DATE: February 11, 2009

SUBJECT: Publication 238, Bridge Safety Inspection Manual
Revisions to Chapters 3 and 6

TO: District Executives

FROM: Brian G. Thompson, P.E. /s/
Director
Bureau of Design

This Strike-off Letter contains revisions to Chapters 3 and 6 of Publication 238, Bridge Safety Inspection Manual, to address the inspection and load rating of steel gusset plates. These revisions are effective immediately.

The attached pages are to be inserted into the October 2002 Edition of Publication 238:

▪ Part IE Table of Contents	Revision to page numbers
▪ Pages IE 03-3 to 03-14	Revision to Article 3.8.3.6 – "Trusses"and Addition of Article 3.8.3.6.1I – "Gusset Plates"
▪ Pages IE 03B-1 to 03B-4	Addition of Appendix 3B – "Truss Gusset Plate Inspection Forms"
▪ Pages IE 06-1 to 06-12	Revision to Article 6.1.6 – "Load Rating for Complex Structures" and Addition of Article 6.6.2.1.3I – "Gusset Plates in Truss Bridges"

If you have any questions, please contact Lance D. Savant, P.E., at (717) 783-7498.

Attachments

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Publication 238 Part IE, Table of Contents

PART IE: EVALUATION

Chapter 1 – Introduction

1.1 – Purpose 1-1
1.2 – Scope 1-1
1.3 – Applicability 1-1
1.4 – Quality Measures 1-2
1.5 – Bridge Management Systems (BMS) 1-2
1.6 – Definitions and Important References 1-3
1.6.1 – Definitions
1.6.2 – Important References

Chapter 2 – Bridge File (Records)

2.1 – General 2-1
2.2 – Components of Bridge Records 2-1
2.2.11 – Flood Data
2.2.18I – Observed Scour Rating (BMS Item W11A)
2.2.19I – Scour Critical Bridge Indicator (BMS Item W06 and E29A)
2.3 – Inventory Data 2-1
2.3.1 – General
2.3.2 – Revised inventory Data
2.4 – Inspection Data 2-3
2.4.2 – Revised Inspection Data
2.5 – Condition and Load Rating Data 2-3
2.5.2 – Revised Condition and Load Rating Data
2.6 – Local Requirements 2-4

Chapter 3 – Inspection

3.1 – General 3-1
3.2 – Types 3-1
3.2.1 – Initial Inspections
3.2.2 – Routine Inspections
3.2.3 – Damage Inspections
3.2.4 – In-Depth Inspections
3.2.5 – Special Inspections
3.3 – Frequency 3-1
3.4 – Qualifications and Responsibilities of Inspection Personnel 3-1
3.6 – Planning, Scheduling, and Equipment 3-2
3.6.3 – Equipment
3.6.3.2 – Inspection Methods and Equipment
3.7 – Inspection Forms and Reports 3-2
3.8 – Procedures 3-2
3.8.1 – General
3.8.1.2 – Cleaning
3.8.1.4 – Critical Deficiency Procedures
3.8.2 – Substructure
3.8.2.4 – Pile Bents
3.8.2.5 – Bridge Stability and Movements
3.8.2.6 – Dolphins and Fenders
3.8.3 – Superstructure
3.8.3.1 – Steel Beams, Girders, and Box Sections
3.8.3.2 – Reinforced Concrete, Beams and Girders
3.8.3.3 – Prestressed Concrete Beams, Girders, and Box Sections

3.8.3.3.1I – Adjacent Non-Composite Prestressed Concrete Box Beams
3.8.3.4 – Timber Systems
3.8.3.6 – Trusses
3.8.3.6.1I – Gusset Plates
3.8.3.7 – Cables
3.8.3.8 – Diaphragms and Cross Frames
3.8.3.9 – Lateral Bracing, Portals, and Sway Frames
3.8.3.11 – Pin and Hangers
3.8.3.12 – Bearings
3.8.3.12.1I – Rocker Bearings
3.8.3.15 – Arches
3.8.4 – Decks
3.8.4.1 – Concrete Decks
3.8.4.5 – Expansion Joints
3.8.4.6 – Railings, Sidewalks and Curbs
3.8.4.7 – Bridge Drainage
3.8.9 – Corrugated Metal Plate Structures
3.9 – Special Structures 3-13
3.9.4 – Prestressed Concrete Segmental Bridges
3.10 – Underwater Inspections 3-13
3.11 – Fatigue Prone Details 3-14
3.12 – Fracture Critical Members 3-14
Appendix 3AI – Rocker Bearing Movement Analysis 3A-1
Appendix 3B – Gusset Plate Inspection Forms 3B-1

Chapter 4 – Material Testing

4.1 – General 4-1

Chapter 5 – Non-Destructive load Testing

5.1I – Bridge Instrumentation Introduction 5-1
5.2I – Contracting Procedures 5-1
5.3I – Instrumentation 5-2
5.4I – Diagnostic Test Program 5-2
5.5I – Evaluation of Instrumentation Results 5-2
5.6I – Reporting 5-3
5.7I – Instrumentation Systems 5-3

