running test vehicles along the same positions at crawl and full speeds, also allows assessing the effects of dynamic impact.

## 8.5I EVALUATION OF INSTRUMENTATION RESULTS

The test results shall be evaluated in such a manner to satisfy the objectives set in the test program. The results of the instrumentation should be compared to analytical predictions to demonstrate the validity of the tests. A comparison of static and dynamic measurements shall be performed to assess the dynamic amplification effects. In addition, a determination of live load distribution factor shall be performed. The calculated live load distribution shall be compared to values predicted from the AASHTO LRFD Bridge Design Specifications.

For fatigue testing results, measured stress range histograms shall be reduced and compared to the S-N fatigue curves in the AASHTO-LRFD Bridge Design Specifications.

SPECIFICATIONS

## 8.6I REPORTING

A comprehensive report shall be prepared describing the general features of the bridge, the objectives of the testing program, description of the testing procedures, instrumentation plan, instrumentation results, interpretation of the instrumentation results, and comparison of results with numerical analysis, as well as the evaluation of the results per Section 5.5I for dynamic amplification effects and live load distribution. The objective of the instrumentation and monitoring is to correct a known deficiency, thus the report must include recommendations and retrofit concepts.

The report shall include an executive summary, photographs of the instrumentation, and graphical and tabular presentation of results. The report shall be signed and sealed by a Professional Engineer Registered in the Commonwealth of Pennsylvania.

## 8.7I INSTRUMENTATION SYSTEMS

The most common method of determining the behavior of a bridge is through instrumentation with strain gages/transducers and displacement measuring devices. The use of strain and displacement measuring devices has proven history of providing reliable results that fulfill the objective of most bridge testing programs.

As technology continues to evolve, additional types of sensors, instrumentation devices, or wireless or remote monitoring systems (both in terms of sensors and data acquisition systems) have been developed or are still being developed. New instrumentation systems that have not been used by the Department shall be evaluated for proof of concept. The proof of concept evaluation may occur by:

- Concerted research project conducted by the Department

Success performance and favorable evaluation of the system by another Department of Transportation or governmental agency

COMMENTARY

IC5.7I Two different monitoring sensors have been tested through Research Project No.: 3900017209, titled "Remote Health Monitoring and Load Modeling of Cracked Fracture Critical Bridge Components". LifeSpan Technologies' Model LST Structural Health Sensor (LST) and Matech Material Technologies' Electrochemical Fatigue Sensor (EFS) systems.

A third project, Contract No 4400017287, 3515R08 "Instrumentation & Monitoring of PA Bridges." The contract duration is 2017 thru 2021 and the instrumentation that is being evaluated under this contract is by Resensys.

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# APPENDIX IE 04-A

Truss Inspection
Findings Summary

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**Truss Inspection Findings Summary**

BMS ID: ____________________ Checked By: ____________________

Insp. Date: ____________________ Date: ____________________

Insp. By: ____________________

## Truss Inspection Findings Summary
## Truss Members and Floorbeam Ends

| Span | Truss Line | Member | FCM (Y/N) | Member Cond. Rating (0-9) | Crack? (Y/N) | Crack Retrofitted (Y/N) | Section Loss (Y/N) | Tack Welds Present (Y/N) | Tack Welds Cracked (Y/N) | NDE Test Recommended (Y/N) | Photo No.'s | Sketch No.'s | Comments |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
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BMS ID: ______________________ Checked By: ______________________

Insp. Date: ______________________ Date: ______________________

Insp. By: ______________________

Truss Inspection Findings Summary
Panel Point Connection Condition
(includes truss member and bracing connections)

| Panel Point | Truss Line | Section Loss (Y/N) | Weld Cracks (Y/N/NA) | Crack Retrofit (Y/N/NA) | NDE Test Recommended (Y/N) | Missing Bolts (Y/N/NA) | Loose Bolts (Y/N/NA) | Photo No.'s | Comments |
|---|---|---|---|---|---|---|---|---|---|
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BMS ID: ______________________ Checked By: ______________________

Insp. Date: ______________________ Date: ______________________

Insp. By: ______________________

| Truss Inspection Findings Summary<br>Bearing Condition | | | | | | | |
|---|---|---|---|---|---|---|---|
| Sub Unit | Truss Line | Type | Fixed/Exp (F or E) | Frozen (Y/N/NA) | Over Extended (Y/N/NA) | Photo No.'s | Comments |
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# APPENDIX IE 04-B

Rocker Bearing Movement Analysis
Guidelines and Procedures

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# Rocker Bearing Movement Analysis Guidelines and Procedures

The following documentation is required for each location where rocker bearings are present. This information shall be collected from the most distressed bearing on each bearing line at a substructure unit for every inspection cycle. In the case of a single pier with two lines of expansion rocker type bearings supporting two different spans, two worst case bearings will be recorded, one for each bearing line. For bearing lines where more than one bearing appears to be distressed, also record information/documentation for these bearings.