Chapter 6 – Load Rating

6.1 – General 6-1
6.1.6 – Load Rating for Complex Structures
6.2 – Qualifications and Responsibilities 6-1
6.3 – Rating Levels 6-1
6.4 – Rating Methods 6-1
6.5 – Rating Equation 6-1
6.5.4 – Condition of Bridge Members
6.5.5 – Bridges with Unknown Structural Components
6.6 – Nominal Capacity 6-2
6.6.2.1 – Structural Steel
6.6.2.1.1– Combined Stresses
6.6.2.1.3I – Gusset Plates in Truss Bridges
6.6.3 – Load Factor Method
6.6.3.3 – Prestressed Concrete

6.7 – Loadings 6-11
6.7.1 – Dead Load
6.7.2 – Rating Live Load
6.8 – Documentation of Rating 6-11

Chapter 7 – Additional Considerations

7.1 – General 7-1
7.2 – Correlation of Inspection, Testing and Load Rating Data 7-1
7.3 – Fatigue Evaluation of Steel Bridges 7-1
7.4 – Posting of Bridges 7-1
7.4.1 – General
7.4.2 – Posting Loads
7.4.3 – Posting Analysis
7.4.4 – Regulatory Signs
7.4.5 – Speed Limits
7.5 – Permits 7-1
7.7 – Special Conditions 7-1

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REQUIREMENTS COMMENTS

Many bridge problems caused by corrosion and concrete deterioration have become emergencies because the structural deterioration was accelerated 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 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 D-450M inspection form.

3.8.1.4 CRITICAL DEFICIENCY PROCEDURES

"The following shall replace the first sentence of M 3.8.1.4".

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.

"The following paragraph shall be added to the end of M 3.8.1.4"

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.

3.8.2 Substructure

3.8.2.4 PILE BENTS

"The following shall supplement M 3.8.2.4".

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.

3.8.2.5 BRIDGE STABILITY AND MOVEMENTS

"Add the following at the end of paragraph 3".

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.

3.8.2.6 DOLPHINS AND FENDERS

"The following shall supplement M 3.8.2.6".

REQUIREMENTS COMMENTS

The Navigational Controls are to be inventoried and noted in BMS Items D12 and D12A. The conditions of these controls should be noted in the inspection report with the substructure unit(s) they protect.

3.8.3 Superstructure

"The following shall replace the first sentence of M 3.8.3".

This article includes discussions covering inspection of all commonly-encountered types of superstructures composed of prestressed concrete, reinforced concrete, structural steel, iron or timber, including bearings, connection devices, and protective coatings.

3.8.3.1 STEEL BEAMS, GIRDERS AND BOX SECTIONS

"The following shall supplement M 3.8.3.1".

Guidance and requirements for the inspection of steel bridges considering fatigue and fracture is presented in IP 2.4.

3.8.3.2 REINFORCED CONCRETE BEAMS AND GIRDERS

"The following shall supplement the first paragraph of M 3.8.3.2".

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.

3.8.3.3 PRESTRESSED CONCRETE, BEAMS, GIRDERS AND BOX SECTIONS

"The following shall supplement the first paragraph of M 3.8.3.3".

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 pre-stressed beam bridges made continuous for live load are varied, the design, shop drawings, and construction records should be carefully reviewed for the inspection.

IC3.8.3.3 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.

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.

3.8.3.3.1I 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:

IC3.8.3.3.1I 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

REQUIREMENTS

Beam Spalls and/or Delaminations:

- Location on beam
- 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

Other General Information:

- Web cracks - number, width , orientation, and location. **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. Notify BQAD immediately to assist in the evaluation.**
- Flange cracks - number, width, orientation, and location
- 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.

Plan and Cross-Section Sketches of Beams

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, use the following symbols on the beam cross-section for documentation during inspection and analysis:

- • Strands still effective
- o Strands presumed (not known) to be not effective
- x Lost strand (Broken or corroded). Exposed strands shall be considered as "lost" unless corrosion is minimal (mostly shiny surface).

Adjacent non-composite prestressed concrete box beam bridges with damaged strands or concrete shall be considered high priority for inspection and ratings.

COMMENTS

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 a bituminous 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 post-tensioning 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 of reduced beam stiffness or stress concentration.

3.8.3.4 TIMBER SYSTEMS

"The following shall supplement M 3.8.3.4".

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 prestressing (post-tensioning) high strength steel bars. This prestressing

IC3.8.3.4 The Transportation Research Record 1740 Paper No. 00-1191 entitled "Field Performance of Stress-Laminated Timber Bridges" provides a good overview

REQUIREMENTS

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 prestressing, 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 the bridge are:

- Live Load Deflection –Should be limited to L/500 as recommended in the AASHTO Specifications.
- Bar Force in the Pre-stressing 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.

3.8.3.6 TRUSSES

"The following shall supplement the second paragraph of M 3.8.3.6".

Check for global buckling of the truss compression member along its length and also check for localized buckling of the 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 3B for truss inspection forms

3.8.3.6.1I 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.

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.

COMMENTS

of this bridge type and was the source for the recommendations in the second paragraph.

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 gusset plate for cracks emanating from the fastener head. 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.

3.8.3.7 CABLES

“The following shall supplement the second paragraph of M 3.8.3.7”.

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.

3.8.3.8 DIAPHRAGMS AND CROSS FRAMES

“The following shall supplement M 3.8.3.8”.

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.

Straight girder bridges designed using FEM analysis may also contain diaphragms that act as main structural members and should be considered as such in inspection and ratings.

3.8.3.9 LATERAL BRACING, PORTALS AND SWAY FRAMES

“The following shall supplement M 3.8.3.9”.