The following procedure shall be used to assess the condition and movement of rocker bearings:

1. Enter the input data on the Rocker Bearing Movement Analysis spreadsheet (Location data, substructure type (Abutment or Pier), radius of rocker plate(R), width of rocker plate(D), and expansion Length(L) from existing plans or field measurements. See Figure 4-B-2 for a sample of the spreadsheet. The spreadsheet will calculate the allowable tilt limits in as outlined in Table 4-B-1. Complete this procedure prior to the inspection.
2. During the inspection, measure the ambient temperature, angle of rotation (θ), and minimum clearance between girders or from girder to abutment(X). Also, confirm the radius of rocker plate(R) and width of rocker plate(D).
3. Compare the field measured rocker tilt to the table of allowable tilt range to determine if a Priority 0 or 1 maintenance item is required based on the current ambient temperature.
4. Input the rotation measurements into the spreadsheet. The spreadsheet will determine the appropriate maintenance priority based on tilt measurement only, in accordance with Pub 100A, based on the current measurements at the ambient temperature. Consider adjusting maintenance priority based on ADT of the bridge, feature under the bridge, flexibility of the substructure, condition of the bearings, number of bearings in a line with excessive tilt, type of superstructure, structural redundancy, ability of the bearing to freely move, and evidence of pintel failure.
5. Upload the final spreadsheet to Inventory section under Documents in BMS2.
6. Attach a printout of the Rocker Bearing Movement Analysis to the current inspection report.

In addition to the procedure above, the inspection report shall include the following guidance for reporting rocker bearing condition and functionality:

- Use language, “expanded” or “contracted”, do not use terms such as “back” or “North”.
- Note whether the bearing shows signs of movement (indicated by cracks in the paint at locations of intended movement between bearing components, polished metal at bearing surfaces, or light colored rust powder created by movement) or if the bearing has sufficient capacity to move further in the direction of travel under temperature extremes.
- Presence of corrosion, including pack rust or any debris under the rocker that could inhibit proper bearing function and freedom of movement.
- Flattened bearing surfaces.
- Section loss to any portion of the bearing assembly including anchor bolts.
- Location of sheared off or bent anchor bolts where applicable.
- Evidence of sheared pintels.
- Evidence of masonry plate movement.
- Cracks in concrete seats.
- Abnormal or unusual gap measurements at deck expansion joints.

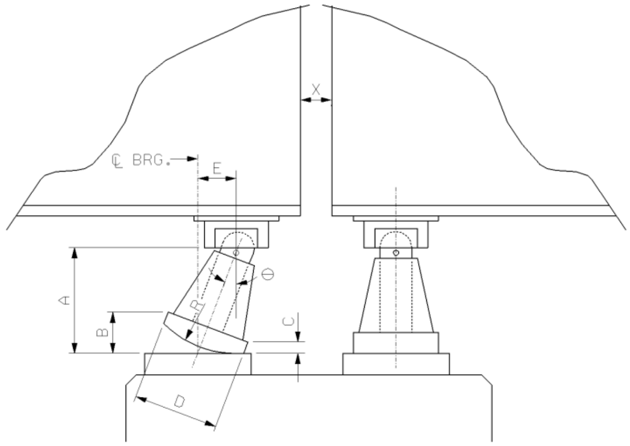



**Figure 4-B-1 Rocker Bearing Field Documentation Reference Sketch**

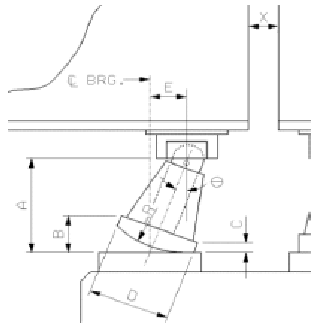
Notation:

A = as-inspected height of rocker (in.)
B = high corner of rocker plate (in.)
C = low corner of rocker plate (in.)
D = width of rocker plate (in.)
E = longitudinal translation (in.)
R = radius of rocker plate (in.)
X = minimum clear distance between girders or from girder to abutment (in.)
θ = angle of rotation (tilt) (degrees)

**Figure 4-B-2 Sample Rocker Bearing Movement Analysis**
This spreadsheet can be downloaded from the BMS2 Home Screen

## ROCKER BEARING TILT DIMENSION DATA

| | | | | |
|---|---|---|---|---|
| Date: | 4/18/2019 | | Inspectors: | |
| Ambient Temperature: | 58 | °F | BMS #: | XX-XXXX-XXXX-XXXX |
| Expansion Length (L): | 230 | ft | BRKEY: | XXXXX |
| Radius (R): | 10 | in | Bridge Location: | SR xx over Unkown Creek |
| Plate Width (D): | 7 | in | | |
| Coeff. of Thermal Exp.: | 6.50E-06 | per °F | Rocker Line: | P02a |
| Min Clearance (X): | 2 | per °F | Support Type: | Pier |




| Rocker Line: | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Beam: | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| $\theta_{Field\ Measured}$: | -1.7 | -11.0 | -4.0 | -3.0 | -1.2 | -0.4 | -0.2 | 0.0 | 1.0 | 4.0 | 5.0 | 6.0 |
| Temperature: | 58 | 58 | 58 | 58 | 58 | 58 | 58 | 58 | 58 | 58 | 58 | 58 |
| $E_{From,68°F}$: | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 | -0.18 |
| $\theta_{From,68°F}$: | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 | -1.00 |
| E min: | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 | -0.53 |
| θ min: | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 | -3.0 |
| E max: | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 | 0.82 |
| θ max: | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 |
| Capacity: | 35% | N/A | 150% | 100% | 10% | 11% | 14% | 18% | 35% | 88% | 105% | 123% |
| Priority: | OK | PR0 | PR1 | PR3 | OK | OK | OK | OK | PR3 | PR3 | PR1 | PR1 |
| Comments: | Provide notes as need (i.e. justification of maintenance priority or reason for priority different from above, pack rust, debris restriction, frozen bearings, evidence of pintel failure, etc.) | | | | | | | | | | | |

## Past Rocker Bearing Tilt Dimension Data

| | Rocker Line: | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a | P02a |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Beam: | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 |
| Date | °F | | | | | | | | | | | | |
| 4/1/2013 | 35 | 0.6 | 0 | -2 | -1 | -0.8 | -0.9 | 6 | 1 | 4.7 | 3.1 | 2.6 | 3.4 |
| 4/27/2015 | 55 | -1.8 | -1.9 | -0.7 | -1.5 | -1.8 | -1.3 | -1.1 | -1.3 | -0.4 | -0.4 | -1.3 | -1 |
| 4/1/2017 | 62 | -1.2 | -1.5 | 1.1 | -1.5 | -1.9 | -1.2 | -0.8 | -1.3 | 2 | -1.7 | -1.3 | -0.8 |
| 4/18/2019 | 58 | -1.7 | -7.5 | -4.2 | -3 | -1.2 | | | | | | | |
| 4/19/2019 | 72 | | | | | | -0.4 | -0.2 | 0 | 0.1 | 4 | 7.1 | 7.2 |
| | | | | | | | | | | | | | |
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**Table 4-B-1 Acceptable Range of Tilt Angles, $\theta_{min}$ and $\theta_{max}$**

Table to be completed prior to inspection using procedure described herein. The Rocker Bearing Movement Analysis spreadsheet can be utilized to complete this table (The spreadsheet is available for download from the BMS2 Home screen). This table provides the maximum and minimum allowable tilts for any given ambient temperature. The inspector should use these tilt angle ranges in the field as a guide to determine if the bearing tilt is within acceptable limits. The acceptable limit of tilt at piers is the outer one-quarter point of the rocker plate. The acceptable limit of tilt at abutments is the outer one-tenth point of the rocker plate. If design plan bearing dimensions are used to generate table, they should be verified by field measurement.

| Temp. (°F) | -10 | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 68 | 70 | 80 | 90 | 100 | 110 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\theta_{min}$ (deg) | | | | | | | | | | | | | | |
| $\theta_{max}$ (deg) | | | | | | | | | | | | | | |
| $E_{min}$ | | | | | | | | | | | | | | |
| $E_{max}$ | | | | | | | | | | | | | | |

Notes:

1) A negative tilt angle denotes contraction (movement towards the fixed bearing) and a positive tilt angle denotes expansion (movement away from the fixed bearing).
2) The longitudinal bridge movement described herein includes thermal movement only.

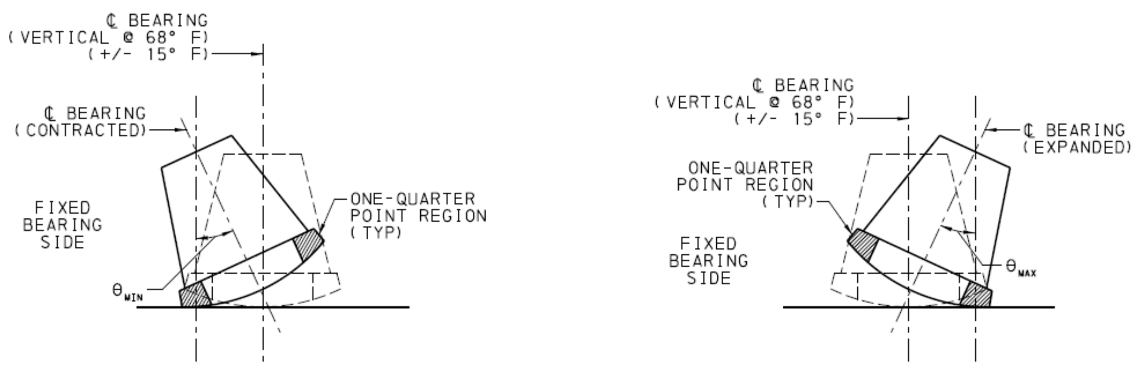


**Figure 4-B-3 Rocker Bearing Tilt Angle Reference Sketch**

**Procedure to calculate $\theta_{min}$ and $\theta_{max}$**

The following procedure is used to determine the acceptable tilt angle range ($\theta_{min}$ to $\theta_{max}$) in Table 4-B-1. Either the one-quarter or the one-tenth point can be determined with this methodology.

Required information:

rocker plate width, D (in.)
rocker plate radius, R (in.)
coefficient of thermal expansion, $\alpha$ ($6.5 \times 10^{-6}$/ºF for steel)
expansion length, L (ft.)

Step 1: Determine the one-quarter point dimension for rocker bearings at piers, 0.25D; this is the maximum allowable movement in one direction. Determine the one-tenth point dimension for rocker bearings at an abutment, 0.4D; this is the maximum allowable movement in one direction.