Note any missing or deteriorated connection bolts or rivets.

REQUIREMENTS

COMMENTS

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 2.4.5 for additional guidance on such details.

3.8.3.11 PIN AND HANGERS

"The following shall supplement M 3.8.3.11".

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.

3.8.3.12 BEARINGS

"The following shall supplement M 3.8.3.12".

Abnormal or unusual gap measurements at deck expansion joints may be an indication of frozen or improperly functioning bearings as described in Article 3.8.3.12.1. 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.

3.8.3.12.1I 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 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 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.

IC 3.8.3.12.1I 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 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 70 feet) and thus relatively flexible compared to adjacent piers
- Inspection documentation over several cycles recorded the bearings being oriented in a parallel displacement

REQUIREMENTS

COMMENTS

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 with two lines of expansion bearings is considered a critical deficiency.

configuration instead of diverging or converging.

Critical and High Priority deficiencies found during the inspection should be documented appropriately with photographs and the required information obtained in Appendix 3A. Critical Deficiencies should be brought to the attention of the bridge owner immediately in accordance with Article 3.8.1.4. Additionally, for every inspection performed on bridges having rocker bearings, the information in Appendix 3A 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.

When steel rocker bearings at a pier are in the contracted position (tilted toward the fixed bearing) in warm weather (above 68° F), in the expanded position (tilted away from the fixed bearing) in cold weather (below 68°) or parallel tilt for two lines of rocker bearings at adjacent spans on a common support, a re-inspection of the steel rocker bearings within six months from the original inspection date during extreme temperature is required. 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. The purpose of the re-inspection during extreme temperature is to ensure that the rocker bearings are not exceeding the acceptable tilt limit.

The amount of allowable tilt varies with respect to bearing geometry, span length, bridge type, and ambient temperature. To compare the actual tilt to the allowable tilt, the inspector should determine the allowable tilt from the contract plans. If no plans are available, the inspector should determine an acceptable tilt from the actual rocker measurements (see Appendix 3A). In addition, the inspector must complete the tables included in Appendix 3A for initial, routine, in-depth and special inspections. Contact the BQAD to obtain a spreadsheet that determines the adequacy of the observed bearing with regard to tilt.

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 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.

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 critical and high priority deficiencies and take action as indicated:

1. Two lines of expansion rocker bearings on a pier that exhibits excessive tilt, bears on the outer one-quarter width of the rocker, as previously described, assign the following:

    - IM03 Action – Flexible Action A744501 Bearings - “Steel (Replace/Rehab)”

    - IM05 Priority – Assign Code 0, Critical

    - IM15 Notes – Note that the bearings exceed the acceptable limit of tilt or bears on the outer one-quarter width of the rocker. Also note that the two lines of bearings on the pier are expansion/expansion.

    Remediation options include (but are not limited to):

- Temporary supports and appropriate monitoring frequency (note: this option does not alleviate unintended transfer of horizontal force to the substructure if bearing freedom of movement is lost).

- 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.

- Replacement.

2. Rocker bearing movement analysis (Appendix 3A) where two lines of expansion bearings on a pier indicates a potential for the bearings to reach or exceed the outer one-quarter limit or parallel tilt for two lines of expansion rocker bearings at adjacent spans on a common support; assign the following:

   - IM03 Action – Flexible Action C744502 Bearings – “Expansion (Reset)”

   - IM05 Priority – Assign Code 1, High Priority (1 to 6 months)

   - IM15 Notes – Note that the rocker bearing movement analysis for two lines of expansion bearings at a pier indicates a potential for the bearing to reach or exceed the outer one-quarter limit and must be regularly monitored. Note whether the rocker bearings are in an expanded position in cold weather, in a contracted position in warm weather or parallel tilt for two lines of expansion rocker bearings at adjacent spans on a common support. The severity of the situation will dictate the frequency of monitoring. A special inspection must be scheduled.

   - IM07 Status – The status may be set to “Deferred” when a re-inspection is to be performed.

3. Rocker bearings located on piers with heavy accumulations of pack rust, corrosion, and/or debris under the rocker could potentially limit or prevent the bearing from operating as it was intended during structure expansion and contraction; assign the following:

   - IM03 Action – Flexible Action C743102 Flush - “Bearing/Bearing Seat”

   - IM05 Priority – Assign Code 1, High Priority (1 to 6 months)

   - IM15 Notes – Note if pack rust, corrosion, and/or debris under the rocker could potentially be limiting or prevent the bearing from operating as it was intended during structure expansion and contraction. In addition to “flushing”, it may be necessary to remove pack rust by mechanical means.

4. Rocker bearing measurement or movement analysis (Appendix 3A) indicates the rocker bearing reached or exceeds the outer one-tenth limit at abutments or the outer one-quarter limit at piers with fixed/expansion bearings, 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) outside the minimum and maximum range determined from the movement analysis; assign the following:

   - IM03 Action – Flexible Action C744502 Bearings – “Expansion (Reset)”
   - IM05 Priority – Assign Code 1, High Priority
   - IM15 Notes – Note that the rocker bearing measurement or movement analysis indicates the bearing reached or exceeds the outer one-tenth limit at abutments or the one-quarter limit at piers with fixed/expansion bearings. Note whether the rocker bearings are in an expanded position in cold weather or in a contracted position in warm weather. Note the differences in movement analysis results compared to the actual field measurements.