Step 2: Calculate the movement increment for the 10° F intervals shown in Table 4-B-1.

Step 3: For $\theta_{min}$, set the maximum allowable thermal movement to -0.25D (-0.4D) (contracted) at -10° F and add the movement increment calculated in Step 2 at each 10° F increment to obtain the adjusted thermal movement, $d_{adj}$, for $\theta_{min}$. Contraction movement is shown as a negative value.

Step 4: For $\theta_{max}$, set the maximum allowable thermal movement to 0.25D (0.4D) (expanded) at 110° F and subtract the movement increment calculated in Step 2 at each 10° F increment to obtain the adjusted thermal movement, $d_{adj}$, for $\theta_{max}$. Expansion movement is shown as a positive value.

Step 5: Determine the subsequent tilt angles, $\theta_{min}$ and $\theta_{max}$, for use in Table 4-B-1.

$$\theta_{min/max} = \sin^{-1}[d_{adj} / \mathrm{SQRT}(R^2 + d_{adj}^2)]\ (180 / \pi)$$

Note: if $d_{adj}$ is the contracted movement, then $\theta$ is taken as the negative of this expression in order to be consistent with the sign convention described in Steps 3 and 4. No change in sign is required if $d_{adj}$ is the expanded movement.

**Sample calculation for $\theta_{min}$ and $\theta_{max}$**

Required information:

D = 7 in.
R = 10 in.
$\alpha = 6.5 \times 10^{-6}$ /°F (for steel)
L = 230 ft.

Step1 (Calculation of maximum allowable movement):
Determine the one-quarter (one-tenth) point dimension. The maximum allowable movement in either the expanded or contracted direction is taken equal to one-quarter of the rocker plate width.

0.25D = 1.75 in. (0.4D = 2.80 in.)

Step 2 (Calculate the movement increment):

E = Structure thermal movement (in.) Note: contraction movement is taken as negative and expansion movement is taken as positive.

= $12\alpha L\Delta_t$

where:

$\Delta_t$ = Temperature change with respect to 68° F

and

$d_{inc} = E_{Thigh} - E_{Tlow}$ = movement increment (in.)

| | Str. Thermal Movement | Movement Increment ($d_{inc}$) | |
|---|---|---|---|
| At -10° F | $E_{From,68^oF}$ = -1.40 in. | | |
| At 0° F | $E_{From,68^oF}$ = -1.22 in. | $d_0 - d_{-10}$ = | 0.18 in. |
| At 10° F | $E_{From,68^oF}$ = -1.04 in. | $d_{10} - d_0$ = | 0.18 in. |
| | etc. | | etc. |

The only non-uniform increment will occur at the 68° F interval.

| | Thermal Movement | | Movement Increment ($d_{inc}$) |
|---|---|---|---|
| At 60° F | $d_{60}$ = | -0.14 in. | |
| At 68° F | $d_{68}$ = | 0 in. | $d_{68} - d_{60}$ = 0.14 in. |
| At 70° F | $d_{70}$ = | 0.04 in. | $d_{70} - d_{68}$ = 0.04 in. |

Step 3 (Calculation of $E_{min}$):

For $\theta_{min}$, set the movement at -10° F equal to -0.25D (-0.4D). Add the movement increment($d_{inc}$) to calculate $E_{min}$ for the next temperature increment.

| | | |
|---|---|---|
| At -10° F | Set -> | $E_{min}$ = -1.75 in. (-2.80 in.) |
| At 0° F | | $E_{min}$ = -1.75 + 0.18 = -1.57 in. (-2.80 + 0.18 = -2.62 in.) |
| At 10° F | | $E_{min}$ = -1.57 + 0.18 = -1.39 in. (-2.62 + 0.18 = -2.44 in.) |
| : | | |
| : | | |
| At 110° F | | $E_{min}$ = 0.22 + 0.18 = 0.40 in. (-0.82 + 0.18 = -0.64 in.) |

Step 4 (Calculation of $E_{max}$):

For $\theta_{max}$, set the movement at 110° F equal to 0.25D (0.4D). Subtract the movement increment($d_{inc}$) to calculate $E_{min}$ for the next temperature increment.

| | | |
|---|---|---|
| At -10° F | | $E_{max}$ = -0.22 – 0.18 = -0.40 in. ( 0.82 - 0.18 = 0.64) |
| : | | |
| : | | |
| At 90° F | | $E_{max}$ = 1.57 – 0.18 = 1.39 in. (2.62 – 0.18 = 2.44 in.) |
| At 100° F | | $E_{max}$ = 1.75 – 0.18 = 1.57 in. (2.80 – 0.18 = 2.62 in.) |
| At 110° F | Set -> | $E_{max}$ = 1.75 in. (2.80 in.) |

Step 5 (Calculation of $\theta_{min/max}$):

The tilt angles are determined using the adjusted thermal movement and rocker plate radius. Round the tilt angle to the next largest integer.

$$\theta_{min/max} = \sin^{-1}[E_{min/max} / SQRT(R^2 + E_{min/max}^2)]\ (180 / \pi)$$

Minimum Tilt Angle ($\theta_{min}$)

At -10° F $\theta_{min}$ = -10 deg (-16 deg)
At 0° F $\theta_{min}$ = -9 deg (-15 deg)
At 10° F $\theta_{min}$ = -8 deg (-14 deg)
:
:
At 110° F $\theta_{min}$ = 3 deg (-4 deg)

Maximum Tilt Angle ($\theta_{max}$)

At -10° F $\theta_{max}$ = -3 deg (4 deg)
:
:
At 90° F $\theta_{max}$ = 8 deg (14 deg)
At 100° F $\theta_{max}$ = 9 deg (15 deg)
At 110° F $\theta_{max}$ = 10 deg (16 deg)

Therefore, Table 4-B-1 can be completed with the calculated values for the given bearing information. This table provides the maximum and minimum allowable tilts for any given ambient temperature for an observed rocker bearing. The inspector should use these tilt angle ranges as a guide to determine if the bearing tilt is within acceptable limits (i.e. at no point during thermal expansion and contraction of the bridge will the rocker bear on the outer one-quarter point of the rocker plate at a Pier or the outer one-tenth point of the rocker plate at an Abutment).

| Temp. (°F) | -10 | 0 | 10 | 20 | 30 | 40 | 50 | 60 | 68 | 70 | 80 | 90 | 100 | 110 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\theta_{min}$ (deg) | -10 | -9 | -8 | -7 | -6 | -5 | -4 | -3 | -3 | -2 | -1 | 1 | 2 | 3 |
| $\theta_{max}$ (deg) | -3 | -2 | -1 | 1 | 2 | 3 | 4 | 5 | 6 | 6 | 7 | 8 | 9 | 10 |

Notes:

1) A negative tilt angle denotes contraction (movement towards the fixed bearing), a positive tilt angle denotes expansion (movement away from the fixed bearing).
2) The longitudinal bridge movement described herein includes thermal movement only.
3) Equation assumes lower rocker bearing surface is circular with its center located at the center of the upper semicircular bearing surface.
4) The table can also be completed with the $E_{min}$ and $E_{max}$ rows.

**Procedure to calculate the Capacity of a Rocker Bearing**

The following procedure is used to determine the Capacity of a rocker bearing (% of maximum capacity based on the current measurement at ambient temperature).

Capacity (when contracted from neutral) = $[\theta_{\text{Field Measured}} - \theta_{\text{From,68°F}}] / [\theta_{min} - \theta_{\text{From,68°F}}]$

Capacity (when expanded from neutral) = $[\theta_{\text{Field Measured}} - \theta_{\text{From,68°F}}] / [\theta_{max} - \theta_{\text{From,68°F}}]$

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# APPENDIX IE 04-C

Adjacent Non-Composite P/S Concrete
Box Beam Inspection Forms

ADJACENT NON-COMPOSITE P/S BOX BEAM INSPECTION FORM

District: ________________ Inspected by: ________________

BMS #: ________________________________ Inspection Date: ________________

6B40 **Wearing Surface Condition:** ☐

5B02 - 5B04 **Wearing Surface Type:** ☐☐☐

1A04 **Superstructure Condition:** ☐

## STRAND CONDITION DATA:

# of pre-stressing strands that are exposed:____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________

# of pre-stressing strands that are cut: ____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________
____ Beam #____ Span #____ Location* ________________

*Location of exposed/cut strands is the distance from centerline of near bearing to c.g. of cut/exposed strands

Strand Notes: ________________________________________________________________

ADJACENT NON-COMPOSITE P/S BOX BEAM INSPECTION FORM

District: ______________ Inspected by: ________________

BMS #: ______________________________ Inspection Date: ________________

## BEAM CONDITION DATA:

Is moisture or moisture staining present at longitudinal beam joints? Yes ☐ No ☐
If yes, describe the location including which beams and span #:

________________________________________
________________________________________
________________________________________

Transverse Cracks: Yes ☐ No ☐
If yes, describe the quantity, location (including beam # and span #), size, and any evidence of rust staining:

________________________________________
________________________________________
________________________________________

Longitudinal Cracks Yes ☐ No ☐
If yes, describe the quantity, location (including beam # and span #), size, and any evidence of rust staining:

________________________________________
________________________________________
________________________________________

Spalls: Yes ☐ No ☐
If yes, describe the quantity, location (including beam # and span #), size, and any evidence of rust staining:

________________________________________
________________________________________
________________________________________

Delaminations: Yes ☐ No ☐
If yes, describe the quantity, location (including beam # and span #), size, and any evidence of rust staining:

________________________________________
________________________________________
________________________________________

Post Tensioning: Yes ☐ No ☐
If yes, describe the quantity, location, and condition (rust staining, grout, and effectiveness):

________________________________________
________________________________________
________________________________________

Evidence of Shear Key Failure: Yes ☐ No ☐
If yes, describe the condition (including leaking at joints, reflective cracking in the wearing surface above the beam joints, and relative displacement between beams.):

________________________________________
________________________________________
________________________________________

# ADJACENT NON-COMPOSITE P/S BOX BEAM INSPECTION FORM

District: ______________ Inspected by: ______________

BMS #: ______________________________ Inspection Date: ______________

Provide sketch of beam(s) and deficiencies (leakage, efflorescence, cracks, delamination, spalls, exposed and broken P/S strands):