All repairs and superstructure jacking procedures must be prepared, signed, and sealed by a Professional Engineer licensed in the Commonwealth of Pennsylvania.

3.8.3.15 ARCHES

“The following shall supplement M 3.8.3.13”.

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 fine materials in the fill.

3.8.4 Decks

“The following shall supplement M 3.8.4”.

3.8.4.1 CONCRETE DECKS

“The following shall supplement M 3.8.4.1”.

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 (BMS Item E17). If the box beams have been covered by a bituminous 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

REQUIREMENTS | COMMENTS

not be higher than the wearing surface condition rating unless there is strong evidence to support otherwise.

3.8.4.5 EXPANSION JOINTS

"The following shall supplement M 3.8.4.5".

Reinforced elastomeric joints are composed of various proprietary combinations of steel supports and sealant material. Inspect for missing anchor bolt covers, separation of joint elements, and audible or visual evidence of loose panels under traffic. Loose panels should be repaired immediately because the bolt failure is progressive and may result in one of the joint panels breaking loose under traffic.

IC3.8.4.5 Refer to DM4 PD 14.5. (Superstructure Joints) for background theory used in designing the bridge deck joint openings. It is important to understand this theory in developing the appropriate information needed to monitor the joint.
Refer to DM4 PP 3.6.1.1 (Backwalls and Concrete End Diaphragms) for additional formulae that can be used (in addition to determining if a backwall is needed) to calculate the deck joint openings for the fixed end and the expansion end of the superstructure.

Modular joints are composed of single or multiple support systems working together to accommodate large bridge movements. Inspect for surface damage to seals and separation beams. Examine underside for evidence of leakage and unusual noise, which may indicate fractured welds or bolts.

Debris in joints causes damage to the joint and a maintenance need for cleaning flushing deck should be recorded to clean the joint.

3.8.4.6 RAILINGS, SIDEWALKS AND CURBS

Additional requirements for PA bridges are contained in BMS Coding Manual (Pub 100A) BMS Item E28A.

3.8.4.7 BRIDGE DRAINAGE

"The following shall supplement M3.8.4.7".

Drainage deficiencies on non-redundant structures, especially those with FCMs shall be given a high priority for maintenance.

3.8.9 Corrugated Metal Plate Structures

Additional requirements for PA bridges are contained in IP 2.5.2.

3.9 SPECIAL STRUCTURES

3.9.4 Prestressed Concrete Segmental Bridges

"The following shall supplement M 3.9.4".

Check for corrosion staining especially at segment joints. Note any clogged drain holes or any standing water in box sections.

3.10 UNDERWATER INSPECTIONS

Additional requirements for PA bridges are contained in IP 2.6.

REQUIREMENTS COMMENTS

3.11 FATIGUE PRONE DETAILS

Additional requirements for PA bridges are contained in IP 2.4.

3.12 FRACTURE CRITICAL MEMBERS

Additional requirements for PA bridges are contained in IP 2.4

Appendix 3B - Gusset Plate Inspection Forms

BMS ID:________
Insp. Date:______
Insp. By:_______

Checked By:_______
Date:_______

Deck 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

Page____ of ____

Figure 3B-1 Truss Members and Floorbeam Ends

BMS ID:________________
Insp. Date:______________
Insp. By:________________

Checked By:_______________
Date:___________________

Deck 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.	Comments

Page _____ of _____

Figure 3B-2 Truss Panel Point Connection

BMS ID:________
Insp. Date:________
Insp. By:________

Checked By:________
Date:________

Deck Truss Inspection Findings Summary
Bearing Condition

Sub Unit	Truss Line	Type	Fixed/Exp (F or E)	Frozen (Y/N/NA)	Debris (Y/N)	Corrosion (Y/N)	Over Extended per SOL 431-08-12 (Y/N/NA)	Photo No.	Comments

Page _____ of _____

Figure 3B-3 Bearing Condition

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REQUIREMENTS COMMENTS

6.1 GENERAL

Additional requirements for PA bridges are contained in IP 3.

6.1.6 Load Rating for Complex Structures

Additional requirements for PA bridges are contained in IP 3.3.3 and IP 3.3.4.

6.2 QUALIFICATIONS AND RESPONSIBILITIES

Additional requirements for PA bridges are contained in IP 2.

6.3 RATING LEVELS

Additional requirements for PA bridges are contained in IP 3.

6.4 RATING METHODS

Additional requirements for PA bridges are contained in IP 3.

6.5 RATING EQUATION

Additional requirements for PA bridges are contained in IP 3.

6.5.4 Condition of Bridge Members

"The following shall supplement M 6.5.4".

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.

6.5.5 Bridges with Unknown Structural Components

"The following shall replace the first sentence of M 6.5.5".

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 qualified engineer is sufficient to determine the Inventory and Operating ratings. These rating shall be recorded in the BMS and the bridge inspection as engineering judgement.

IC6.5.5 Engineering Judgment is an acceptable method for determining Inventory and Operating ratings. See Part IP, Section 3.5

6.6 NOMINAL CAPACITY

Additional requirements for PA bridges are contained in IP 3.

REQUIREMENTS

COMMENTS

6.6.2.1 STRUCTURAL STEEL

"The following shall replace the first sentence of M 6.6.2.1".

The allowable unit stresses used for determining the Inventory Ratings and Operating Ratings depend on the type of steel used in the structural members.