Span: ___

NEAR SUBSTRUCTURE | FAR SUBSTRUCTURE

| BEAM | 1/3 Pt → | 2/3 Pt → | |
|---|---|---|---|
| #1 | | | |
| #2 | | | |
| #3 | | | |
| #4 | | | |
| #5 | | | |
| #6 | | | |
| #7 | | | |
| #8 | | | |
| #9 | | | |
| #10 | | | |
| #11 | | | |
| #12 | | | |

ADJACENT NON-COMPOSITE P/S BOX BEAM INSPECTION FORM

District: ______________ Inspected by: ______________

BMS #: ______________________________ Inspection Date: ______________

## PARAPET DATA:

Left Parapet

Does the parapet have an open joint? Yes ☐ No ☐

Do any parapet joints exhibit any signs of leakage? Yes ☐ No ☐

Does the beam around the joint exhibit cracking? Yes ☐ No ☐

What is the distance between the open parapet joint and the nearest spall in the beam? ______________

Describe the locations and conditions in the notes below:

Right Parapet

Does the parapet have an open joint? Yes ☐ No ☐

Do any parapet joints exhibit any signs of leakage? Yes ☐ No ☐

Does the beam around the joint exhibit cracking? Yes ☐ No ☐

What is the distance between the open parapet joint and the nearest spall in the beam? ______________

Describe the locations and conditions in the notes below:

ADJACENT NON-COMPOSITE P/S BOX BEAM INSPECTION FORM

District: ______________ Inspected by: ______________

BMS #: ______________ Inspection Date: ______________

SUPPLEMENTAL INSPECTION NOTES:

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# APPENDIX IE 04-D

Adjacent Non-Composite P/S Concrete
Box Beam Analysis Example

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A prestressed concrete box beam section is illustrated in Figure 1. The damage within a region of analysis window (as determined by Table 6B.5.3.3.1-1I) is included in the section image and occurs within the middle 1/3 of the span. Field inspection of the beam identified three longitudinal cracks, spalling, and an area of unsound concrete. The construction documentation indicates that the beam is reinforced with 36 – 3/8 in. diameter seven-wire grade 270 prestressing strands. The spacing and arrangement of the strands is shown in Figure 1.

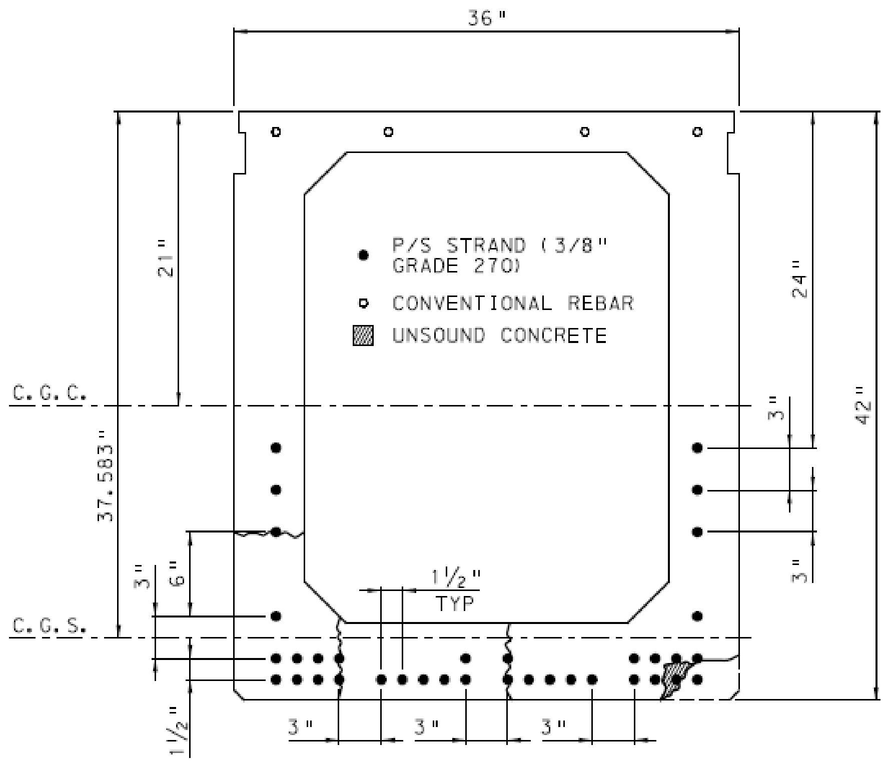


**Figure 1 – Original Beam with Deterioration**

| Original Beam | | | |
|---|---|---|---|
| Row | # Strands | Dist from Bot of Beam ($d_b$) | # Strands x $d_b$ |
| 1 | 18 | 1.5" | 27.0" |
| 2 | 10 | 3.0" | 30.0" |
| 3 | 2 | 6.0" | 12.0" |
| 4 | 2 | 12.0" | 24.0" |
| 5 | 2 | 15.0" | 30.0" |
| 6 | 2 | 18.0" | 36.0" |
| | | **C.G.S. =** | **4.417"** |
| | | **Strand Area =** | **0.085 in$^2$** |
| | | **# Strands =** | **36** |

**Table 1: Strand Layout and CGS of Original Beam**

**Bridge Rating Evaluation Example using Method A:**

Using Method A, a total of 12 strands would be removed from the analysis. Because the deterioration occurs in the middle 1/3 of the span, these strands are removed for the entire length of the beam. The strands above each crack in both rows are removed, as well as strands immediately adjacent to each crack in the bottom row. The strand adjacent to the crack on the side of the beam in addition to the crack 3” above this crack have been removed (Assume the strand 6” below is not affected). The strand exposed by the spall has been removed and the strand located in the delaminated concrete has been removed. The final strand pattern used for analysis is shown in Figure 2.