6.6.2.1.1 Combined Stresses

"The following shall replace the first sentence of M 6.6.2.1.1".

The allowable combined stresses for steel compression members may be calculated by the provisions of the AASHTO Standard Design Specifications as modified below or by the procedure contained in Appendix A11.

"The following shall replace the formula for compression in concentrically loaded columns in Table 6.6.2.1-2"

With $C_c = \sqrt{2\pi^2 E / F_y}$

$$F_a = F_y / F.S. \left[1 - \left[\left[(KL/r)^2 F_y \right] / 4\pi^2 E \right] \right] \quad \text{when } KL/r \le C_c$$

$$F_a = \pi^2 E / \left[F.S. (KL/r)^2 \right] = 168{,}363{,}840 / (KL/r)^2 \quad \text{when } KL/r \ge C_c$$

With F.S. = 1.70

IC6.6.2.1 Safe load capacity is discussed under bridge postings, Part IP, Section 4.3.2.

IC6.6.2.1.1 AASHTO Design Specification is clarified to AASHTO Standard Design Specification to refer to the LFD and ASD methodologies rather than the LRFD methodology.

6.6.2.1.3I Gusset Plates in Truss Bridges

Gusset connections of non-redundant load path steel truss bridges shall be evaluated during a bridge load rating analysis. Non-redundant load path bridges are those with no alternate load paths and whose failure of a main component is expected to result in the collapse of the bridge.

The evaluation of gusset connections shall include the evaluation of the connecting plates and fasteners. The capacity of a gusset connection is determined as the smaller capacity of the fasteners or gusset plates.

Use PennDOT Gusset Plate Analysis and Ratings spreadsheet until BAR 7 is revised. The maximum loads are the loadings specified in Section 6.1.

6.6.2.1.3.1 I Capacity of Fasteners

For concentrically loaded bolted and riveted gusset connections, the maximum axial load in each connected member may be assumed to be distributed equally to all fasteners.

At maximum loads, the fasteners in bolted and riveted gusset connections shall be evaluated to prevent fastener shear and plate bearing failures. The provisions of SD Article 10.56.1.3.2 shall apply for determining the capacity of fasteners to prevent fastener shear and plate bearing failures for the LFD methodology.

For unknown rivet types, the shear capacity of one rivet shall be taken as:

$$\phi R = \phi F m A_r$$

where:

ϕF = shear strength of one rivet. The values in Table IE-6.6.2.1.3.1-1 may be used for ϕF based on the year of construction:

Table IE-6.6.2.1.3.1-1

Year of Construction	ϕF (ksi)
Constructed prior to 1936 or of unknown origin	18
Constructed after 1936 but of unknown origin	21

m = the number of shear planes
A_r = cross-sectional area of the rivet before driving (in^2)

The shear capacity of a rivet in connections greater than 50.0 in. in length shall be taken as 0.80 times the value given in the above equation.

6.6.2.1.3.2 I Capacity of Gusset Plates

The capacity of a gusset plate shall be determined as the least capacity of the plate in tension including block shear, shear, and compression.

6.6.2.1.3.2.1I Gusset Plates in Tension

Gusset plates subjected to axial tension shall be investigated for two conditions:

- Yield on the effective gross section, and
- Block shear rupture

The capacity for gusset plates in tension for the LFD methodology, R_r, shall be taken as the least of the values given by either yielding on the effective area or the block shear rupture capacity.

Effective Gross Section Yielding

$$R_r = A_e F_y$$

where:

A_e = effective gross cross-sectional area taking into account the possibility of net section fracture (in^2).

$A_e = A_n + \beta A_g \leq A_g$

A_n = net cross-sectional area of the plates as specified in SD Article 10.16.14 (in^2).

β = 0.0 for M 270 Grade 100/100W steels, or when holes exceed 1¼ inch in diameter.
= 0.15 for all other steels and when holes are less than or equal to 1¼ inch in diameter.

A_g = gross cross-sectional area of the plates (in^2).

F_y = minimum yield strength of the plates, as specified in SD Table 10.2A (ksi).

For the determination of the gross and net section areas, the effective width of the gusset plate in tension may be determined by the Whitmore method. In this method, the effective width is measured across the last row of fasteners in the connection under consideration. The effective width is bound on either side by the closer of the nearest adjacent plate edges or lines constructed starting from the external fasteners within the first row and extending from these fasteners at an angle of 30 degrees with respect to the line of action of the axial force. Figures IE-6.6.2.1.3.2.1-1 and IE-6.6.2.1.3.2.1-2 provide examples for determining the effective width in tension in accordance with the Whitmore method.

REQUIREMENTS COMMENTS

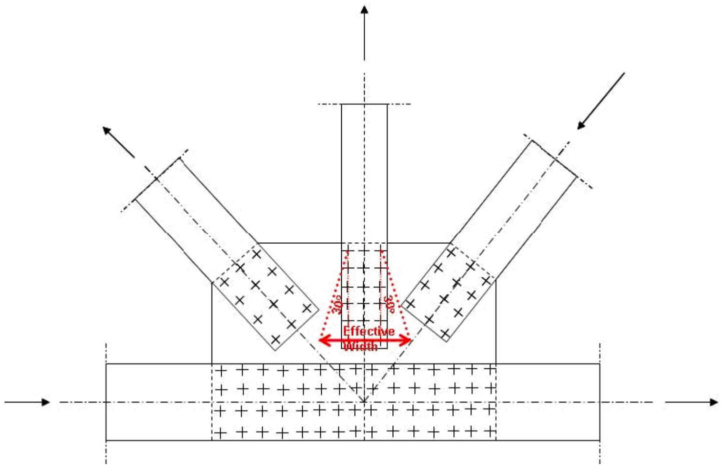


Figure IE-6.6.2.1.3.2.1-1 – Example 1 for using the Whitmore method to determine the effective width in tension