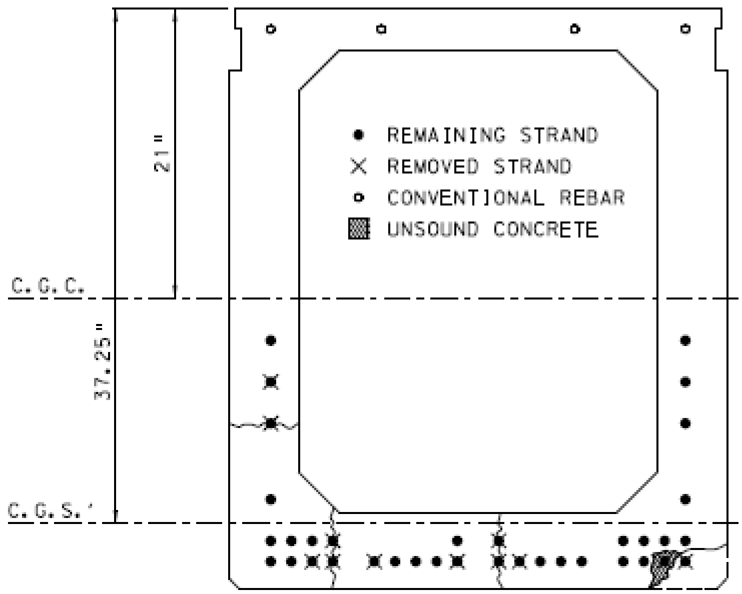


**Figure 2 – Strand Configuration for Method A Analysis**

| **Method A** | | | |
|---|---|---|---|
| Row | # Strands | Dist from Bot of Beam ($d_b$) | # Strands x $d_b$ |
| 1 | ***10*** | 1.5" | 15 |
| 2 | ***8*** | 3.0" | 24 |
| 3 | 2 | 6.0" | 12 |
| 4 | ***1*** | 12.0" | 12 |
| 5 | ***1*** | 15.0" | 15 |
| 6 | 2 | 18.0" | 36 |
| | | **C.G.S.' =** | **4.75"** |
| | | **Strand Area =** | **0.085 in$^2$** |
| | | **# Strands =** | **24** |

**Table 2: Strand Layout and CGS’ for analysis using Method A**

**Bridge Rating Evaluation Example using Method B:**

Using Method B, a total of 4 equivalent strands are removed from the analysis. The exposed strand and the strand within the delaminated area are removed from the analysis. The strands above and immediately adjacent to the cracks within 3" are reduced to 75% of their area, and all other strands were reduced to 95% of their area. The final strand pattern used for analysis is shown in Figure 3.

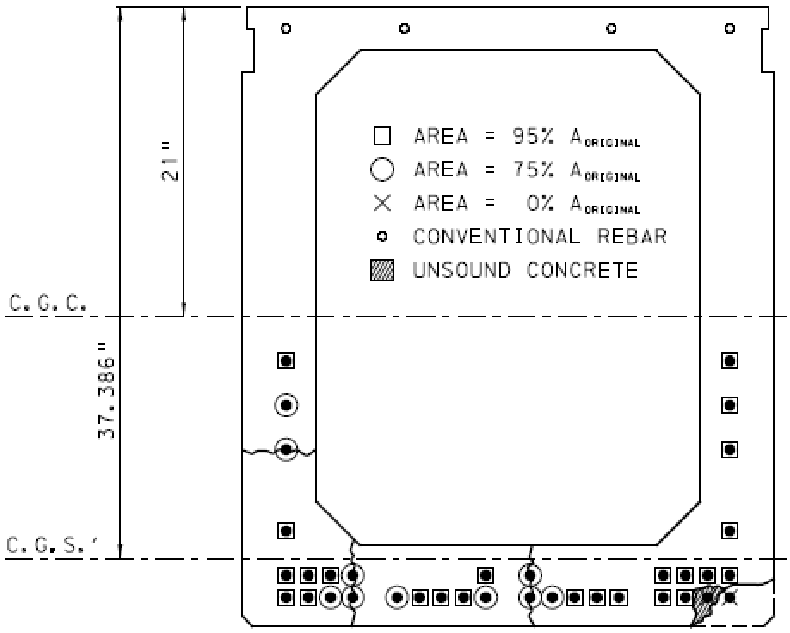


**Figure 3 – Strand Configuration for Method B Analysis**

Since the PS3 program does not accept a strand pattern with various individual strand areas, an equivalent C.G and area for the strands with section reduction needs to be determined for PS3 input. Table 3 shows the additional calculations required to provide this revised section.