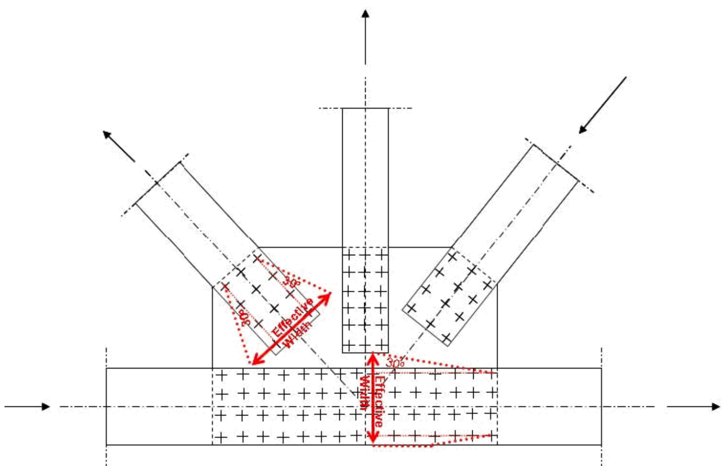


Figure IE-6.6.2.1.3.2.1-2 – Example 2 for using the Whitmore method to determine the effective width in tension

Block Shear Rupture

The resistance to block shear rupture is that resulting from the combined resistance of parallel and perpendicular planes; one in axial tension and the others in shear. The resistance of the plate for block shear rupture shall be taken as:

- If $A_{tn} \geq 0.58A_{vn}$, then $R_r = 0.85\left(0.58F_yA_{vg} + F_uA_{tn}\right)$
- Otherwise: $R_r = 0.85\left(0.58F_uA_{vn} + F_yA_{tg}\right)$

Where:

0.85 = resistance factor for block shear. This value is calculated as the LRFD resistance factor for net section tension fracture (0.8) divided by the resistance factor for gross section tension yielding (0.95)
A_{vg} = gross area along the plane resisting shear stress (in^2)
A_{tg} = gross area along the plane resisting tension stress (in^2)
A_{vn} = net area along the plane resisting shear stress (in^2)
A_{tn} = net area along the plane resisting tension stress (in^2)

F_y = minimum yield strength of the plate, as specified in SD Table 10.2A (ksi)
F_u = minimum tensile strength of the plate, as specified in SD Table 10.2A (ksi)

The analysis of block shear rupture involves the evaluation of several patterns of planes to arrive at the governing pattern. Figure IE-6.6.2.1.3.2.1-3 provides some examples of potential block shear rupture planes for gusset plates in tension.

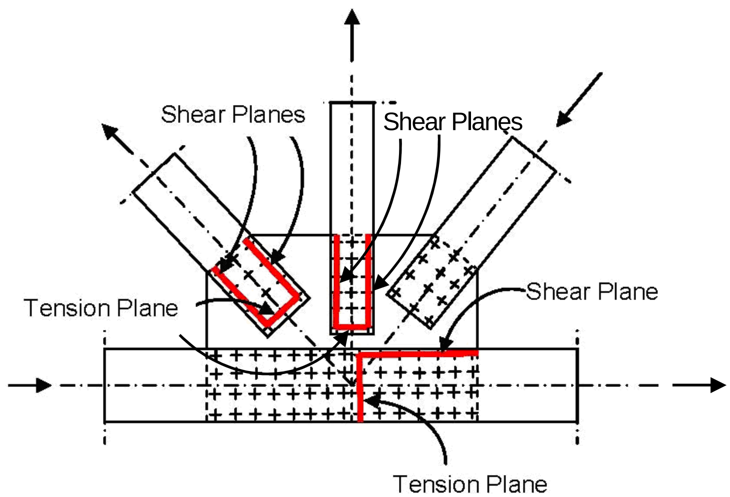


Figure IE-6.6.2.1.3.2.1-3 – Examples of potential block shear rupture planes for gusset plates in tension

6.6.2.1.3.2.2I Gusset Plates Subject to Shear

The shear capacity for the LFD methodology, R_r, for gusset plates subject to shear shall be taken as the lesser of the shear yield and the shear fracture resistance:

Shear Yield (kips):

$$R_r = 0.58 F_y A_g \times \Omega$$

Shear Fracture (kips):

$$R_r = 0.85 \times 0.58 F_u A_n$$

where:

0.85 = resistance factor for shear fracture on the net section. This value is calculated as the LRFD resistance factor for net section tension fracture (0.8) divided by the resistance factor for gross section tension yielding (0.95)
A_g = gross area of the plates resisting shear (in^2)
A_n = net area of the plates resisting shear (in^2)
F_y = minimum yield strength of the plates (ksi)
F_u = minimum tensile strength of the plates (ksi)
Ω = reduction factor taken as:

REQUIREMENTS

- Ω = 1.00 for the case of uniform shear stress distribution where the gusset plates are of ample stiffness to prevent buckling and develop the plastic shear force of the plates, or
- Ω = 0.74 for the case of flexural shear stress distribution, and in the absence of a more rigorous analysis or criterion to assure and quantify the stiffness requirements to develop the plastic shear force of the plates.

The analysis of gusset plates for shear involves the evaluation of several shear sections to arrive at the governing section. Figures IE-6.6.2.1.3.2.2-1 and IE-6.6.2.1.3.2.2-2 provide examples of shear sections to be evaluated in gusset plates in gross section shear yielding and net section shear fracture.

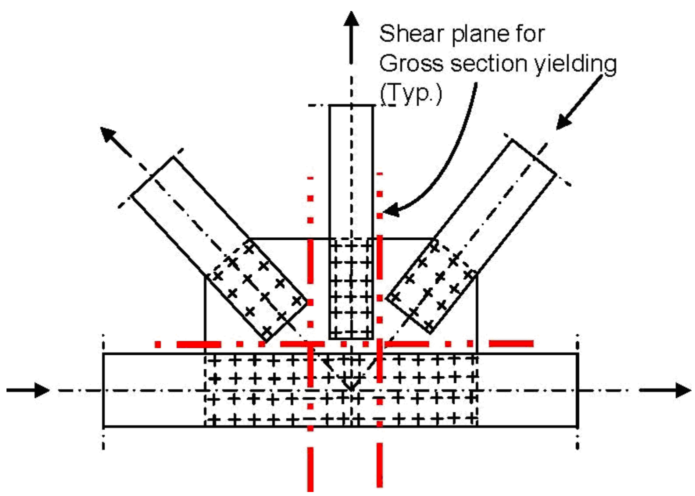


Figure IE-6.6.2.1.3.2.2-1 – Examples of gross section shear yielding planes

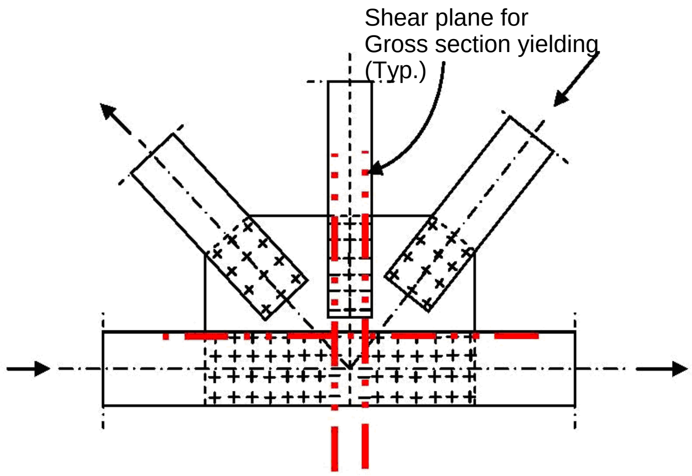


Figure IE-6.6.2.1.3.2.2-2 – Examples of net section shear fracture planes

COMMENTS

IC 6.6.2.1.3.2.2I
For PennDOT Truss Gusset Plate Analysis and Ratings spreadsheet, the reduction factor, Ω, was taken as 0.74.

6.6.2.1.3.2.3I Gusset Plates in Compression

The proximity of connected members, complex state of stress, and boundary conditions can influence the capacity of gusset plates in compression. Therefore, special care must be exercised to properly assess the buckled shape and compressive capacity of gusset plates in compression.

In the absence of a more rigorous analysis, the capacity of gusset plates in compression for the LFD methodology may be determined as that of idealized members in compression, in accordance with the provisions of SD Article 10.54.1.1.

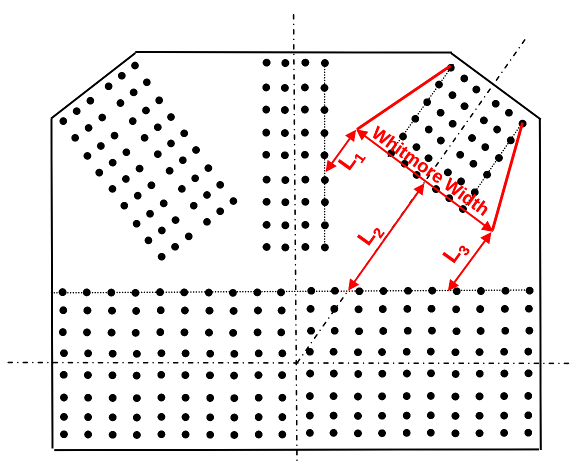
The effective width of the idealized compression member may be determined in accordance with the Whitmore method. The unbraced length, L_c, may be determined as the average of three distances (L_1, L_2, L_3) as follows:

where:

L_2 = The distance from the last row of fasteners in the compression member under consideration to the first row of fasteners in the closest adjacent member, measured along the line of action of the compressive axial force.

L_1, L_3 = The distance from each of the ends of the Whitmore width to the first row of fasteners in the closest adjacent member, measured parallel to the line of action of the compressive axial force. When the Whitmore width enters into the adjacent member, the associated distance at that end should be set to zero.

Figure IE-6.6.2.1.3.2.3-1 provides an example showing L_1, L_2, L_3, and effective width for a gusset plate in compression.



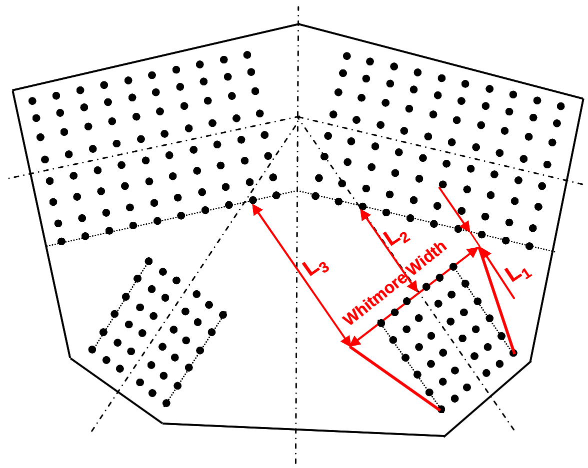
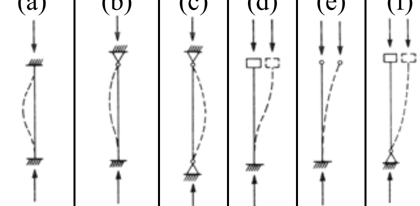


Figure IE-6.6.2.1.3.2.3-1 – Examples showing L_1, L_2, L_3, and effective width for a gusset plate in compression.

When lateral sway of gusset plates is possible, the effective length factor, K, for gusset plates may be taken from Table IE-6.6.2.1.3.2.3-1 for Cases (d), (e), or (f), depending on the anticipated buckled shape. When lateral sway of gusset plates is not possible, the effective length factor, K, for gusset plates may be taken from Table IE-6.6.2.1.3.2.3-1 for Cases (a), (b), or (c), as appropriate

IC 6.6.2.1.3.2.3I
For PennDOT Truss Gusset Plate Analysis and Ratings spreadsheet, the effective length factor, K, is taken as 2.0 if the gusset plate analysis shows that the plate has yielded due to shear on the horizontal section shown in Figure IE-6.6.2.1.3.2.2-1, otherwise it is taken as 1.2.

Table IE-6.6.2.1.3.2.3-1 – K Values



	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Design *K* value	0.65	0.80	1.0	1.2	2.1	2.0

6.6.2.1.3.2.4I   Gusset Plates under Combined Flexural and Axial Loads

Gusset plates behave as deep members. Therefore, the application of flexural theory to the analysis of gusset plates is not required.

6.6.2.1.3.3 I   Limiting Slenderness Ratio

The existing requirement of length-to-thickness ratio (for the design of unsupported edges of gusset plates) not to exceed $11{,}000/\sqrt{F_y}$ is equivalent to the slenderness ratio requirement of $\frac{l}{r} \leq 200$ for Grade 36 tension members not subject to stress reversal. Although an appropriate slenderness limit is advisable for the design of new gusset plates, it is not

REQUIREMENTS | COMMENTS

required in this guidance for load rating purposes.

If the limit is violated, assess for any distortion during inspections and provide stiffening in conjunction with the rehabilitation project.

6.6.3 Load Factor Method

"The following shall replace the first sentence in the second paragraph of M 6.6.3".

Allowable fatigue strength should be checked based on the AASHTO Standard Specifications or the AASHTO LRFD Specifications.

6.6.3.3 PRESTRESSED CONCRETE

"The following shall replace the equation for the reduction factor *k* in M 6.6.3.3".

k = the larger of the following two values:

$$\Phi M_n / 1.2 M_{cr}$$

Or

$$\Phi M_n / (4/3) M_u$$

IC6.6.3.3 This is in line with the AASHTO Standard Specifications for Highway Bridges, Article 9.18.2.2 in the 1998 Interim Revisions. The minimum steel requirement (of M6.6.3.3) restricts the nominal moment capacity of prestressed concrete beams to (k)(Φ)(M_n) when $\Phi M_n < 1.2M_{cr}$ (where $k = \Phi M_n/1.2M_{cr}$) which causes very low ratings for beams especially at beam ends where ΦM_n will most always be less than $1.2M_{cr}$.

6.6.3.3.1I ADJACENT NON-COMPOSITE PRESTRESSED CONCRETE BOX BEAMS

Load ratings of beams with deteriorated and/or damaged prestressing strands are to be based on the following procedures:

- 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.
- 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 BQAD 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 BQAD for further instructions.
- 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 BQAD.
- These analysis methods may also be applicable to other pre-stressed box beam bridges

IC 6.6.3.3.1I 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 stands 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.

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.

6.7 LOADINGS

Additional requirements for PA bridges are contained in IP 3

6.7.1 Dead Load

"The following shall supplement the second paragraph of M 6.7.1".

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 the structure was built using Shored construction, the composite section may be used to carry the superimposed dead load and the live load.
- If the structure was built using Unshored construction, the non-composite section is to be used to carry the superimposed dead load and the live load.
- If the Deck or Superstructure (BMS Item E17 or E18) is in poor condition, the non-composite section is to be used to carry the superimposed dead load and the live load regardless of the construction method used to build the structure.

IC6.7.1 See BAR7 computer program documentation for discussion regarding EIB beams and their analysis.

"Add the following paragraph at the end of M 6.7.1".

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.

6.7.2 Rating Live Load

Additional requirements for PA bridges are contained in IP 3.2.2

6.7.3 Distribution of Loads

"Add the following paragraph at the end of M 6.7.3".

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.

6.8 DOCUMENTATION OF RATING

Additional requirements for PA bridges are contained in IP 8.

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