| Method B | | | | | | |
|---|---|---|---|---|---|---|
| Row | Dist from Bot of Beam ($d_b$) | # Strands (0 % Area) | # Strands (75 % Area) | # Strands (95 % Area) | Remaining Area | Area x $d_b$ |
| 1 | 1.5" | 2 | 6 | 10 | 1.190 in$^2$ | 1.79 |
| 2 | 3.0" | 0 | 2 | 8 | 0.774 in$^2$ | 2.32 |
| 3 | 6.0" | 0 | 0 | 2 | 0.162 in$^2$ | 0.97 |
| 4 | 12.0" | 0 | 1 | 1 | 0.145 in$^2$ | 1.73 |
| 5 | 15.0" | 0 | 1 | 1 | 0.145 in$^2$ | 2.17 |
| 6 | 18.0" | 0 | 0 | 2 | 0.162 in$^2$ | 2.91 |
| | | | | | **C.G.S.' =** | **4.614"** |
| | | | | | **Strand Area =** | **0.085 in^2** |
| | | | | | **# Eqiv. Strands =** | **30.3** |

**Table 3: Strand Layout and CGS' for Analysis using Method B**

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# APPENDIX IE 04-E

Corrugated Metal Culvert
Inspection Forms

## CORRUGATED METAL ARCH CULVERT INSPECTION FORMS

District: ______________ Inspected by: ______________

BMS #: ______________________________ Inspection Date: ______________

### CULVERT GEOMETRY:

Height of Cover: ______________

Structure Length: ______________ Number of Barrels: ______________

Max Span Length: ______________ Max Barrel Length: ______________
(inlet to outlet)

Headwall Type: ______________ Wingwall Type: ______________

### DEFECTS:

Is roadway settlement or repair apparent? Yes ☐ No ☐

Roadway Settlement/Repair Notes: ______________________________

______________________________

______________________________

______________________________

Headwall and Wingwall Defects: ______________________________

______________________________

______________________________

______________________________

Shape: Is flattening or bulging present? Yes ☐ No ☐
If yes, is it caused by corrosion? Yes ☐ No ☐
Is this a change from the previous inspection? Yes ☐ No ☐

If flattening/bulging is present complete the following:

Location of defect: ______________________________

Length of defect (along barrel): ______________ Width of defect (along arc): ______________

Additional Shape Notes: ______________________________

______________________________

Maintenance Priority Codes for Corrosion Induced Flattening/Bulging:

> 3" distortion caused by corrosion = Code 0 ☐
> 2", ≤ 3" distortion caused by corrosion = Code 1 ☐
≤ 2" distortion caused by corrosion = Code 2 ☐

## CORRUGATED METAL ARCH CULVERT INSPECTION FORMS

District: ______________ Inspected by: ________________

BMS #: ______________________________ Inspection Date: ________________

## DEFECTS (continued):

Corrosion and Section Loss: Original Metal Thickness: ____________

Notes: ______________________________________________________________

______________________________________________________________

______________________________________________________________

______________________________________________________________

______________________________________________________________

______________________________________________________________

______________________________________________________________

***Worst 4' Length of Corrosion:***

Location: ________________________________________

\# of corrugations w/ holes: ___________________________

% of corrugations w/ holes: ___________________________

\# of corrugations w/ 0, 25, 50 or 75% section loss*: _______ w/ 0% Estimated Section Loss

*Account for all remaining corrugations in the 4' length.
This data is used to calculate remaining life below.

_______ w/ 25% Estimated Section Loss

_______ w/ 50% Estimated Section Loss

_______ w/ 75% Estimated Section Loss

% bolts missing/ineffective: ________________________________________

Consecutive bolts missing over 1' to 2' L? Yes ☐ No ☐

Maintenance Priority Codes for Corrosion (descriptions apply to worst 4' length documented above):

Severe corrosion (holes in ≥ 50% of corrugations) = Code 0 ☐
Serious corrosion (some minor holes) = Code 1 ☐
Advanced corrosion = Code 2 ☐

If Priority Code 2 is assigned above, estimate the remaining useful life of the culvert. To assist with this calculation there is a spreadsheet in the link on the BMS2 message board named: PennDOT Bridge Inspection Forms and Templates.

Maintenance Priority Codes for Bolt Deficiencies (descriptions apply to worst 4' length documented above):

Approximately 50% or more bolts/nuts missing or ineffective = Code 0 ☐
Consecutive bolts/nuts missing for a length of 1' to 2' = Code 1 ☐

## CORRUGATED METAL ARCH CULVERT INSPECTION FORMS

District: ________________ Inspected by: ________________

BMS #: ________________________________ Inspection Date: ________________

Provide a sketch of the culvert and stream including alignment, scour conditions, debris/blockage, erosion, defects, corrosion/section loss and associated dimensions as applicable.

## CORRUGATED METAL ARCH CULVERT INSPECTION FORMS

District: ________________ Inspected by: ________________

BMS #: ________________ Inspection Date: ________________

SUPPLEMENTAL INSPECTION NOTES/SKETCHES: